RAPID EXCAVATION and TUNNELING CONFERENCE
2013 PROCEEDINGS

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On the Cover:
Inside Tunnel A Launch Block
Queens Bored Tunnels & Structures
Granite - Traylor - Frontier/Kemper JV

PREFACE

Every two years, industry leaders and practitioners from around the world gather at the Rapid Excavation and Tunneling Conference (RETC), the authoritative program for the tunneling profession, to learn about the most recent advances and breakthroughs in this unique field. This comprehensive book includes more than 100 papers from industry experts, highlighting their most recent projects and sharing real-world experiences that will keep you up to date on the latest tunneling trends and technologies.

The Washington, DC, location of this 21st RETC has a rich history of tunneling that has been used for transportation and vital infrastructure.

Design and construction of the Washington Metropolitan Area Transit Authority (WMATA) system in the District of Columbia and surrounding areas in the latter part of the last century helped to grow and expand the tunneling industry in the United States. The many landmark projects on the WMATA system provide an excellent opportunity for individuals to obtain first-hand experience with the state-of-the-art practices utilized at that time. This 103-mile rapid transit system is now the second-busiest rail transit system in the United States.

The design and construction of challenging tunnel projects continues in the DC area, with developments such as the recently completed Tysons Corner tunnels on Phase I of the Dulles Corridor Metro Rail Project and tunnels currently under construction on the Clean Rivers project.

The tunneling industry is experiencing significant technological advances, enabling the design and construction of increasingly challenging and complex projects. From caverns and large spans, contracting practices, tunnel linings, and design and planning to geotechnical considerations and instrumentation, ground stabilization, equipment applications, and risk management, this proceedings has the cutting-edge and innovative information you should have to meet the needs of the ever-important tunneling field.

We extend our sincere appreciation to the session chairs, the session co-chairs, the authors, and the members of the RETC Executive Committee for volunteering their valuable time. We also extend a special thanks to the staff at SME for their dedication and enthusiastic support.

Michael A. DiPonio
Chris Dixon

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Caverns and Large Spans

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DESIGN AND CONSTRUCTION OF 86TH ST. STATION ROCK CAVERNS IN NEW YORK

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ABSTRACT

The Second Avenue Subway Project is a major capital expansion project of the New York City subway that will provide a dedicated line for the east side of Manhattan with a link to the existing subway network. The proposed alignment runs from Harlem in the north to the financial district in the south with possible extension to Brooklyn. The project is approximately 13.7 km long including 16 stations, and its estimated cost is about $17 billion. Under the current design of the whole subway route, 10 stations will be cut-and-cover and 6 mined caverns which will be constructed through vertical shafts within the right-of-way of Second Avenue. In addition, there are numerous multi-track tunnels, crossovers and connections that will be constructed in caverns. The excavated diameter of the bored tunnels is 6.6 m and the caverns span ranges from 12.0 m to 21.0 m. All caverns have rock cover less than their span. As the geology of Manhattan varies along its length, the subway will pass through both hard rock and soft ground and there will be multiple rock/soil interfaces along the alignment. The final engineering of the project was undertaken for the New York City Transit Authority by an AECOM led joint venture and the construction of the 86th St. Station, which is the focus of this paper, is currently underway by a joint venture of Skanska, and Traylor Bros.

INTRODUCTION

The Second Avenue Subway Project is a major capital expansion project of the New York City subway that will provide a dedicated line for the east side of Manhattan with a link to the existing subway network. The proposed alignment runs from Harlem in the north to the financial district in the south with possible extension to Brooklyn. The project is approximately 13.7 km long including 16 stations, and its estimated cost is about $ 17 billion. Under the current design of the whole subway route, 10 stations will be cut-and-cover and 6 mined caverns which will be constructed through vertical shafts within the right-of-way of Second Avenue. In addition, there are numerous multi-track tunnels, crossovers and connections that will be constructed in caverns. The excavated diameter of the bored tunnels is 6.6 m and the caverns span ranges from 12.0 m to 21.0 m. All caverns have rock cover less than their span. As the geology of Manhattan varies along its length, the subway will pass through both hard rock and soft ground and there will be multiple rock/soil interfaces along the alignment.

The philosophy behind the construction methodology was to minimize the impact at street level on neighboring communities and businesses during the construction period. With most of the work being done within the right-of-way of Second Avenue, the largest impacts will be related to the maintenance and protection of traffic and street restoration as travel lanes are reduced from six to four during the construction. Because of the nature of the work, close proximity of high rise buildings, critical nature

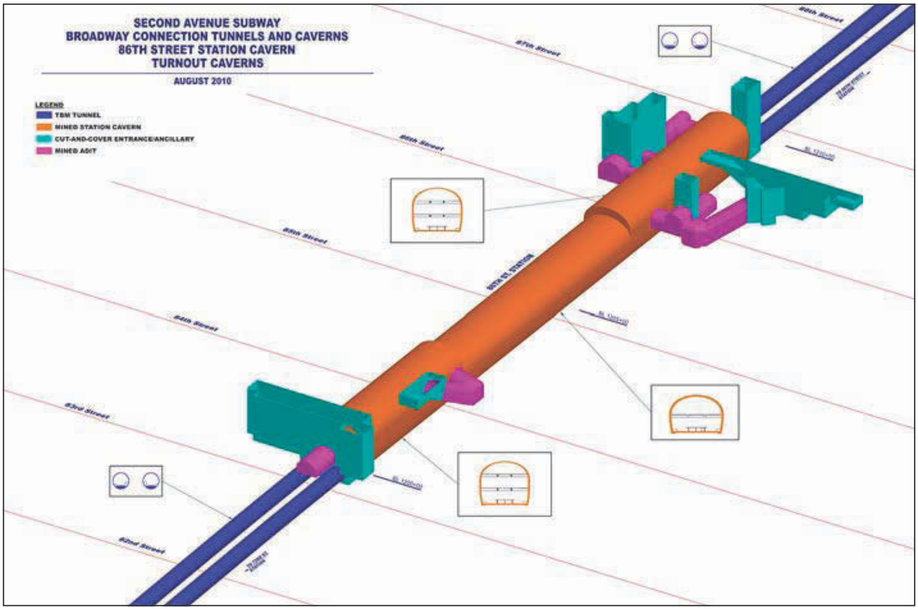

Figure 1. 3D model of 86th Street Station

of adjacent utilities, the characteristics of the ground along alignment, and the visibility of this project, strict performance criteria and limitations were imposed and comprehensive instrumentation and monitoring programs were designed to ensure compliance with action and trigger levels to protect third parties for noise, vibration, subsurface movements and protection of overlying utilities and structures.

The Second Avenue Subway project has been broken into four construction phases, which could potentially overlap, to make funding of this mega project more manageable. The project is being funded by a combination of State and Federal contributions. The budget for phase 1 is $4.5 billion, in year of expenditure dollars, and it is scheduled for completion by the end of 2016. Phase 1 includes 3.9 km of twin TBM rock tunnels, double-track 21.0 m span mined rock cavern stations at 72nd Street, and 86th Street, and a double-track cut-and-cover station at 96th Street. The overall configuration of the stations aimed to achieve as shallow a cavern as feasible to minimize passenger access time between entrances and platforms and to avoid interaction with existing subway underground structures. This had to be balanced against the need to provide an adequate rock cover for the caverns. Phase 1 of the Second Avenue Subway provides early revenue service, with ridership expected to be over 200,000 weekday riders when operational. The final engineering of the project was undertaken for the New York City Transit Authority by an AECOM led joint venture and the construction of the 86th St. Station is currently underway by a joint venture of Skanska, and Traylor Bros. The 3D model of 86th Street Station is shown in Figure 1.

GEOTECHNICAL CONTEXT

The project area mainly consists of the Manhattan schist rocks, calcareous rocks of the Inwood marble and Fordham gneiss. Manhattan schists are typically crystalline variations of essentially quartz and mica composition with quartz and feldspar rich zones, garnetiferous biotite and muscovite mica schist, quartz-hornblende-mica-garnet schists, and chlorite schists. Numerous pre and post to late thrust kinematic pegmatite intrusions of varying size have been emplaced within these schists typically along and occasionally across the foliation and along other fractures. The rocks of Manhattan area have undergone multiple deformation events and are characterized by three principal joint sets with sub-sets and the dominant joint set is parallel to the foliation.

At a very early stage in the design process the significance of the fundamental geological structures were recognized as being a key to understanding the rock mass behavior. The exploration program included geological studies from microscopic to regional in addition to conventional geotechnical methods to advance this understanding. The geological investigation of the site started with collection and assimilation of existing information. This database included more than 600 historic borings which were sufficient to develop a preliminary conceptual geological model for the alignment.

An exploration program was designed to obtain data to current standards, to check, correlate and enhance existing boring data, to transform the conceptual geological model to a definitive model, and to compare this with the published geological model. Historic maps showing geomorphology, geology, land use and progressive development from early colonization of Manhattan were used to make initial interpretations of structural geology, particularly the location of major fault trends because these were postulated to be a significant influence on the natural drainage pattern of Manhattan Island before its development. This preparatory work was used to plan the exploration program with clearer focus on geological zones of importance and their relevance to design of the project.

During preliminary engineering over 350 new borings were taken along the Second Avenue Subway corridor to determine and/or verify ground conditions. In addition, over 200 environmental borings were taken in the soil overburden at locations where present or prior activities may have resulted in hazardous or industrial soil contamination. The investigation included not only the basic soil sampling and rock coring for laboratory testing and classification, but also oriented core drilling, cone penetration tests, geophysical surveys of boreholes, installation of monitoring wells and vibrating wire piezometers, observation wells, packer testing in bedrock, cross-hole seismic testing, seismic refraction testing, and in-situ stress testing. Total number of borings along the Phase 1 alignment during preliminary and final engineering was more than 180.

The primary source of geological information was rock core, although nearby excavations for deep basements and tunnels were mapped as part of the program. The rock cores were logged in general accordance with ASTM and ISRM. The approach that was developed and operated for the Second Avenue Subway project had three basic components: fabric and petrographic logging of rock core in a field laboratory, fracture and joint set orientation and classification by core logging, core scribing and borehole geophysics, and petrographic analysis by thin-section.

A solid model was needed for the orientation of discontinuities, their properties, discrete features that may have a local influence on behavior and the potential risks from major failure. To achieve this goal, subsurface exploration and testing program included orientation and frequency of fractures, shear strength properties of fractures, abrasivity of rock, faults and shear zones, intrusions and alteration, rock material properties, rock mass properties, and soil-rock interface profile and condition.

The variable quality of the rock mass along the cavern alignment required the development of multiple models representing the zonal differentiation of the rock mass in terms of foliation, jointing, and the presence of joint swarms and fractured zones. The methodology adopted, which can be described as “deterministic” was based on: the geometrically exact projection of the main rock mass features (e.g., shear zones, etc.) found in adjacent boreholes onto the section of analysis, and the inclusion of the sets of joints onto the plane of analysis on the basis of statistically derived spacing and dip angles as determined from adjacent boreholes to the section of analyses (Figure 2).

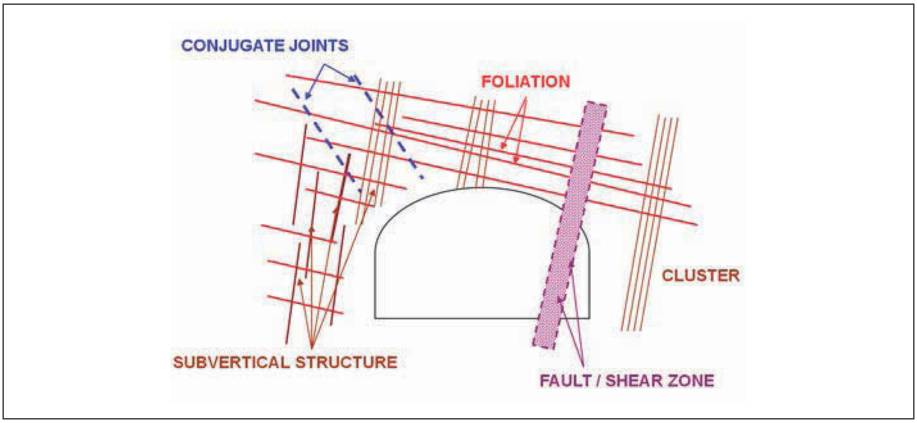

Figure 2. Types of rock joint discontinuities

The low bound spacing values were selected in all cases, whereas all joints in the sets projected were inferred to be through-cutting.

DESIGN APPROACH

Large excavation spans, low rock cover, variable geotechnical conditions, relatively large and complex intersections, and dense urban environment characterize the design challenges of the Second Avenue Subway caverns (Figure 3). The mined cavern excavation sequence and support system were designed to ensure the stability of the rock mass and adjacent structures. Therefore, maximum allowable vertical ground movement in crown was limited to 50 mm and maximum allowable differential settlement for historical buildings near cavern to less than 1/1000.

Large cavern sections require multiple drill and blast drifts. The design of drift sizes and shapes was governed by excavation rate, different drifts and cavern stability, and ground settlement and vibration concerns. Various possible cavern excavation sequence including center out drift, side in drift and their combination was considered and their pros and cons were studied through numerical modeling. The analyses show that given the nature of the rock mass (generally competent) and the tendency for gravity induced rock mass stability mechanisms, a center out sequence of excavation may be potentially more beneficial than an equivalent side in approach (Figure 3).

The center out sequence will allow continuous dissipation of the induced stresses away from the excavation profile and will facilitate the gradual formation of a rock arch over the crown. The side in sequence can be considered to initiate an increasing concentration of stresses in the central pillar, which will add to the gravity loadings released upon pillar removal during the final development of the top heading. The critical top heading excavation drifts need to be separated longitudinally to allow optimum stress redistribution to occur as well as to facilitate parallel excavation and stabilization activities in the different headings. A minimum distance of one cavern span would be appropriate in our case.

In addition to excavation sequence and support system impact on the cavern stability, the size of various drifts (cross section and round length) was adjusted in order to limit the amount of charge per delay for each blasting cycle to satisfy the strict vibration limit of 12.5 mm/sec peak particle velocity under the historical buildings. The Phase 1 construction schedule requires that the TBM tunnels be excavated prior to drill and blasting of station caverns, which imposes some restriction on the excavation sequence configuration and mucking process.

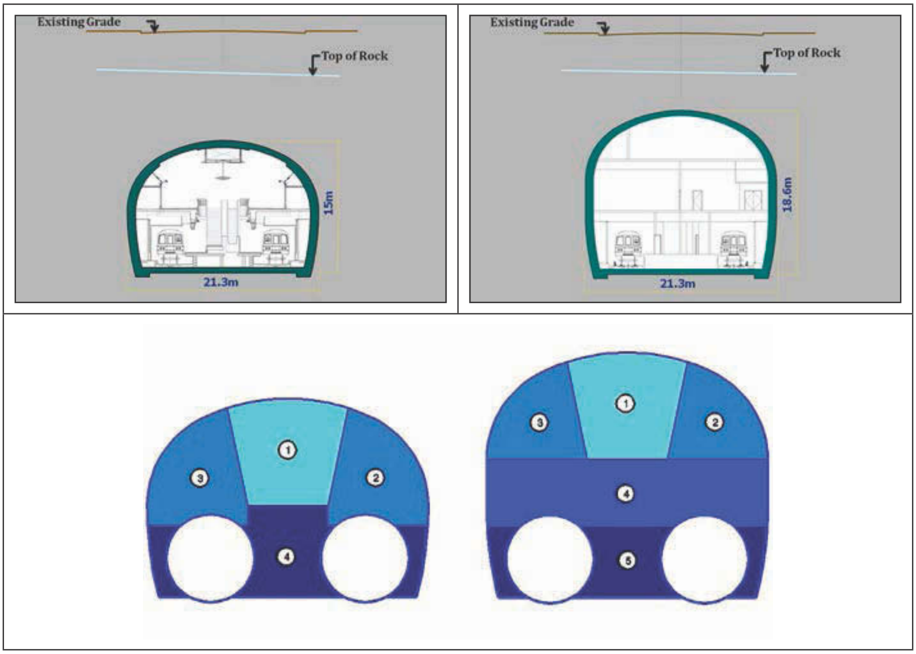

Figure 3. Double track public and ancillary caverns at 86th St. Station

Empirical data shows that there is a breakdown of the natural arching concept below some minimum cavern rock cover to span ratio. Underground rock engineering practice sets a limiting cover to span ratio of ≥⅓. To avoid heavy support requirements and allow conventional construction methods in hard rock, the cover to span ratio over all of the cavern length was kept above ⅓.

Design Based on Q Empirical Method

The cavern design features represented by large, shallow openings, jointed rock masses, random shear zones, and variable rock covers required a robust design procedure including a combination of empirical methods, continuum and discontinuum analyses. Barton's Rock Tunneling Quality Index empirical method, Q, was employed to ensure that the designed support system was compatible with successful existing and similar rock caverns.

The raw Q values were developed for each core run from more than 50 deep borings encompassing a zone that extended at least ¼ cavern span above and below the crown. From these raw Q values, the weighted average over the crown zone was taken to obtain representative Q values. Using these representative values, along with the northing and easting coordinates for each of the borings, an input file was generated to plot Q contours across the cavern plan and the centerline Q values were obtained by cutting a longitudinal section across the contours.

Discontinuum Analysis

The existence of low rock cover within a jointed rock mass led the designers to consider a block interaction problem rather than a stress strength one. Discontinuum analysis was used to ensure that the presence of joints and faults in the rock mass around the cavern does not result in unacceptable bolt loads or displacements in the cavern structure. The Universal Distinct Element Code, UDEC, was employed to perform the discontinuum analysis and calculate the ground response, and rock bolt and shotcrete forces.

The first step in the design process was to divide the cavern into different ground class zones. For each ground class zone two deterministic jointing patterns (expected worst condition and expected typical condition) and a support class obtained from empirical methods were assigned and the available data for intact rock and rock joints properties were interpreted and best estimate and lower bound values were determined. UDEC was used to evaluate the global stability of each excavation drift and the entire cavern after each drift excavation and before and after its support installation. The analysis aimed at optimization of the design in terms of excavation sequence and type and quantity of support.

For intact rock the Hoek-Brown criterion, for the foliation and cross foliation joints the Barton-Bandis joint behavior model, and for the shear zones the Mohr-Coulomb shear failure criterion were used. The convergence-confinement analysis method was applied to account for the three dimensional effects of the excavation face and a relaxation coefficient of 50% was applied after the excavation of each drift and prior to the installation of the initial liner. Three different shotcrete strengths (1, 7, 28 days) were used according to the timing of different excavation stages.

The main modeling steps consisted of: development of a rock mass model representing physical and mechanical characteristics of the ground, which was the principal factor controlling the structural behavior, initialization of the primary stress state through model consolidation under rock, soil and buildings gravity loading, excavation of various drifts and entire cavern without support to assess intrinsic stability state, and installation of the primary support in line with the appropriate excavation sequencing.

The evaluation of the results included: evaluation of the principal stability mechanisms, review of the induced stress-displacement fields, assessment of supporting function of various rock reinforcement systems in terms of tunnel profile deformation control and their load capacity requirements, and overall engineering evaluation of the modeling results.

Continuum Analysis

Continuum analysis was used to ensure that the design does not result in adverse stress strength condition in the rock mass around the cavern opening. Rock mass parameters were determined using RockLab and the excavation sequence and support installation of the cavern was modeled using Phase^2.

The structure of the rock mass was expected to be blocky with fair to good joint surface conditions. The expected typical condition assuming good joint surface condition resulted in a GSI value of 60 while the expected worst condition assuming fair joint surface condition resulted in a GSI value of 50. Two 2D continuum models with 1.8 m × 1.8 m bolt spacing for the expected typical condition and 1.5 m × 1.5 m bolt spacing for the expected worst condition and their corresponding sets of rock mass properties were developed. The caverns were excavated in 3 top heading drifts and one or two benches and the corresponding support systems were installed after each excavation stage. A relaxation coefficient of 50% was applied.

Based on the analyses performed, it can be concluded that this type of continuum analysis for jointed hard rock cases results in very small deformations and a very low level of stress in bolts and shotcrete. Therefore, continuum analysis in this kind of jointed rock cases fails to detect the local and global failure mechanisms generated by the joint sets and cannot be reliably used in the design or verification of the design of excavation sequence and support systems.

CONSTRUCTION CONSIDERATIONS

There are many factors that must be considered for excavating major quantities of rock in a highly urbanized setting such as New York City. While the technical considerations are critical, including actual excavation methods, equipment types, the means of handling over 100,000 m^3 of mucked rock, there are also important supporting concerns, including electric power, surface equipment, and managing the relationship with what is largely a residential community. The following outlines some of the measures taken to not only efficiently and safely execute the contract, but to mitigate the effect of the construction upon the surrounding community.

Major rock excavation and concrete construction underground necessitates the development of a large array of logistics support on the site surface. The site provided for 86th is, due to the dense urban environment, almost exclusively in the actual city streets, with most of the worksite situated on Second Avenue itself. Such support includes shaft cranes, muck handling systems, ventilation, equipment shops, compressors, craft labor tool boxes and offices, pumps, water treatment, electrical distribution, equipment staging areas, and material staging/storage areas. Several months of planning were required to agree upon the site configurations, with utility distribution (electrical, water discharge, water, compressed air, and communications) and muck handling figuring highly in the design phase.

Similar to other contracts on the Second Avenue Subway program, the most efficient means of excavating rock for the 86th St. Station cavern was determined to be by drilling and blasting (Figure 4), for a variety of reasons including logistics, geometry, and hard and abrasive nature of rock. This decision was clearly anticipated in the contract documents, as well as in the proposals of the bidding teams.

Blasting operations in Manhattan require a high degree of control and coordination as they are strictly controlled in New York City, due to obvious reasons such as the population density as well as concerns of terrorism and the possibility of accidents. With respect to explosives, strict limits are placed on the types, quantities permitted onsite, and means of delivery and storage of explosives, along with who is permitted to not only handle and load explosives, but even who is permitted to be in the vicinity of the explosives. Blasting times are restricted to day and evening hours, due in large part to community quality of life concerns. The charge per delay is limited to minimize vibrations which could negatively affect nearby historical structures.

The use of advanced equipment is essential to maintain the schedule, and these include computer controlled drilling jumbos, shotcrete placement and pumping equipment, material handling equipment including loaders and excavators, and many other items for support of the operations.

One of the major constraints of mining on the 86th St. Station is the removal of muck from the site. The advance of excavation is directly tied to the ability to remove muck from the ground. The first challenge in handling the muck after blasting is moving it to the shafts. This is usually difficult in the beginning as there is only a minimal amount of room available to maneuver. The second challenge involves vertically hoisting muck and storing it on street level which is in itself difficult due to the confined worksites provided. Typically, on large excavation projects, the muck is hoisted with a large crane and dumped into muck bin or pile on the surface, where it is then transferred by handling equipment into haulers which convey the material offsite. However, such an operation creates levels of noise and dust which would exceed the strict limits placed upon the jobsite. Furthermore, the size of the worksite and the fact that it would be placed in the street over utilities made such an approach infeasible.

It was decided to construct a mechanized muck handling system which was customized to fit in the site provided. The system is comprised of the main hoist, or gantry crane, which runs on rails that are elevated above grade to minimize the possibility of striking/pinch point hazards to the workforce, and also permits a more free flow of

Figure 4. Drill-and-blast excavation of cavern top heading

equipment below the system. The gantry crane lifts muck boxes which are loaded in the bottom of the shaft, up to the surface, and then lands the box onto the second component of the system, which is called the carousel deck, which is elevated relative to the grade. The carousel deck acts as a temporary muck storage area capable of storing up to about half a shift worth of muck, and is able to move the muck boxes into a muck transfer house. The third component of the system is the dumping device, which takes the boxes to a position where they can be dumped into hauling trailers which pull in below the structure. It should be noted that the first component of the system, the gantry crane, not only lifts full muck boxes and lowers empty boxes, but also services all other hoisting needs of the mining operations.

Several measures are taken in order to address noise and emissions concerns. First, most stationary or defined path equipment is powered with electricity. Electric power is more efficiently produced than onsite diesel power, and results in less onsite pollution. Furthermore, electric equipment is generally much quieter than similar diesel equipment. Additional measures taken to minimize noise and dust include the installation of modern air scrubber systems, which reduce particulate and nuisance gas emissions, and the erection of sound damping panels around most of the jobsite perimeter.

Quality control is a crucial part of the excavation process, and the project accomplished this by use of a skilled group of underground survey teams, along with the latest technology in survey equipment including robotic total stations and laser scanning systems. Equally important are the office staff, computers, and software necessary to process this information and distribute it back to the field.

The labor force that, by agreement, works in underground operations is predominately the local 147 tunnel workers union, or the 'sandhogs.' They have extensive experience in working the rock of Manhattan, including rock excavation and temporary support installation, as well as concrete construction. Their experience comes from having been active for over 100 years in New York City, and having a strong mentoring and internal training program where senior members pass on valuable experience to younger members, who incorporate this knowledge with current technology and information.

This contract includes the construction of the concrete liner within the cavern and in the associated adits and the adjacent open cut entrance structures. Underground concrete construction represents unique challenges relative to general surface construction, due to the logistical challenge of fitting equipment and components into the tunnel environment, and the lack of large overhead cranes to service the work.

The construction of the cavern is multi-layered, and includes the installation of a ground-water drainage/pressure relief system consisting of gravel and piping, a fully enveloped PVC waterproofing system which is designed to prevent any inflow of water into the system, reinforcement installations with provisions to protect against stray current, and concreting of various arch shapes. The complexity of these systems necessitate the development of many shop drawings for piping, concrete placements, formwork, and waterproofing systems.

A major consideration in such construction is the procurement and fabrication of major tunnel formwork systems. Such systems are custom built, usually out of steel, and must be designed in such a way to be able to be lowered into the tunnel, conveyed into position, and erected efficiently in the cavern. Design considerations also include set up for placement, the actual placement of concrete through pour doors, an integral concrete distribution system with a placer car or manifold that can move amongst different placement ports, provisions to vibrate the form, and an efficient means of demolding the formwork after the initial set of the concrete. These forms require up to six months for fabrication, and as such require a significant amount of advance planning and design. Other considerations include concrete conveyance, and mix design.

CONCLUSION

The Second Avenue Subway Project is one of the largest and most complex construction projects in the United States and a critical part of the success for the project was the safe and optimum design of its large and shallow rock caverns. Empirical method and two and three dimensional continuum and discontinuum analyses were used in designing the excavation sequence and initial support system. Understanding each method's limitations in comparison of their results provided a comfortable margin of safety compensating for the unknowns in the design process.

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DESIGN AND CONSTRUCTION OF A MASSIVE TUNNEL JUNCTION FOR HONG KONG'S EXPRESS RAIL LINK PROJECT

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Derek Kwok ▪ MTR Corporation

ABSTRACT

The 26km (16.2 mile) long Express Rail Link (XRL) Hong Kong section is a cross boundary transport infrastructure project, which will form part of the high speed rail services between Hong Kong and Guangzhou, China. As part of Contract 821, the Dragages-Bouygues JV constructed one of the largest tunnel intersections ever attempted in Hong Kong, a 24m (78.7 ft.) span adit forming a junction with the 22m (72 ft.) span, resulting in an effective span of 32m (105 ft.).

The design for temporary support is presented including development of the geological model and implementing this into 2D and 3D numerical models. The construction sequence was fundamental in controlling stress re-distribution and interaction between tunnels. Instrumentation and mapping were used to verify assumptions during construction. Convergence data is compared to design estimates. Lastly, recommendations for successful design and construction of similar tunnel intersections in rock are made.

INTRODUCTION

As one of the most densely populated cities on the planet, Hong Kong is in the midst of an underground construction boom as the transportation network expands to meet demand. One of the major projects announced by the Chief Executive in the 2007–2008 Policy Address was construction of the Hong Kong section of the Guangzhou-Shenzhen Hong Kong Express Rail Link (XRL). Operating between a terminus station at Kowloon and the boundary to the north with mainland China, the 26km (16.2 mile) dedicated high speed underground rail corridor is currently being constructed by a combination of drill and blast, TBM, and cut and cover methods.

The Joint Venture of Dragages-Bouygues (DBJV), supported by Arup, was awarded two of the eight main tunneling contracts by the MTR Corporation in May and July 2010. The focus of this paper is on the drill and blast excavated Contract 821.

Contract 821 comprises 3.64km (2.26 mile) of twin track, single tube tunnel excavated within hard rock conditions. Adjoining Contract 822 to the north and the TBM driven Contract 820 to the south, 821 is situated under the Kwai Chung area on the Kowloon side of Hong Kong. A single, permanent access and ventilation adit was provided in Kwai Chung, grading down 660m (2,165 ft.) on an 8% slope to intersect with the running tunnel at a depth of 270m (886 ft.) below ground surface.

Geometry

At the initial design stage, it was realized that this single access point, and its intersection with the running tunnel, would be the bottleneck in terms of construction planning.

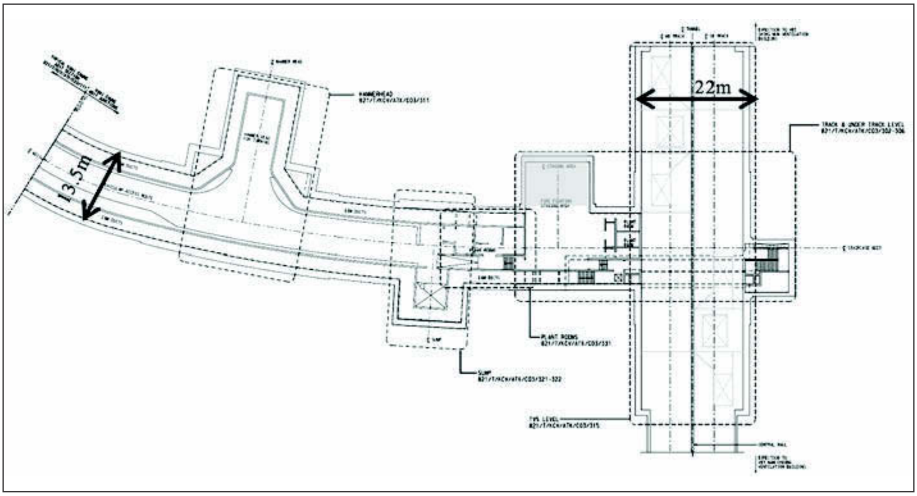

Figure 1. Plan view of conforming Kwai Chung junction area, with 22m (72 ft.) span enlarged running tunnel shown at right

Not only was the adit to provide permanent ventilation to the running tunnel (which was enlarged for overhead plenum ducts at this location), but it was also to provide a staging and turnaround area for emergency services in case of evacuation, and an adit tunnel sump. The multiple functions of the conforming adit geometry, shown in Figure 1, resulted in the creation of local enlargement and several stub tunnels, and narrow pillars, which was expected to delay access to the main running tunnels until completed.

The conforming design was hence reviewed after the contract award to DBJV. In order to gain access to the running tunnel as early as possible, it was suggested to take the junction off critical path by excavating two temporary bypass tunnels around the area. At the same time, the geometry of the junction itself was proposed to be rearranged, so as to avoid formation of the narrow pillar between the running tunnel and staging area (stub tunnel, shaded in Figure 1) by "enveloping" the stub tunnel through a flaring out of the adit. It was also proposed to put the sump "in line" with the adit by overexciting the adit invert to produce the required volume. The proposed geometry, shown in Figure 2, would secure the secure the construction program.

GEOLOGICAL CONDITIONS

Overview

The geological conditions were well interpreted along the alignment and the ground model documented in the Contract Documents. The junction site was known to be located within the Needle Hill Formation, a porphyritic fine-grained monzogranite, containing numerous fine-grained rhyolitic dykes and quartz veins. The Geotechnical Engineering Report (GER) described the rock as being, pink spotted dark greenish grey and white, mottled dark grey, strong to moderately strong, slightly to moderately weathered, and silicified. No major faults were interpreted from air photos or observed in borings.

Existing Investigation

A vertical borehole had been drilled during preliminary engineering which intersected the junction area. Both in situ and lab tests were previously carried out on samples obtained from this hole and documented in the GER.

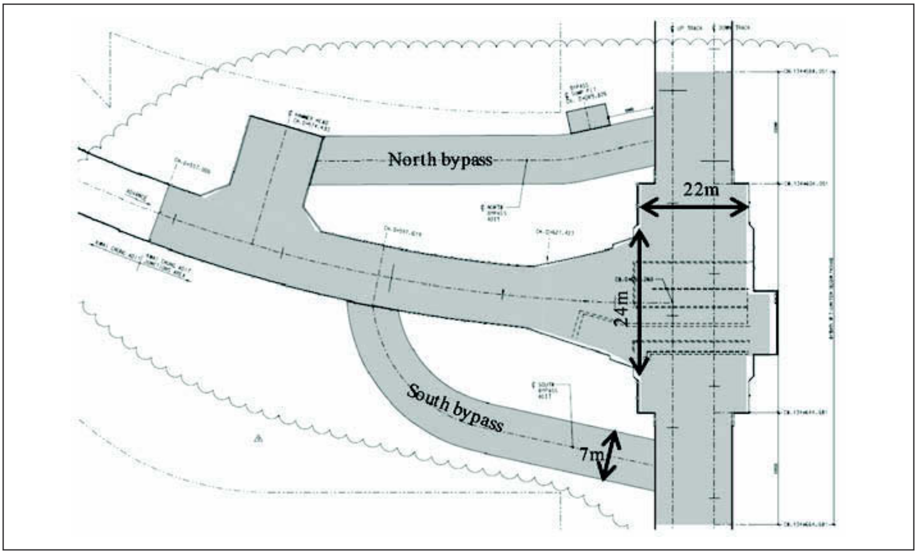

Figure 2. Plan view of proposed Kwai Chung junction area

Table 1. Average results of hydrofracture tests at junction location

Depth (mbgl)	σ_H (MPa)	σ_h (MPa)	σ_v (MPa)	$K = \sigma_H/\sigma_v$	Azimuth of σ_H
281	11.7	7.6	7.6	1.5	021

In Situ Testing

An acoustic televiewer was dropped down the borehole to map the dip and dip direction of intersecting discontinuities. In addition, hydrofracture testing was carried out at the junction depth to establish the magnitude and orientation of in situ stresses. The average results of the hydrofracture tests are given in Table 1.

Low standard deviations for all the measured principal stresses gave a high confidence in the results. The orientation of the maximum principal stress was found to trend obliquely across the enlarged running tunnel. Therefore, only a certain component of this stress could be considered to act perpendicular to the tunnel cross sections in the 2D designs which would be undertaken. Laboratory testing established the typical uniaxial compressive strength of the rock at 80MPa (11.6 ksi). The results obtained from the above in situ tests would later prove fundamental to the successful design and construction of the junction.

Rock Mass Classification

A review of the core and discontinuity surface conditions led to the conclusion that the rock mass conditions at the junction location would be uniform and consistent, with 3 to 4 sub vertical joint sets having planar rough, slightly chloritized joint surfaces. There was no evidence of infilling, and the lack of thick mineralization and heavy staining on joint surfaces suggested that the aperture was very tight and circulation of water minimal. It was found that the typical 'Q' rating could be expected to be between 4–7, with an average value of 6. The active stress parameter (Jw/SRF) was assumed equal to 1.

TEMPORARY SUPPORT DESIGN

The proposed junction design would need to be proven numerically owing to the complex stress distribution/superposition and lack of precedent. However, basic empirical and analytical design methods were used to establish preliminary temporary support classes for the typical adit profile (13.5m span) as well as the bypass tunnels. These methods consist of the Q system, as well as reinforced rock arch theory after Bischoff and Smart (1977) and kinematic block analysis using the program UNWEDGE.

For the main junction area, it was decided to carry out 2D numerical models in order to establish the controlling failure mechanisms and assess the sensitivity to various construction sequences that were being proposed at the time. Subsequently, 3D modeling and pillar stability analyses were conducted to validate the 2D models.

2D Numerical Modeling

Two design sections were selected for modelled: one across the 24m (78.7 ft.) span shown in Figure 2 (including the two 7m × 7m [23 ft. × 23 ft.] horseshoe shaped bypass tunnels) and another across the enlarged 20m span running tunnel. The aim of the 2D modelling was to develop a temporary support type for each section and carry out construction sequence sensitivity analyses and their impact on support capacity.

The first stage in the numerical modelling process was to decide whether the rock mass behaviour was more suited towards a continuum "mass" approach or a discontinuum "block" approach where discontinuities are discretely modelled. The semi-empirical methods of Palmstrom (2005) and Palmstrom and Stille (2010) have been found to be extremely useful in delineating boundaries between different rock mass behaviour types based on block size estimation.

Based on the methodology described above, a discontinuum approach using the distinct element program UDEC was adopted for the 2D modeling of the Kwai Chung junction area. Therefore, a detailed understanding of joint set orientations and shear strength properties was fundamental to calibrating the model for reliable results.

The 2D modeling also required estimation of a ground relaxation factor, which was assumed as 50% based on experience in hard rock tunneling. The models simulated the hardening effect of shotcrete as excavation of partial drifts progressed and verified support capacity at each interim stage. After several runs for each design section, the following support and sequence was proposed:

- Rock dowels: 6m (19.7 ft.) long, 32mm (1¼ in.) diameter fully grouted on a 1.25m (5 ft.) grid;
- Shotcrete: 100mm (4 in.) thick, fiber reinforced;
- Excavation sequence: (3) top heading drifts for the adit and (2) for the enlarged running tunnel, maximum span and height of 13.5m (44.3 ft.) × 7m (23 ft.), respectively, followed by split bench.

One conclusion which was particularly apparent from the 2D UDEC modeling was that the performance (and acceptance) of the proposed temporary support system was closely dependent on the amount of relaxation that was permitted prior to installation. This was a direct consequence of the junction depth and higher in situ stresses. Unsupported, or "intrinsic," runs were carried out to evaluate the expected failure modes, which were identified as excessive joint shear displacement leading to loss of self-arching ability (through reduction in shear strength to residual friction values). Therefore, a delicate balance between the amount of pre-deformation, or relaxation allowed, and mobilization of joint shear strength had to be investigated and understood. Two options were developed which would allow for the required elastic stress relaxation prior to main rock dowel installation while limiting the amount of joint shearing. The first option involved installation of two different rock reinforcement types—namely a

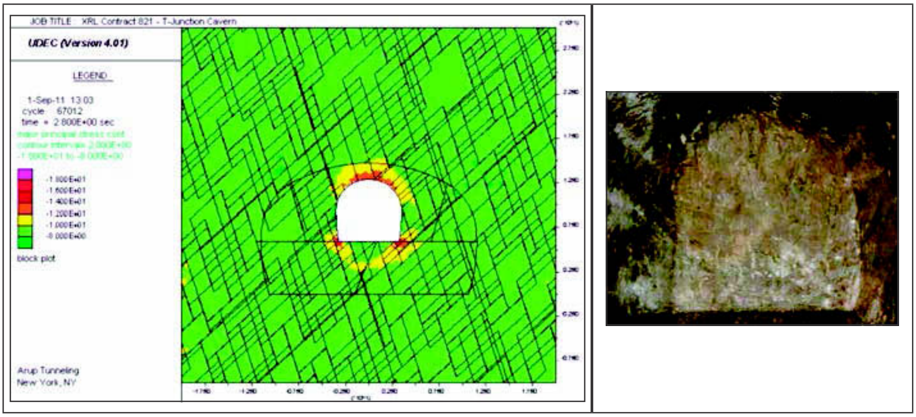

Figure 3. UDEC model showing contours of principal stress around the proposed pilot tunnel excavation (left) and actual pilot excavation face (right)

primary bolt to support the initial drift(s), followed by longer secondary dowels installed in between at larger spacing. The same methodology was adopted for the 61m (200 ft.) span Gjovik ice hockey cavern in Norway, which used 6m (19.7 ft.) long grouted primary dowels and 12m (39.4 ft.) long secondary cable bolts. For the Kwai Chung junction area, 3m (10 ft). long Swellex MN24 were proposed as the primary bolt, followed some distance back by the 6m long, 32mm grouted dowels. It was anticipated that the yielding characteristic of the Swellex bolt would allow for some rock mass movements without compromising safety as significant bolt strains could be accommodated without loss of load carrying capacity. Once installed, the stiffer dowels would limit further shear and radial displacements that had been allowed with the Swellex. The second, and ultimately adopted methodology, involved the drilling and blasting of a small 7m (23 ft.) × 7m (23 ft.) pilot tunnel from the adit into and through the junction area. The pilot tunnel also had the advantage that it could be used to verify the ground conditions and confirm the assumed jointing characteristics. Output from the UDEC model (with inclusion of the pilot tunnel) is shown in Figure 3 for the 24m (78.7 ft.) adit span. From an empirical support assessment, the pilot tunnel was self-supporting and only required localized measures for safety. This aided in allowing for controlled convergence to occur and providing the necessary relaxation.

3D Numerical Modeling

Once the temporary support and construction sequence had been agreed upon between all parties, a final 3D finite element model was developed using the software package Midas GTS. The choice of continuum over discontinuum in 3D was a function of the time and cost required for such a model. As a 3D FE model could be carried out in house and in a relatively short amount of time, it was deemed appropriate for the purpose of justifying the 2D UDEC models and assessing any unforeseen short term stress interaction issues that may have been overlooked. The full geometry was input into the 3D model as shown in Figure 2, along with a 17 stage excavation sequence which represented the full junction development plan as well as the aging process of shotcrete. The mesh used in the 3D Midas model is shown in Figure 4.

The primary purpose of the 3D model was to verify the 2D UDEC models through similar deformations and support forces. It was also expected that the preliminary support design would need to be fine- tuned around areas of stress superposition or concentration that were not considered in the 2D analyses. In general, the results were

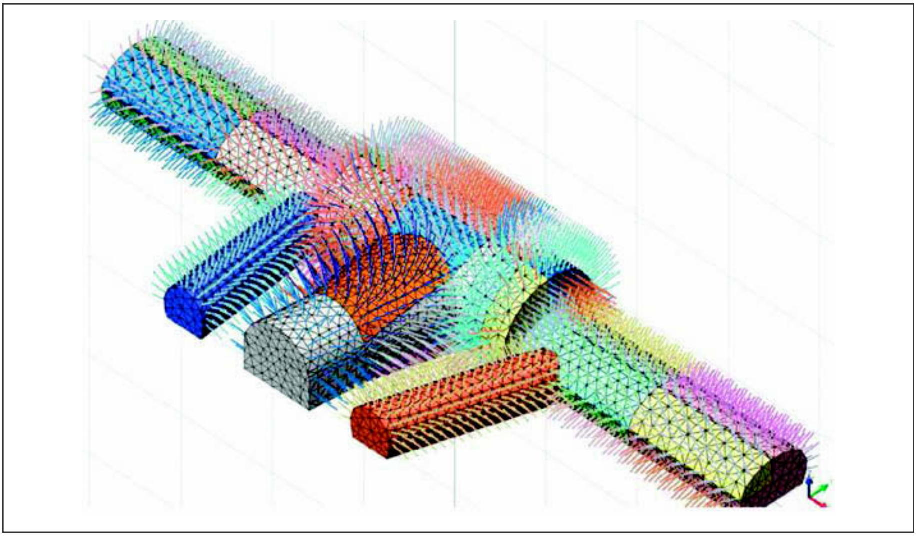

Figure 4. Midas GTS 3D finite element model developed for the final verification of the junction temporary support and excavation design

Table 2. Comparison of results between 2D and 3D models for the Kwai Chung 24m (78.7 ft.) span junction

Model	Max. Crown Displacement (mm)	Ave. Rock Dowel Axial Force (kN)	Max. Shotcrete Axial Force (kN/m)	Max. Shotcrete Bending Moment (kN-m/m)	Tangential Stress at Crown, σ_1 (MPa)
UDEC (2D)	19	98	951	43	10
Midas GTS (3D)	19	80	876 (for 85% of elements) Max. = 3 MN	6.6 (for 98% of elements) Max. = 36	11

very similar to those obtained in UDEC, which gave confidence in the design. A comparison between the two models is presented in Table 2 for several key parameters of the 24m (78.7 ft.) span section.

The major difference between the 2D and 3D models was found in the shotcrete lining bending moment. This is not surprising as UDEC models the lining using 2D beam elements, while Midas utilizes a 3D plate element, which is much more efficient in distributing moments compared to a beam. Similar vertical stress distributions were observed in both models.

Few changes were made to the support and construction sequence after reviewing the results of the 3D model. Shotcrete thickness was locally increased to 200mm (8 in.) in some areas to address high stress concentrations; however rock reinforcement details remained unchanged. It is believed that two processes specifically resulted in the very close agreement between the models. The first was the availability of an accurate in situ stress measurement. This was fundamental for the 3D continuum model as all three stress tensors are key inputs which generally have a large influence over the results, in addition to the deformation modulus. The other aspect was in the process of taking the geological model and developing a compatible numerical UDEC model. Use of the block size analysis and distribution method has been shown to help improve the results and reliability of the model.

Pillar Stability

Pillar stability was carried out at critical locations using the results of the numerical models. Overstress was checked by comparing the average pillar maximum and minimum principal stresses to a Hoek-Brown failure criterion. The UDEC output for pillar stress shown in Figure 5 closely resembles the expected convex (minimum or confining stress) and concave (maximum stress) distributions which would be expected from theoretical calculations. It is also possible to discern zones of elastic and plastic behaviour from the bypass tunnel alone (middle curve) prior to excavation of the cavern. The empirical dowel length was checked and ensured to extend beyond this zone of plasticity, which extend for approximately 2m (6.6 ft.) around the 7m (23 ft.) wide bypass. Figure 6 shows the bypass tunnel junction with the Kwai Chung adit and the pillar of interest.

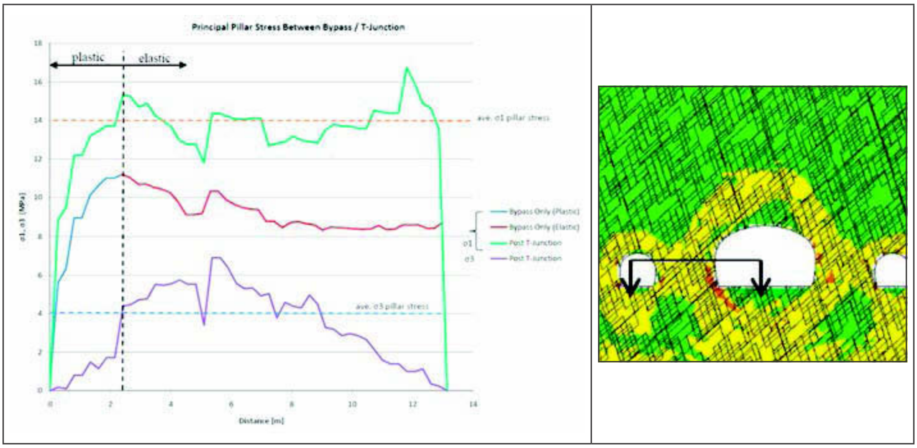

Figure 5. Pillar stresses between 24m (78.7 ft.) span adit tunnel and 7m (23 ft.) span bypass tunnel as extracted from UDEC

Figure 6. Intersection of south bypass (right) and Kwai Chung adit. The drill jumbo is faintly visible at left working in the junction area.

CONSTRUCTION

Overview

Excavation of the Kwai Chung junction area was successfully completed in April 2012. The final design saw macrosynthetic polypropylene fibers substituted for steel fibers in the shotcrete at a dosage rate of 7kg/m^3.

Weathered Zone and Convergence

During development of the top heading in the enlarged running tunnel, a sub vertical weathered zone was found to cut across the junction crown at an oblique angle to the running tunnel axis, a very unfavourable orientation for junction stability. The weathered zone belonged to a persistent joint set which had been identified and mapped early on in the adit excavation, although this particular zone was found to be more weathered than usual, receiving a Ja = 3 rating on the Q scale. With Q values being mapped below the design applicability level of Q=1, it was felt that a reduction in joint friction angle to 25° was justified and the 2D UDEC models re-run in order to validate the adopted design against the actual ground conditions. Once complete, a new set of design applicability parameters was issued on revised drawings, including a lower bound Q value of 0.7.

After observing the issues noted above during top heading excavation, it was considered prudent to closely monitor joint shear displacements during benching of the junction area. The problematic joint set had been captured in the original UDEC model of the enlarged running tunnel, so trigger levels for sidewall deformation and joint shear movement were set based off this.

It was proposed to install a horizontal multiple point borehole extensometer approximately 1m up from the current top heading invert in order to capture sidewall movement once the bench was removed. Although this was done, subsequent bench blasting damaged the instrument and typical convergence monitoring had to be relied upon. A snapshot of measured horizontal convergence across the 22m (72 ft.) enlarged running tunnel section is given in Figure 7.

Figure 7 indicates a maximum of 8mm (5⁄16 in.) convergence during benching of the junction area. From the 2D UDEC model, a total horizontal displacement of 23mm (1 in.) was predicted (as shown in Figure 8), with approximately 15mm (5⁄8 in.) of that being realized following top heading excavation. Therefore, the values recorded in Figure 7 are very close to the predicted magnitude that would be expected from UDEC. In fact, the maximum surveyed convergence in the junction area did not exceed 10mm (3⁄8 in.) for any of the 19 convergence arrays specified around the junction area.

Design Verification

Throughout the course of excavation, the DBJV team worked closely together to pass along instrumentation and monitoring data and interpret the results in a timely manner. The geological model was continually updated and compared with design assumptions to ensure the validity of the support. Table 3 provides a summary of the original pre-construction key design parameters compared to the observed parameters for the junction.

Conclusion and Recommendations for Junction Design in Hard Rock

The success of the Kwai Chung junction was made possible by having a detailed ground investigation program, including in situ tests, with which to base the design on, an experienced team of on-site engineering geologists who were familiar with the key rock mass parameters which controlled the design, as well as the continued involvement of the design team during construction. For junction design methods, there are

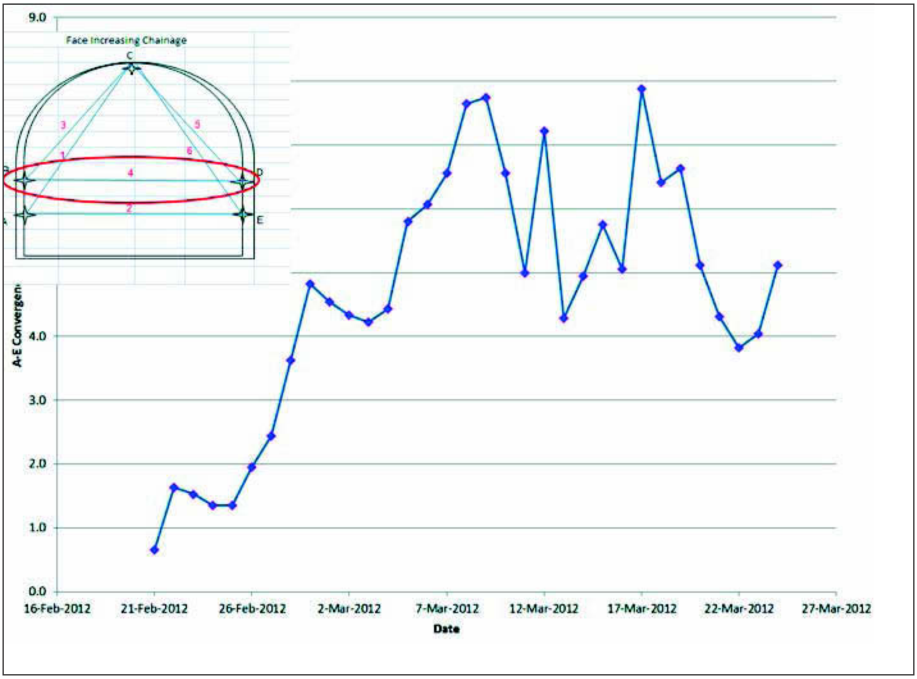

Figure 7. Horizontal convergence measured between ribs in the 22m (72 ft.) span tunnel

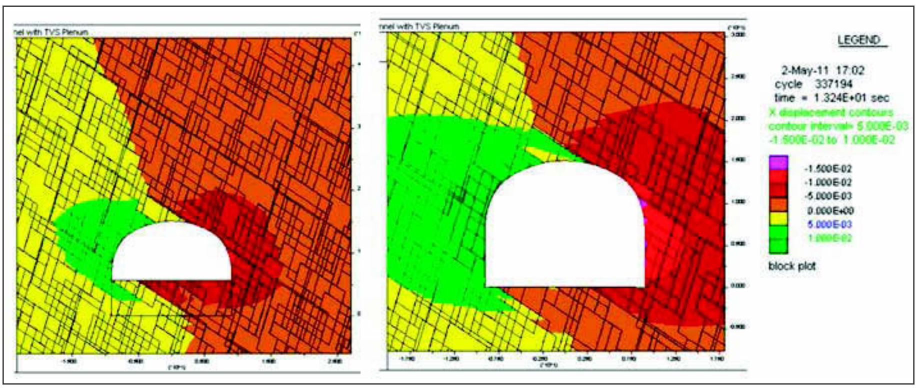

Figure 8. Predicted horizontal convergence measured between sidewalls in the 22m (72 ft.) span enlarged running tunnel from UDEC model. Values of x-displacement are contoured in units of meters.

numerous tools that can be used in the preliminary stages, such as design tables by Hsiao et al. (2009). Larger and more complex junctions warrant the use of numerical methods. It may not be necessary to use 3D modelling in every situation, and certainly 3D modelling should not be undertaken until a good understanding of ground behaviour and sensitivity analyses have been addressed in 2D models. Trigger levels (alert, alarm, action) should be set based on the results obtained from models, typically using 80% of the design convergence as the alert level (Figure 9).

Table 3. Comparison of design and observed rock mass properties for the junction

Parameter	Design	Observed
Q range (ave.)	4–10 (6)	3.3–4.2 (3.8)
Deformation modulus for 3D model, E_m	16 GPa	E_m = 25log(3.8) = 14.5 GPa E_m = $10Qc^{1/3}$ = 13.8 GPa
Major joint set orientations (dip/dip direction)	74/242 53/242 65/007	75/270 40/220 77/355
Joint set spacing (refer to above sets)	0.5–1.5m 0.4m 1.5–2.0m	0.2–0.6m 0.2–0.6m 0.2–0.6m
Joint persistence (refer to above sets)	8–12m 8–12m 8–12m	>7m >5m >3m
Joint friction angle, φ	25–30°	$\varphi \approx \tan^{-1}(Jr/Ja)$ = 27–37°

where Qc = (UCS/100) × Q

Figure 9. Completed development of junction as viewed down Kwai Chung adit (left) and the in line tunnel sump viewed from the junction (right)

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NEW YORK CITY—SECOND AVENUE SUBWAY: MTA'S 72ND STREET STATION AND TUNNELS PROJECT CONSTRUCTION OF A LARGE SPAN STATION CAVERN, RUNNING TUNNELS, CROSS-OVER AND TURN-OUT CAVERNS, SHAFTS AND ENTRANCES

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ABSTRACT

New York City's extensive transit system had not been expanded for several decades until construction of the MTA's Second Avenue Subway program was re-started in 2007. Under Phase 1, the 72nd Street Station and Tunnels Project represents a significant portion of the extensive program. This paper describes the overall project design and unique construction challenges encountered to date for the fast-track excavation and final concrete lining of the underground works. This includes a large span station cavern, cross-over and turn-out caverns, running tunnels as well as three entrances and two ancillary shafts. All work is being performed in a densely developed urban environment, with a restricted surface work site, significant operational constraints and an aggressive construction schedule.

SUMMARY

The design and construction of the 72nd Street Station and Tunnels Project represents a substantial undertaking by the MTA as part of its ambitious Second Avenue Subway construction program. It is one of three primary station and tunnel contracts that forms the core of the civil works portion for Phase 1 of the program. The integrated design of the 72nd Street project required approximately eight years to complete, in conjunction with the corresponding designs for adjacent stations and connecting tunnels in the program. Construction of the heavy civil portion of this station will require more than three years of continuous, multi-shift operations—to complete the excavation and final concrete lining of the station, cross-overs and turn-out caverns as well as running tunnels, Ancillaries and Entrances. The fast-paced construction schedule will result in the timely commencement of the follow-on finishing and systems contracts; linked to a planned Revenue Service Date in December 2016 for Phase 1.

The 72nd Street project is located between the existing 63rd Street

Figure 1. MTA's general arrangement route plan for the Second Avenue Subway in Manhattan, New York City

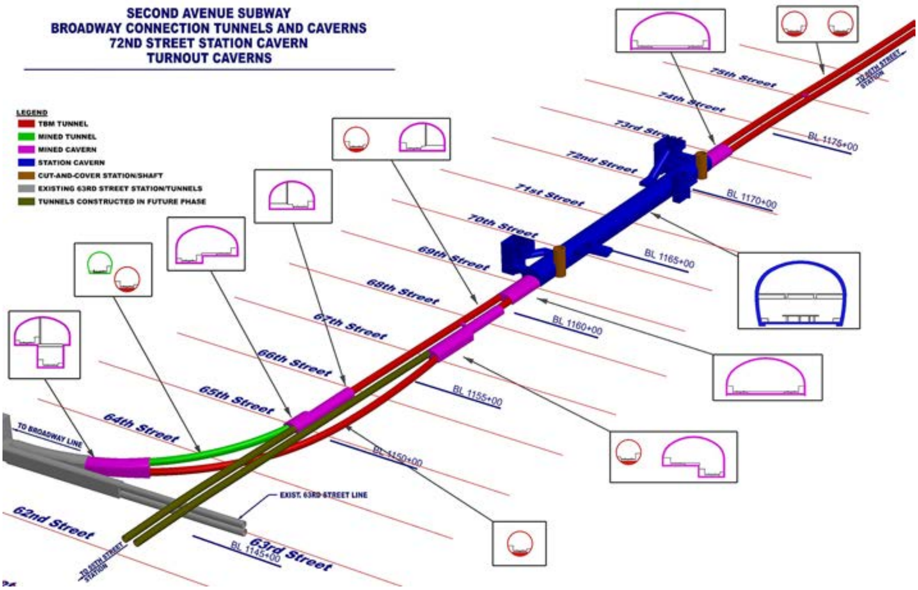

Figure 2. Overall site plan for the 72nd Street Station and Tunnels Project for the MTA. The project extends from 73rd Street in the north to the tie-in point at the existing 63rd Street Station for a distance of about 3,000 LF.

Bellmouth section and 73rd Street where it connects to running tunnels constructed under a separate contract. The project includes a 1,300 foot long station cavern with two cross-over caverns. The south tunnels and turn-out caverns cover an additional 2,000 LF south of the station. The site is located in a densely developed portion of the Upper East Side in New York City and as such, provides limited construction areas.

Work restrictions and controls confine day-to-day operations to the extent that extensive planning is constantly needed to meet the 37 month construction schedule with timely deliveries of materials and equipment, along with continuous removal of spoil materials.

At present (early 2013), the excavation work is in its final stage. Meanwhile, approximately 30% of the final lining is already in place in accordance with the construction schedule requiring multiple concurrent activities. An interim milestone date covered approximately 40% of the northern portion of the station. Refer to Figures 1 and 2 for location and layout details.

KEY DESIGN FEATURES FOR THE PROJECT

The overall design of the station, turn-out caverns and south tunnels along with the Entrance and Ancillaries had to conform the NYCTA requirements but also satisfy recent NFPA and FTA guidelines. The new stations on the Second Avenue Subway would, therefore, be substantially different than most other existing MTA stations in New York City. They would have more voluminous interiors, center platforms, have more entrances, be fully ADA compliant and focus on longer life cycle use with minimum maintenance requirements. Some of the key features of the design of the caverns and tunnels for the project include the following.

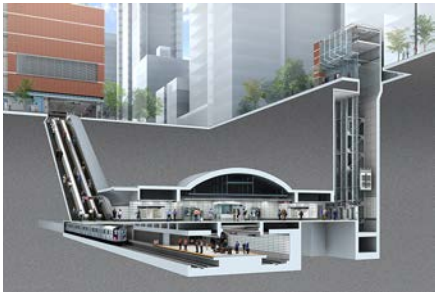

Figure 3. Section rendering of the 72nd Street station at completion. Finishing work will be done separately.

Figure 4. Rendering of the station mezzanine level at completion. Finishing work will be done separately.

- **Fully drained tunnels and caverns**
  - All tunnels and caverns are designed as fully-drained and waterproof-lined structures, complete with a final concrete lining installed.
  - Ground water pressure is relieved through an extensive system of piping and gravel filters to a passive sump and discharge facility in the station invert.
- **Initial ground support with shotcrete and rock bolts defined in the Contract**
  - The ground support design for the caverns, tunnels and adits included detailed requirements, known as *Initial Support*, to be supplemented as needed with *Additional Support* of the same nature and materials.
  - Ground support included fully-grouted and tensioned rock bolts and dowels as well as multiple layers of steel fiber reinforced shotcrete (SFRS).
  - The design lengths and patterned spacing of rock bolts and dowels varied depending on location in the tunnels and caverns. Shotcrete thickness was also variable depending on the location with the final (exposed) layer placed without steel fibers.
- **Two TBM-bored tunnels running throughout the project length**
  - The station and tunnel design included the pre-excavation by TBM of two "pilot" tunnels. This was primarily done to complete the lengthy running tunnel excavation by mechanical methods (and avoid blasting) starting at 96th Street and ending at 63rd Street. Portions of these TBM bores were enlarged into the station, cross-over and turn-out caverns.
- **Provisions made for follow-on mezzanine and platform construction**
  - The heavy civil contract did not include the construction of the mezzanine or station platform. Just the same, considerable provisions were made for this specialized construction in the exterior walls and invert in the station cavern (Figures 3 and 4).
  - Access for the construction of the station mezzanine and platform would be through both Ancillary structures as part of the follow-on finishing contract.
- **Starter construction shafts provided for early access to the station cavern**
  - The civil works contract did not include the construction of the initial 30 VF of the two temporary construction shafts—one located on 2nd Avenue

near 69th Street and other on 2nd Avenue near 72nd Street. These shafts were later developed and became the primary access to the station cavern and south tunnels to a maximum depth of 95 feet.
    - The temporary construction shafts will be backfilled and the areas restored as part of the heavy civil contract. This is consistent with completion of the underlying station cavern arch final cast-in-place lining.
- **Fan driven ventilation system with a cooling system for the station**
    - Forced ventilation and an "air tempering" system for cooling in the station cavern and entrances
    - Dedicated intake and exhaust systems for normal and emergency operations
- **Twin track station and parallel running tunnels**
    - The station cavern includes two tracks (Uptown and Downtown directions)
    - Two parallel running tunnels contain a single track to connect the 72nd Street Station to the Bellmouth area of the existing 63rd Street Station
- **Station and Cross-Over Cavern center platform and mezzanine**
    - The Station Cavern and Cross-Over caverns are self-supporting in rock
    - A center platform runs the full length of the station
    - The mezzanine (to ticketing level) is supported from the station walls
- **Entrances with escalators/stairs and a separate bank of five elevators**
    - Public access to the station cavern mezzanine level includes two entrances equipped with twin escalators and stairs. Additional escalators and stairs connect the mezzanine to the platform level.
    - Five elevators enclosed in a separate entrance connect the street level to the mezzanine. Additional elevators connect the mezzanine to the platform level.

SCOPE OF WORK FOR THE HEAVY CIVIL CONTRACT

The scope of the heavy civil contract—for the excavation and final lining of the station and turn-out caverns as well as the running tunnels, ancillaries and entrances—included the following generalized primary scopes covered under the Lump Sum contract. Refer to Figures 5 and 6.

- **Underground excavation, support and lining in rock of the following:**
    - Station cavern 980 LF
    - Cross-Over caverns 2 × 160 LF
    - G3 Turn-Out cavern 1 × 285 LF
    - G4 Turn-Out cavern 1 × 385 LF
    - 63rd Street Stub Cavern 1 × 165 LF
    - Horseshoe tunnel 1 × 410 LF
- **Surface excavation, support in soil and rock of the following:**
    - Ancillary 1 shaft 11,800 BCY
    - Ancillary 2 shaft 14,000 BCY
    - Entrance 1 and 2 inclines 4,400 BCY
    - Entrance 3 shaft 5,600 BCY

Figure 5. General arrangement of the station cavern, cross-overs, ancillaries and entrances

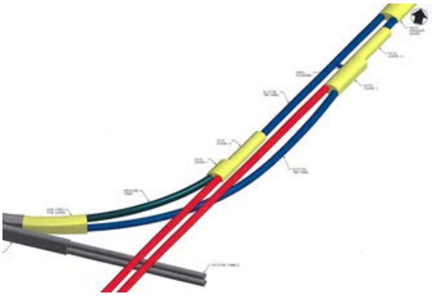

Figure 6. General arrangement of the south tunnel caverns and tunnels to the existing 63rd Street Station

The scope of the work also was performed within a 37-month construction schedule with one interim milestone date after 31 months. The work restrictions also included two separate "no blasting" periods that were planned to occur in the midst of the station and tunnel rock excavation programs. This was due to the concurrent (and conflicting) excavation of two underlying TBM bored running tunnels—located between 96th and 63rd Streets.

CONSTRUCTION SCHEDULE

The overall schedule for the civil construction contract was 37 months long beginning on 01 Oct 10. The schedule included one interim milestone date, *Milestone 1,* after 31 months and a *Substantial Completion* date after 37 months. This schedule was always considered to be very aggressive and considerably challenging in light of other site and contractual conditions.

It should be noted also that the Notice-of-Award (NOA) of the Contact also coincided on the same date with the MTA's Notice-to-Proceed (NTP) with the work and, therefore, the start of the contract time. *Milestone 1* defined the completion of approximately the northernmost 40% of the station along with the North Cross-Over, Ancillary 2 and Entrance 2.

The construction schedule was detailed in CPM using Primavera P6 software, then submitted for approval by the MTA. The CPM schedule was carefully reviewed and updated monthly. The Critical Path and float relative to *Milestone 1* (Month 31) and *Substantial Completion* (Month 37) were computed and analyzed. The MTA used this data to assess progress relative to its master Second Avenue Subway construction program. Overall, the CPM schedule included the primary construction activities and durations as listed below and shown in Figure 7.

- **Rock excavation and support** NOA + 23 months (±)
  - Station, adits, cross-overs
  - South caverns and tunnels
- **Final concrete lining (cast-in-place)** NOA + 23 to 37 months (±)
  - Station, adits, cross-overs
  - South caverns and tunnels
- **No blasting periods (per the contract)** No.1 3 months (fixed duration)
  No.2 4 months (fixed duration)

Owing to SSK's ability to coordinate its construction operations with an adjacent contractor responsible for boring two underlying TBM tunnels, the two "no blast" periods were substantially eliminated. Nonetheless, other blasting restrictions were experienced which impacted the work and extended the planned excavation period.

Rock excavation was completed generally as listed in Figure 8. Meanwhile, several concurrent operations including the initial stages of the final lining were underway in the same period (but not shown in this figure for clarity).

Concrete final lining operations are currently underway and tracking well with the CPM schedule for the planned completion of *Milestone 1* and the *Substantial Completion* date. Refer to Figure 9 that lists the primary final lining operations in the station and south tunnels. In order to complete the post-mining schedule, several concrete lining operations were underway concurrently. This posed many logistical problems that had to be balanced with competing and conflicting goals in the station and south tunnel work areas in the final 15 months of the schedule. Nonetheless, the work

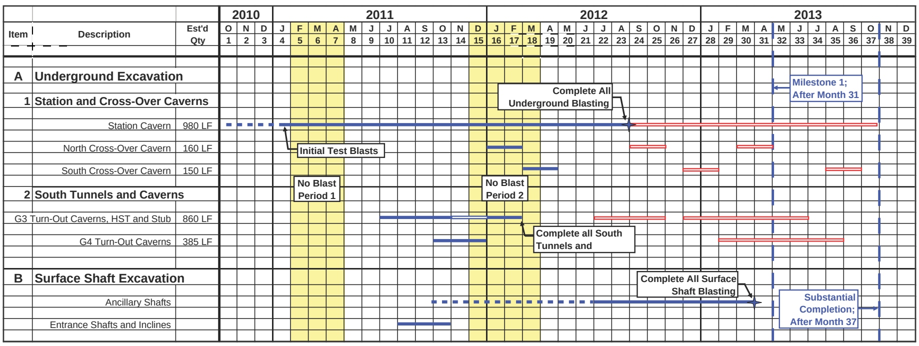

Figure 7. Overall excavation and final lining schedule for the project—including all caverns, tunnels, adits, surface shafts and entrances. Additional construction activities are detailed in the following figures.

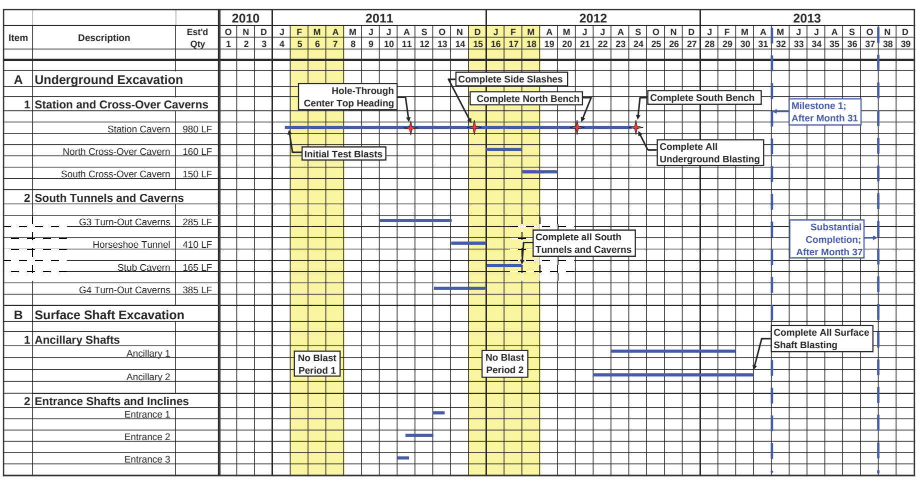

Figure 8. Overall excavation schedule for the project—including all caverns, tunnels, adits, surface shafts and entrances. The work required multiple heading operations while constantly mucking to two construction shafts.

proceeded well with planned early completion and hand-over of the south tunnels and caverns to the MTA for follow-on contracts.

SITE CONDITIONS AND WORK RESTRICTIONS

The project is located in a very densely developed area in New York City with the “linear site” confined only to designated portions of 2nd Avenue. Refer to Figures 10 and 11. No “off-site” areas were included by the MTA in the Contract for staging and storage of materials and equipment. The site is surrounded by numerous high-rise residential buildings with 2nd Avenue acting is the primary thoroughfare to and from the area. In fact, 2nd Avenue is also one of the primary access routes to mid and lower Manhattan from the Bronx (to the north) and Brooklyn (to the east). As such, all construction operations had to be carefully planned to provide for “just-in-time” deliveries (and removals) on a daily basis while complying with numerous other traffic and site use working hours and restrictions. In this manner, there is a very delicate “dynamic balance” between essential construction activities and local neighborhood interests—all while maintaining the day-to-day pace in the construction schedule.

All work for the construction of the station caverns and south tunnels was performed through two temporary shafts located on 2nd Avenue near 69th and 72nd Streets. Significant all-weather temporary enclosures were built at these locations to minimize construction-related environmental issues such as noise, dust, odors, construction activities, security and appearance within the well-established neighborhoods. These structures were also designed to enclose the electrically-powered (silent) overhead

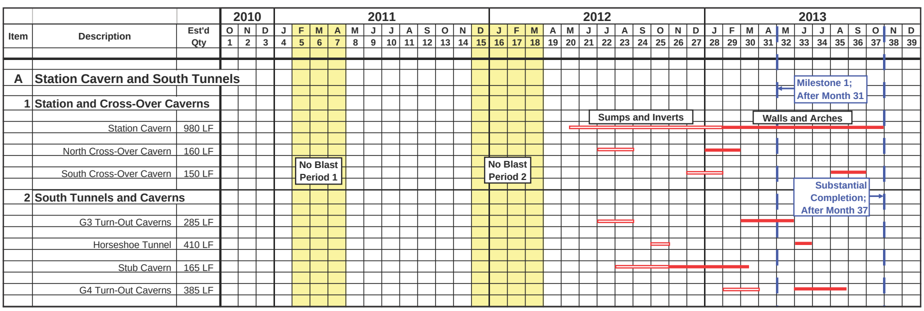

Figure 9. Overall final lining concrete schedule for the project—including all caverns, tunnels, adits, surface shafts and entrances. The work required many concurrent operations while constantly supplying concrete from drop shafts.

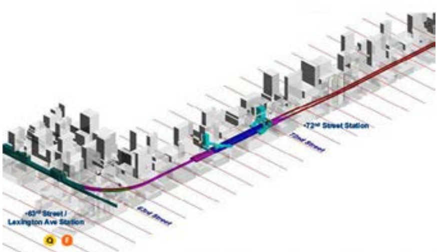

Figure 10. Rendering of the general project layout along 2nd Avenue—from 73rd to 63rd Streets

Figure 11. Aerial view of 2nd Avenue in the area of the project site. Note the dense building development.

Figure 12. Construction of one of two Muck Houses erected at the site—for efficient materials handling

Figure 13. Operational Muck House at 72nd Street site. Ventilation and electrical systems are also enclosed.

Figure 14. Aerial view along 2nd Avenue as the shaft sinking work started in late 2010 at 69th and 72nd Streets. Four lanes of traffic had to be maintained except for specific periods of the day—Monday to Friday.

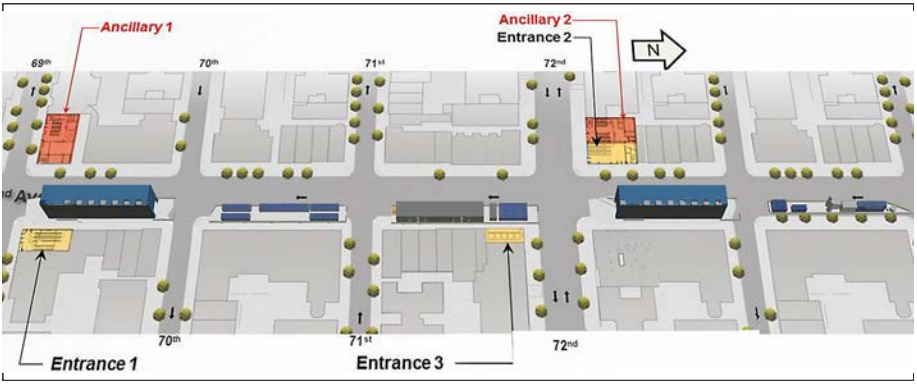

Figure 15. Rendering of the site with temporary facilities in place. These facilities include Muck Houses equipped with electrically-powered overhead cranes for materials handling, field offices, materials storage and water treatment.

gantry cranes that were used throughout the excavation and final lining phases. Muck handling and sequent concrete form and rebar handling tasks were efficiently performed by the overhead cranes from within these enclosures (Figures 12 and 13).

Figure 14 provides an aerial view of the site on 2nd Avenue at the start of the project, with the two temporary construction shafts being excavated. Later and as the underground operations progressed, two Muck Houses were constructed over the shafts. Refer to Figures 15–18.

Figure 16. Interior view inside one of two construction shafts at the site

Figure 17. Looking up inside one of two construction shafts at the site

Figure 18. Hoisting a muck box in the shaft—for off-site disposal

The Contract imposed significant daily restrictions on the performance of the surface work activities even though underground operations could potentially proceed on a 24-hour per day basis, 7 days per week. In general, all surface work operations could be performed from 7:00 AM to 10:00 PM daily, Monday to Friday with restricted hours on weekends. Blasting was restricted to the hours of 7:00 AM to 7:00 PM.

Overall, there was reasonable "give and take" in light of the confined site and restrictive working conditions—with neighborhood concerns frequently addressed along with periodic construction-related special operations needing accommodations outside of the normal work space and hours.

GEOLOGICAL CONDITIONS AND SUBSURFACE BEHAVIOR

The anticipated geological conditions and behavior for the project were described in the Contract Documents that included a GBR (contractual), a GDR and GIR (reference documents only). The site geological profile was enclosed in the GIR. An extensive geological investigation program was undertaken by the MTA for the entire Second Avenue Subway Program with key portions of this substantial task incorporated into the Contract Documents.

The rock, soil and ground water conditions were anticipated and later found to be generally very good for the construction of the caverns, tunnels, adits and inclines. In broad terms, the rock was found to be very amenable for drill and blast excavation methods while being far too hard for roadheader and demolition hammers. Only in a few isolated locations where faults and shears were encountered, was the repetitive drill, blast, bolting and shotcrete cycle interrupted for more intensive excavation and/or ground support measures. The soil layer (including historical fills) was shallow but highly variable in depth. Ground water inflows were expected to be modest, localized and controllable through panning and sumping methods. The following subsections will describe the anticipated and "as encountered" conditions and behavior for rock, soil and ground water to date and as the project approaches the final stages of excavation.

Rock Conditions and Behavior

Throughout the project site, the rock was generally Manhattan schist with some intrusions of amphibolite, granite and pegmatite. In all but a few locations, the rock conditions

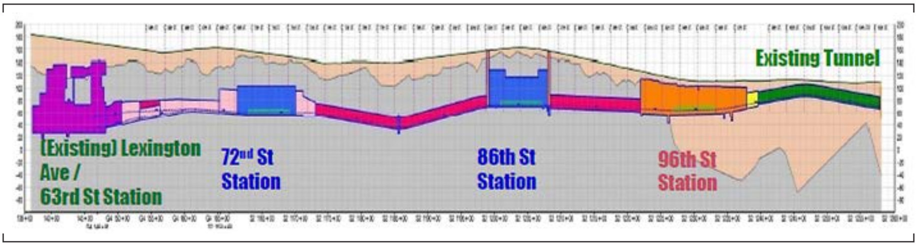

Figure 19. Simplified geotechnical profile of the Second Avenue Subway showing all three stations and tunnels

were anticipated to be tight, largely impermeable and respond well to controlled blasting techniques without extensive support or ravelling. Accordingly, the prescriptive ground support design, known as *Initial Support* together with the detailed excavation sequence, took account of the relative consistency and anticipated behavior of the rock for all caverns, tunnels and adits. In only a few locations based on information in the GBR, GDR and GIR documents were the rock conditions anticipated to be difficult and potentially in need of *Additional Support* provisions. The simplified geological profile for the Second Avenue Subway route is shown in Figure 19.

Soil Conditions and Behavior

Soil (and historical fill) conditions were encountered in all Ancillary and Entrance excavations. The soil cover depth varied considerably from location to location but was generally in the range of 5 to 20 feet. Prior building construction at the sites contributed to the depths encountered. When found, undisturbed soil layers consisted of naturally occurring sands and silts, often in gouge areas overlying weathered bedrock.

Only in the Ancillary excavations was the soil layer deeper (highly variable in depth) and required Support-of-Excavation systems—for retention of soil and surrounding utilities as well as for the overlying temporary decking system. Due to the modest quantity of ground water, Support-of-Excavation systems and designs did not need to be watertight.

Ground Water Conditions and Inflows

The ground water table was located above the caverns and tunnels throughout the project site and may have been influenced by tidal fluctuations in the nearby East River. Just the same, and with the benefit of tight rock conditions, only small inflows were anticipated. This proved to be the case and especially after the underlying TBM-driven tunnels had been completed under a prior contract. During the excavation and final lining phases in the caverns and tunnels, only small quantities of ground water have been encountered and were easily captured with panning methods. To date, no grouting for ground water control has been required.

ROCK EXCAVATION AND GROUND SUPPORT

The majority of the work (measured in cost and schedule time) for the project has been devoted to rock excavation by drill and blast methods. Mechanical and chemical excavation methods were only seldom used—with limited success owing to the in-situ rock quality. The following subsections will describe the rock excavation methods, materials and equipment as well as the ground support systems used in the station and turn-out caverns, tunnels and adits.

Table 1. Summary of the prescribed ground support, generally known as *Initial Support*. These materials and installations were used in all designated areas of the site—without regard to the ground conditions encountered.

Item	Description	Overall Length	Principle Dimensions				Estimated Qty		Principle Ground Support Materials				
			W	H	Arch	Slope	Face	Vol	SFRS	Bolts	Dowels	Spiles	Girders
A	**Station Cavern and Access**												
	Station Cavern	980 LF	68'-10"	48'-10"	variable	0°	3,125 SF	113,400 BCY	●	●	●		
	Construction shafts	85 VF	30'-0"	85 VF	2 each	vertical	707 SF	4,450 BCY	●	●	●		
B	**Cross-Over Caverns**												
	North Cross-Over	155 LF	61'-0"		variable	0°	1,777 SF	10,200 BCY	●	●	●	❍	
	South Cross-Over	165 LF	61'-0"		variable	0°	1,777 SF	10,860 BCY	●	●	●	❍	

Figure 20. Drilling the top heading west side slash in station cavern after completing the center drift

Figure 21. Final muck removal in the station cavern after completing the cross-overs and bench excavation

Station Cavern Excavation and Ground Support

The project included the excavation of six caverns in addition to tunnels and adits for Ancillary shafts and Entrances. Table 1 summarizes the principle dimensions and quantities of rock excavation, without regard to the initial "pilot" tunnels (TBM-bored) underlying the Station Cavern, Cross-Overs and throughout the south turn-out caverns.

The above-listed work required approximately 20 months to complete but not before the permanent final (concrete) lining had commenced. The work was performed on a three shift-per-day basis in all areas with a fleet of underground equipment specially designed for this work and supported with surface hoisting facilities as described above. Figures 20, 21 and 23 illustrate the work underway, whereas Figure 22 is a diagram of the general work sequence used for the excavation of the station cavern.

All blasting for the caverns and tunnels was performed under the guidance and authority of the New York City Fire Department (FDNY) who also provided licenses for powder handlers and Blasters-in-Charge as well as permits for the supply of explosives to the site. All underground blasting was performed on swing shift, Monday to Friday.

Powder and detonators used were all commonly available from Austin Powder Company and included, for example, Emulex (emulsion) and Red-E Lite-D (trim powder) as well as 200/5,000 milli-second non-electric detonators and 9 to 42 milli-second surface delays. No primacord or ANFO was used anywhere on the project—in accordance with FDNY.

The quantity of powder per detonator varied linearly with the blast hole depths and would range from 3.0 to 9.0 lbs per delay. The Powder Factor for a typical top-heading (center-cut) round varied from 4.5 to 6.0 lbs/BCY (Figures 24–25).

Blast hole patterns and loading for the station cavern varied considerably in accordance with the face area, round length (vibration limited) as well as other factors

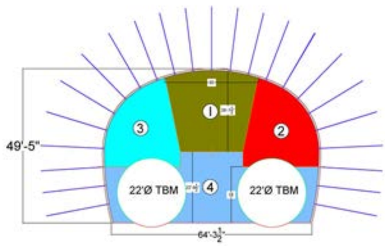

Figure 22. Station cavern excavation sectional plan showing the top heading and bench excavations

Figure 23. Holing-through the center top heading ① before starting the east and west side slashes, ② and ③

Figure 24. Final stage in the station cavern—for the removal of the rock surrounding the TBM tunnel

Figure 25. Final stage of station cavern excavation before the start of final lining operations

Table 2. Summary of the prescribed ground support, known as *Initial Support*. These materials and installations were used in all designated areas of the site—without regard to the actual ground conditions encountered.

Item	Description	Est'd Qty	Blast Round Length (Plan)			Prescribed Ground Support Materials and Installation											
						Shotcrete		Rock Bolts					Rock Dowels				
			Var	10'	12'	5"	7"	Dia	Len	Tension	Load	Pat'n	Dia	Len	Tension	Pat'n	
A	**Station Cavern and Cross-Overs**																
	Main Station Cavern	980 LF		❍	●		●	1.25"	20'	Yes	30 kip	6' x 6'	1.25"	20'	No	6' x 12'	
	North and South Cross-Overs	310 LF		●			●	1.25"	16'	Yes	20 kip	6' x 6'	1.25"	20'	No	6' x 12'	

including geological conditions, rock fragmentation and ground support considerations. Similarly, the prescribed *Initial Support* varied in accordance with the overall dimensions of the heading and included tensioned; fully resin-grouted rock bolts above springline on a preset pattern in addition to untensioned fully resin-grouted dowels in the walls below springline, also on a preset pattern. The steel fiber reinforced shotcrete (SFRS) layers varied also with the heading dimensions—from 5 to 7 inches thick. Refer to Table 2.

Limiting ground vibration values in many areas of the project often curtailed the blast round length and, therefore, the total weight of powder per delay. Notwithstanding, the maximum round lengths were shown in the Contract documents and varied from 10 to 24 feet. Field vibration measurements were intended to only measure ground transmitted blast energy, and not dynamic building response. Refer to Table 3 and Figure 26.

Table 3. Summary of the prescribed (and adjusted) ground born allowable blast vibrations; velocity and frequency data, as measured at the location of various buildings and utility systems along the route tunnel and cavern route

Item	Description	Blast Vibration Data					Affected Work Areas					
		Original Values			Revised Values		Caverns			Tunnels and Adits		
		Velocity	Freq.	Distance	Velocity	Freq.	Station	T/Os	Stub	G3	G4	Adits
1	Normal Buildings	1.92 ips	>40 hz		1.92 ips	>40 hz	●	●	●	●	●	●
2	Fragile Buildings	0.50 ips	>40 hz	None	1.20 ips	>40 hz	●	●	●	●	●	●
3	Sensitive Buildings	0.50 ips	>40 hz	None	1.20 ips	>40 hz	●					●
4	Historic Buildings	0.50 ips	>40 hz	None	1.20 ips	>40 hz	●	●	●	●	●	●
5	Landmark Buildings	**0.50 ips**	>40 hz		**0.50 ips**	>40 hz		●		●		
6	Underground Utility Systems	0.50 ips	>40 hz		0.50 ips	>40 hz	●		●			●

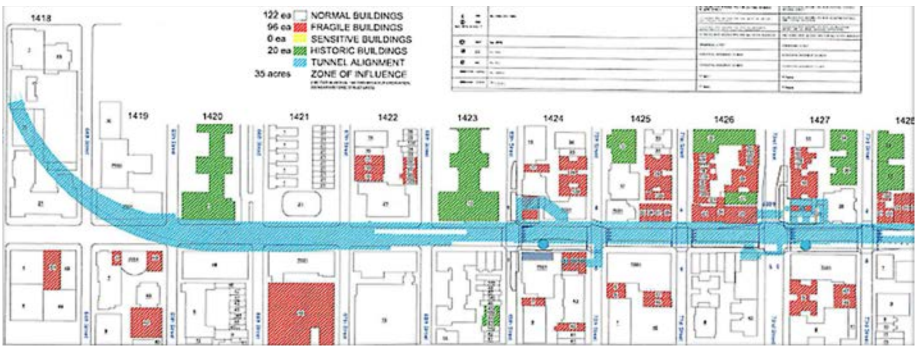

Figure 26. Plan of the project site showing the type and number of buildings having special character and classifications related to vibrations from underground and surface blasting

Considerable attention was given to blast vibration measurements and control and especially when excavations were in the vicinity of fragile, sensitive, and historic structures where the threshold value was restricted to 0.50 in/sec. After some discussions, this threshold value was adjusted to the levels, listed in Table 3. The required placement of seismographs, vibration measurements were greatly influenced by the building dynamic response and not solely blast induce ground vibrations.

A more detailed study of ground and building vibration responses is warranted in light of the Contract directed means for placement and measurement of "fixed" and "floating" seismographs. Only in areas where the "fixed" and "floating" seismographs were properly installed—in a manner to avoid measurement of building responses—could the true blast induced ground transmitted energy be correctly measured.

SOUTH TUNNEL AND ADIT EXCAVATION AND GROUND SUPPORT

The project included the excavation of one running tunnel (Horseshoe Tunnel) as well as ten unique adits to connect the station cavern to the adjacent Ancillaries and Entrances. A small cross-passage was also excavated between the G3 and G4 running tunnels. Table 4 summarizes the principle dimensions and quantities of rock excavation

Table 4. Principle dimensions and the quantities related to excavation of the south tunnels in addition to adits leading from the station cavern to the ancillaries and entrances

Item	Description	Overall Length	Principle Dimensions				Estimated Qty		Principle Ground Support Materials				
			W	H	Arch	Slope	Face	Vol	SFRS	Bolts	Dowels	Spiles	Girders
A	**South Tunnels**												
	Horseshoe Tunnel	410 LF	22'-0"	22'-0"	variable	variable	405 SF	6,150 BCY	●	●	●		
	Cross-Passage	20 LF	10'-0"	10'-0"	variable	variable	113 SF	85 BCY	●	●	●		
B	**Station Area Adits**												
	Ancillary 1												
	Ventilation tunnel	38 LF	24'-10"	20'-4"	variable	0°	589 SF	830 BCY	●	●	●	●	❍
	Egress / Service tunnel	110 LF	35'-0"	18'-0"	variable	0°	655 SF	2,670 BCY	●	●	●	●	❍
	Ancillary 2												
	Ventilation tunnel	80 LF	24'-10"	20'-4"	variable	0°	457 SF	1,350 BCY	●	●	●	●	❍
	Service tunnel	20 LF	20'-0"	18'-0"	variable	0°	328 SF	250 BCY	●	●	●	●	❍
	Egress tunnel	20 LF	20'-0"	28'-6"	variable	0°	539 SF	400 BCY	●	●	●	●	❍
	Entrance 1												
	Access adit	75 LF	24'-10"	17'-1"	variable	0°	377 SF	1,050 BCY	●	●	●	●	❍
	Escalator incline	70 LF	24'-10"	17'-1"	variable	30°	536 SF	1,390 BCY	●	●	●	●	❍
	Entrance 2												
	Access adit	60 LF	24'-10"	17'-1"	variable	0°	428 SF	950 BCY	●	●	●	●	❍
	Escalator incline	70 LF	24'-10"	17'-1"	variable	30°	511 SF	1,325 BCY	●	●	●	●	❍
	Entrance 3												
	Access adit	17 LF	29'-0"	17'-9"	variable	0°	450 SF	1,090 BCY	●	●	●	●	❍
	Emergency tunnel	90 LF	10'-0"	11'-0"	variable	0°	97 SF	825 BCY	●	●	●	●	❍

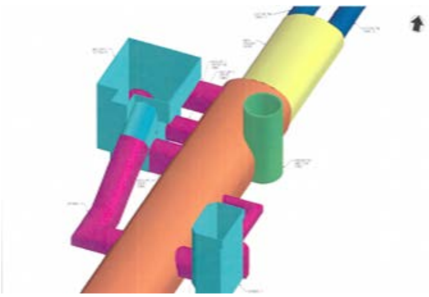

Figure 27. Rendering of north portion of the station cavern, showing Ancillary 2, Entrances 2 and 3

Figure 28. Drilling an adit to Ancillary 2 from the station cavern area—as the top heading proceeds

for the tunnels and adits. Figures 27 and 28 illustrate the nature and complexity of station adits.

The above-listed work required approximately 10 months to complete but not before the permanent final (concrete) lining had commenced. The work was performed on a three shift per day basis in all areas with a fleet of underground equipment specially designed for this work and supported with surface hoisting facilities as described above. Figures 29 and 30 illustrate the work underway in the turn-out and stub cavern enlargements.

Powder and detonators used were all commonly available from Austin Powder Company and included, for example, Emulex (emulsion) and Red-D Lite-E (trim powder) as well as 200/5,000 milli-second non-electric detonators and 9 to 42 milli-second surface delays.

Blast hole patterns and loading data for the tunnels and adits varied considerably in accordance with the face area, round length (vibration limited) as well as other factors including geological conditions, rock fragmentation and ground support considerations. Similarly, the prescribed *Initial Support* varied in accordance with the overall dimensions of the heading and included tensioned, fully resin-grouted rock bolts above springline on a prescribed pattern in addition to untensioned fully resin-grouted dowels

Figure 29. Drilling for rock excavation in the G4 turn-out cavern. A previous TBM tunnel is on the left.

Figure 30. Completion of the G3 and G4 tunnels at the Stub Cavern—with a final separation of only 6 feet

Table 5. Summary of the prescribed ground support, known as Initial Support. These materials and installations were used in all designated areas of the site—without regard to the actual ground conditions encountered.

Item	Description	Est'd Qty	Blast Round Length (Plan)			Prescribed Ground Support Materials and Installation										
						Shotcrete		Rock Bolts					Rock Dowels			
			Var	10'	12'	5"	7"	Dia	Len	Tension	Load	Pat'n	Dia	Len	Tension	Pat'n
A	**South Tunnels and Caverns**															
	G3 - Cavern I	85 LF	○	●			●	1.25"	14'	Yes	20 kip	6' x 6'	1.25"	14'	No	6' x 12'
	G3 - Cavern II	185 LF	○	●		●		1.25"	12'	Yes	20 kip	6' x 6'	1.25"	12'	No	6' x 12'
	G4 - Cavern I	285 LF	○	●			●	1.25"	14'	Yes	20 kip	6' x 6'	1.25"	14'	No	6' x 12'
	G4 - Cavern II	85 LF	○	●		●		1.25"	12'	Yes	20 kip	6' x 6'	1.25"	12'	No	6' x 12'
	Horseshoe Tunnel	410 LF	○	●		●		1.25"	10'	Yes	20 kip	6' x 6'	1.25"	10'	No	6' x 12'
	63rd Street Stub Cavern	165 LF	○	●		●		1.25"	14'	Yes	20 kip	6' x 6'	1.25"	14'	No	6' x 12'
B	**Adits and Inclines**															
	Ancillary Adits	Various	○	●			●	1.25"	10' - 12'	Yes	20 kip	5' x 5'	1.25"	20'	10' - 12'	5' x 10'
	Entrance Adits and Inclines	Various	○	●		●		1.25"	12'	Yes	20 kip	5' x 5'	1.25"	20'	12'	5' x 10'

in the walls below springline, also on a prescribed pattern. The steel fiber reinforced shotcrete (SFRS) layers varied also with the heading dimensions—from 5 to 7 inches. Refer to Table 5.

FINAL CONCRETE LINING—CAVERNS AND TUNNELS

Station Cavern and Cross-Overs—Inverts, Walls, and Arch

Final lining operations in the station and cross-over caverns involved considerable planning to allow for multiple concurrent operations for inverts, walls and arches. To this end, the deep sump at the 72nd Street shaft was completed first, followed by inverts in north and south directions, with station walls cast immediately afterwards. Refer to Figures 31 & 32. The station and North Cross-Over arch forming systems were deployed following the completion of a minimum number of station wall pours.

Station Inverts

The station and cross-over inverts were complicated to the extent that they were considered to be the most complex and burdensome of all concrete pours on the project. This was due to the extensive system of embedded ductile iron and PVC pipes and

Figure 31. Constructing a station invert slab with rebar and forms. Note the waterproofing in the NXO.

Figure 32. Placing concrete in a station invert slab after all forms, rebar and piping were in place

Figure 33. Station cavern single-wall form in operation—following a checker-board pattern

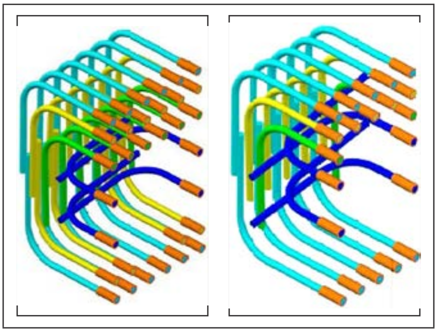

Figure 34. Renderings of complex prefabricated beam couplers needed for the station lower wall pours

fittings needed for the pressure relief drain system in addition to the traditional track drain system—both discharging into the large sump pit at the north end of the station cavern. All invert pours were 60 feet long and coincided with the placement of water barrier materials. PVC waterproofing membrane was installed throughout the entire underside of all invert pours. A double layer of reinforcing steel was also placed.

Station Lower Walls

The station lower wall pours were limited to 30 feet in length and included 33 pours on each side of the station cavern. There were no discrete wall pours needed for the cross-over caverns owing to the shape of the arch that extended to the invert level. Whereas the station lower wall pours had little embedded piping materials, they did, however, include several special forming arrangements needed to suit the intersecting adits—inverts and walls.

All station lower wall pours were formed and poured using MCT single-wall forms as shown in Figure 33. The forms were rail-mounted and included pour platforms and a slickline distribution system. The station wall pours included a double layer of reinforcing steel as well as a complex prefabricated beam coupler arrangement—needed for the follow-on mezzanine beams and slab pours. Refer to Figure 34.

Station Arches

The station and cross-over arch pours were limited to 30 feet in length and included 33 pours overall for the station cavern in addition to 5 more for each Cross-Over cavern. Whereas the station arch pours had no embedded piping materials they did include formed coffer recesses and embedded unistrut channels in public areas. Special forming arrangements were needed to suit the intersection of numerous adits.

All arch pours were formed and poured using MCT arch forms as shown in Figures 35, 36 & 37. As shown, the station arch forms, gantry crane and rebar template were all located on a mobile, rail-mounted platform deck—needed to efficiently access and construct the arch as well as place key portions of reinforcing steel. The forms were rail-mounted on the deck and included pour platforms, a slickline and placer concrete distribution system. The station arch pours required a double layer of reinforcing steel.

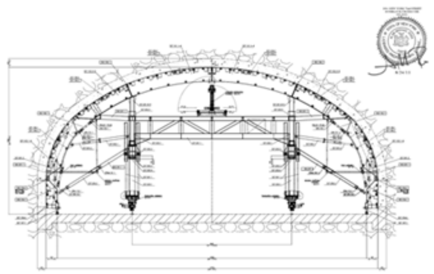

Figure 35. Cross-Over arch form sectional drawing. This was a custom-built form for the NXO and SXO.

Figure 36. Cross-Over arch form in place in the North Cross-Over—ready for the first pour

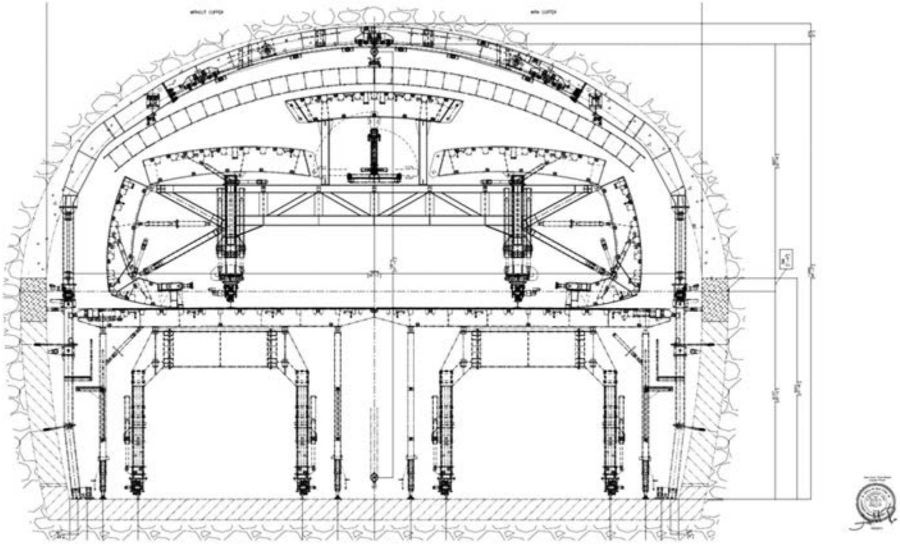

Figure 37. Detailed view of the elaborate custom-built station arch form and traveller over top of a rail-mounted sectional decking system. The lower station walls are poured first, to be followed by the cavern end walls and arch pours—while connecting the adjacent adits—to entrance and ancillary facilities.

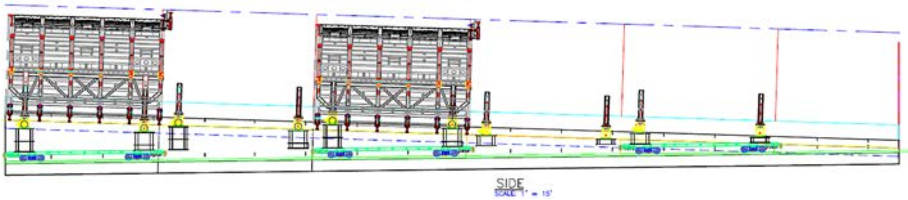

Figure 38. Planned general arrangement for installing the final lining in the Turn-Out cavern arches—after accounting for complex track alignment and grade changes

Turn-Out and Stub Caverns—Invert and Arch

Final lining operations in the turn-out and stub caverns involved considerable planning to allow for multiple concurrent operations for inverts, walls and arches. To this end, the lower level of the 63rd Street Stub Cavern was completed first, followed by inverts in the G3 and G4 running tunnels, then inverts in the G3 and G4 turn-out caverns—all in a northward direction—retreating from the 63rd Street Bellmouth area. Cavern arches followed afterwards. The final concreting stage required lining of the running tunnel arch.

Cavern Inverts

The turn-out cavern inverts were complicated to the extent that they had an extensive system of embedded ductile iron and PVC pipes and fittings needed for the pressure relief drain system in addition to the traditional track drain system—both ultimately discharging into the large sump pit at the north end of the station cavern. All invert pours were 60 feet long with construction joints coinciding with the location of water barrier materials. PVC waterproofing membrane was installed throughout the entire underside of all invert pours. Whereas steel fiber reinforced concrete (SFRC) was used for the adjacent running tunnel inverts, a double layer of reinforcing steel was placed in the cavern inverts (and arches).

Cavern Arches

The turn-out and stub cavern arch pours were limited to 30 feet long and included 27 pours overall; 5 each for the stub cavern, 9 each for the G3 cavern, and 13 each for the G4 cavern. Special temporary works were needed for the arch forms in the G3 and G4 caverns due to the slope and bifurcating track alignments.

All arch pours were formed and poured using MCT arch forms as shown in Figure 38. As shown, the arch forms were all mounted on a mobile, rail-mounted form traveller—needed to efficiently form and pour the arch. The forms included pour platforms and a slickline and placer concrete distribution system. The cavern arch pours required a double layer of reinforcing steel placed in advance of arch form assembly.

The 63rd Street Stub Cavern was a difficult undertaking owing to the over/under configuration of the tracks and tie-in to the existing Bellmouth structure. The lower portions this structure was straightforward and required single-wall forms and soffit shoring. Following this, a custom-built arch form was obtained to place the final lining in the arch along the curved track alignment. Refer to Figures 39 and 40.

RUNNING TUNNELS—INVERTS AND ARCH

The TBM bored and Horseshoe running tunnel final lining was divided between inverts and arch pours. Overall, there was approximately 2,400 LF of tunnel split into four separate sections—all having the same final interior dimensions and reinforcing

Figure 39. Stub Cavern walls at G4 tunnel. A soffit slab will follow to complete this over/under structure.

Figure 40. Stub Cavern with G4 (lower track level) completed. A soffit slab and arch pours will follow.

Figure 41. Invert waterproofing installation in the running tunnel. The blue material is water barrier.

Figure 42. Tunnel invert forms in place for the pour. A follow-on arch pour will complete the tunnel lining.

requirements. Difficulties, however, were experienced whenever the tunnel arch lining intersected a cavern. In these cases, additional special forms were needed to the address the transition geometry from flat walls to curved walls and arches in the tunnel. Figure 41 shows the general arrangement of the waterproofing in these tunnels and Figure 42 illustrates the running tunnel invert form general arrangements.

WATERPROOFING SYSTEM AND MATERIALS

All tunnels and caverns were designed to not only be fully drained but also to be fully enclosed in a waterproofing system. The specified waterproofing system included the following components—supplied and installed by WISKO America under a fixed price Subcontract Agreement. A performance warranty was also provided.

- Fleece layer in all areas
- Geodrain layer in specified locations
- PVC membrane
- PVC water barrier materials
- Grouting tubes

Figure 43 illustrates the general arrangements and components of the waterproofing system, linked the pressure relief drainage system in the tunnels and caverns, throughout the project. The complexity of the pressure relief drainage system should not be overlooked since it required careful planning and integration into many separate operations; including waterproofing, reinforcing steel and concreting. Figure 44 provides a view of a completed installation in one of the running tunnels.

ANCILLARIES AND ENTRANCES

The station cavern construction required numerous connections to adjacent Entrances and Ancillary areas. While the Entrances are generally for public needs, the Ancillary areas were needed to enclose essential services to the station—for fire life safety needs as well as for routine station operations and maintenance. The following provides a summary of the excavation and the *Initial Support* requirements for the entrances and ancillaries. There was no final lining requirement needed in these areas under the Contract since this work was assigned to the follow-on finishing contract. Refer to Figures 45 & 46 for renderings of these work areas.

Excavation and Initial Support Scope and Requirements

The Ancillaries and Entrance excavations were constructed in similar ground conditions and, therefore, required similar ground support provisions. These excavations were approached as shafts with multiple faces—to sequence the progress of the advance in a manner to allow for manageable blast size while concurrently installing ground support.

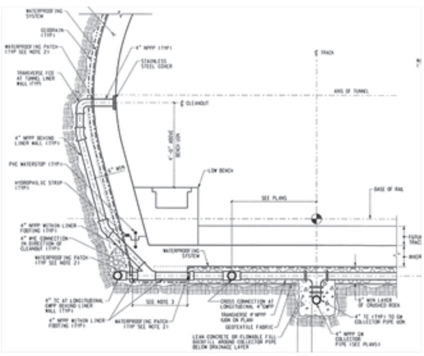

Figure 43. Sectional view of the waterproofing and drainage system. Similar for all areas of the site.

Figure 44. Completed waterproofing materials in the Horseshoe Tunnel, complete with water-barrier

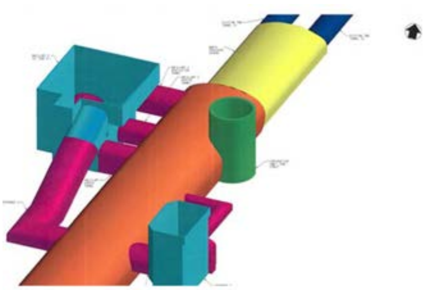

Figure 45. Rendering of adits for Entrances 2 & 3 and Ancillary 2—all at the north end of the station

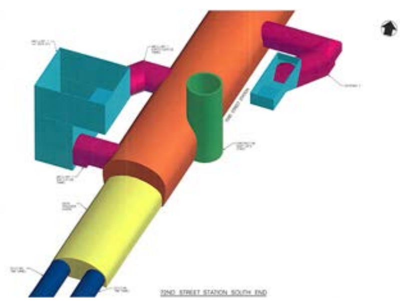

Figure 46. Rendering of adits for Entrance 1 and Ancillary 1—all at the south end of the station

- **Ancillary 2**
  - 14,000 BCY rock by drilling and blasting methods
  - Final bottom elevation 80’ below street level
  - Full channel drill near columns and in areas of faulty rock (5" holes)
  - 10' Swellex rock bolts on 6' × 6' pattern
  - 5" minimum SFRS shotcrete liner
  - Escalator adit mined primarily from station cavern
  - Egress, Service, and Ventilation adits excavated from station cavern
- **Entrance 3**
  - 3,229 BCY soil excavation (ENT-3 and ANC-2 combined)
  - 5,600 BCY rock excavation by drilling and blasting methods
  - Final bottom elevation and 86' below street level
  - Extensive planning and coordination in “Top Down” break-in shots
  - Emergency tunnel mined from station cavern
  - 10’ Swellex rock bolts on 6' × 6' pattern
  - 5" minimum SFRS shotcrete liner
- **Ancillary 1**
  - 2,104 BCY soil excavation
  - 11,800 BCY rock excavation by drilling and blasting methods
  - Final bottom elevation 78’’ below street level
  - Full channel drill near columns and in areas of faulty rock (5" holes)
  - 10' Swellex rock bolts on 6' × 6' pattern
  - 5" minimum SFRS shotcrete liner
  - Egress/Service, and Ventilation adit tunnels excavated from the station cavern

Temporary Decking Systems

Elaborate temporary decking systems were designed, fabricated and installed at the shaft sites at Ancillaries 1 and 2 as well as at Entrance 3. These decks were designed to support unique construction and equipment loads owing to the confined nature of the sites and the specialized material handling needs during construction periods.

Support-of-Excavation Systems (Soils)

Support-of-Excavation systems were designed and installed at the shaft sites at Ancillaries 1 and 2 as well as at Entrance 3. These system were not designed as water tight structures but were very effective for retaining the soil and fill layers, prior the placement of temporary decks and the advancement of shaft excavation through rock by drilling and blasting methods. The principle components included the following.

- Minimum 5" SFRS shotcrete liner
- No. 10 Dywidag bars, 10' long at 6' × 6' pattern
- Additional rock support (25' Grade 150 No. 11 bars) required in certain areas around deck beam support columns
- Mine straps and welded wire mesh as required

Schedule and Work Sequences

The construction of the entrances and ancillary shafts was a challenging portion of the project—largely due to their location, size, depth and to some extent, the intricacies of the excavation surrounded by multi-storey buildings. The schedule and coordination efforts required attention to the following tasks, for example.

- Coordination with the station cavern during blasting operations and drift break-ins (9 each)
- ANC 2 was on the Critical Path and, therefore, needed special attention
- All adits mined from station cavern to allow excavation to continue
- Building demolition delays caused impacts to excavation schedules

CHALLENGES FOR PROJECT COMPLETION

At the time of this writing, approximately 70% of the entire scope of work (and time) has been accomplished. The work is generally on schedule with some Extensions-of-Time for the performance of Extra Work pending MTA approvals. Nonetheless, there are still many challenges to address as the project moves quickly into the final lining stages in all remaining areas of the site. While the station cavern and the northern portion of the work are subject to the *Milestone 1* date, concurrent completion of the south tunnels and caverns is also important. At present there are *seven active areas* receiving the final liner. These specialized operations require five arch and two wall forming systems in addition to separate tunnel invert and arch forms. The work includes the concrete lining of cavern inverts, walls, arches as well as tunnel inverts and arches—all on a well sequenced and closely coordinated basis—while linked to *Milestone 1* and the *Substantial Completion* dates.

Milestone 1—North of Grid Line 17 in the Station Area for Turn-Over

As described earlier, *Milestone 1* occurs at the end of Month 31 in the CPM schedule. The work includes excavation and final lining from Station Grid Line 17 northward—or approximately 40% of the station length together with the North Cross-Over, Entrance 2, and Ancillary 2. Six out of the nine adits leading from the station cavern are included. Overall, the coordinated work requires waterproofing, reinforcing steel, forming and concrete placing from the 72nd Street construction shaft while the final excavation phases in Ancillary 2 (4 adits) and Entrance 3 (2 adits) are still underway.

Substantial Completion—Entire Remainder of the Job for Turn-Over

The *Substantial Completion* date occurs at the end of Month 37 in the CPM schedule. Whereas, *Milestone 1* addressed completion and turn-over of the northern ±40% of the station area, *Substantial Completion* defines the completion and turn-over the remaining portion of the project, including the south tunnels and turn-out caverns. Only Punch List tasks will be outstanding after the *Substantial Completion* date. At present, the south turn-out caverns are being concrete lined on a sequential basis using three separate arch forming systems—to be followed later with other arch forms in the running tunnels. In general, the G3 and G4 running tunnels will be lined concurrently in a retreat direction from the 63rd Street Station Bellmouth area. This will provide for an efficient and concurrent use of all forming systems, followed by an early turn-over of these tunnels and caverns to the MTA for the follow-on Systems and Finishes Contract.

CONCLUSIONS

This project is generally considered to be one of the more difficult challenges in the Second Avenue Subway construction program. This is due to the scope and complexity of the work in conjunction with the fast-paced schedule and milestone dates. Limited access to the underground work areas in addition to street level restrictions have had a continuous influence on planning and day-to-day construction operations. Nonetheless, and after over two years continuous successful construction activities, the project has progressed well and is tracking for completion in the scheduled time. The ground conditions for excavation have generally been favorable and with few exceptions, the prescribed *Initial Support* has been satisfactory. Concreting operations started as planned during the final stages of rock excavation and have grown to include seven separate concurrent operations. Custom-built wall and arch forms are in use for the placement of the final cast-in-place concrete lining in the tunnels and caverns.

ACKNOWLEDGMENTS

The authors would like to acknowledge the considerable contributions to this paper (and corresponding oral presentation) from the MTACC (Program Manager), NYCTA (Project Owner), Aecom-Arup (Project Designers), Parsons Brinckerhoff (Construction Manager) and Hatch Mott MacDonald (SSK's geotechnical engineer). In particular, very helpful information was provided and included in this paper from Bill Goodrich and Anil Parikh (MTACC), Tom Peyton, Amitabha Mukherjee, and Jaroslav Prokop (PB) as well as Jaidev Sankar and Verya Nasri from Aecom-Arup. Additionally, Julian Prada, Eric Jordan and Philip Lloyd from Hatch Mott MacDonald provided timely assistance and review.

REFERENCES, SELECTED READINGS, AND ADDITIONAL INFORMATION

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DESIGNER/CONSTRUCTION MANAGER/CONTRACTOR PERSPECTIVE OF DESIGN AND CONSTRUCTION OF THE NEW 72ND STREET STATION OF THE SECOND AVENUE SUBWAY

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Amitabha Mukherjee ▪ Parsons Brinckerhoff

Andrejs M. Delle ▪ Schiavone Construction Co., LLC

ABSTRACT

Construction of New York City's Second Avenue Subway has been through various starts and stops since the 1920s, with little construction progress until recently. MTA Capital Construction commenced construction of Phase 1 in 2007, located on the Upper East Side of Manhattan, which is one of the most densely populated residential areas in the United States. Phase 1 consists of a two-track three station segment of the new subway from 96th Street to an existing station at 63 Street, with provision made for construction of future Phases. The new station at 72nd Street includes the design and construction of caverns excavated in hard rock and highlights the complex considerations of transit construction in an urban environment. The planning, design and construction of this new station was a challenge from the standpoint of managing an aggressive schedule and mitigating impacts to the local community.

HISTORY AND EARLY DESIGN

The history of New York City's Second Avenue Subway has had a long history and has been well documented in newspapers, magazines and similar technical papers. This paper will highlight the design and construction of the new 72nd Street Station under Construction Contact C-26007.

The new 72nd Street Station is one of three new stations that will be part of the first operating segment referred to as "Phase 1" that will extend the Q train from its current northern terminus at 57th Street and 6th Avenue in Midtown Manhattan to 96th Street and 2nd Avenue in the Upper East Side of Manhattan providing much needed relief for the existing Lexington Avenue Line. Currently the Lexington Avenue line is the only subway line which serves the East side of Manhattan.

The Metropolitan Transportation Authority (MTA) awarded a design contract to the joint venture of AECOM-Arup (formerly DMJM*Harris-Arup, JV) at the end of 2001 which included the conceptual and preliminary design of the full 16 station Second Avenue Subway line stretching from 125th Street in the north to Hanover Square in the south. In 2004, the MTA awarded the final design of Phase 1 of the Second Avenue Subway to the AECOM-Arup, JV which included the design of three new stations and the reconstruction of the existing 63 St/Lexington Ave Station. MTA New York City Transit was responsible for the design of the signals and track for the Phase 1 segment (Figure 1).

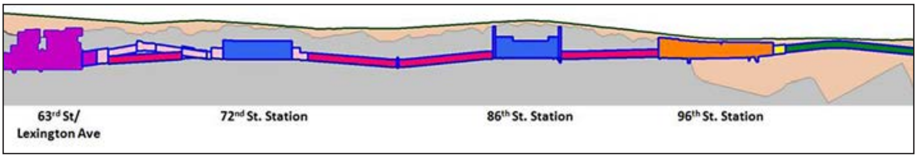

Figure 1. Profile of Phase 1

THE NEW 72ND STREET STATION

The design of the new 72nd Street Station was a collaborative effort which involved the owner, MTA Capital Construction Company (MTACC) and the design team. The MTACC involved the relevant stakeholders within the MTA organization to input in the design requirements and to review design documents.

The location of the 72nd Street station was chosen to be under Second Avenue between 69th and 72nd Streets. The top of sound rock is shallow so the station is designed as a mined cavern in hard rock. This has the advantage of minimizing the amount of shaft work thereby minimizing utility relocations and impacts to the surface. Three entrances were included with the design. The two entrances on 72nd Street provide transfer to the MTA cross-town bus and include one elevator-only entrance making the station accessible to persons with disabilities.

It was decided in the early phases of the project to construct the new station with three tracks in order to provide the optimum train operations characteristics. This is especially true once the full 16 station line is built where the Q and T trains will be operating simultaneously. In order to meet the design and operational requirements, the station cavern had a span of nearly 100 ft and a height of 50 ft.

As design was progressing, technical reviews and constructability reviews were conducted of the station designs. It was decided in order to minimize risk and costs that the station should be reduced to a two track station. The change from three tracks to two tracks reduced the cavern span to 70 ft thereby reducing the mining risk and costs for construction. This was the basis for the final design for the new 72nd Street Station.

72ND STREET STATION CONTRACT PACKAGING

In order to increase competition given the amount of construction work that was being concurrently bid in the New York Metropolitan area, it was decided to create multiple station contracts in order to construct 72nd Street Station and create smaller value contract packages thereby encouraging more contractors to bid.

The first contract is C-26007 which is for the mining and heavy civil portion of the station. The station is then completed in two subsequent contracts: C-26011 includes the station finishes, mechanical, electrical, plumbing and entrance/ancillary construction and C-26009 includes the area-wide communications, signal, track, and traction power systems contract.

C-26007 Contract

Contract C-26007 includes the excavation of the station cavern, two crossover caverns and the complex of tunnels and caverns that connects to the existing station under 63 Street. The cavern and tunnel excavation sequence and initial rock support were prescribed in the contract but could be modified by the Contractor with the Engineer's approval. A Geotechnical Data Report (GDR), Geotechnical Interpretive Report (GIR) and a Geotechnical Baseline Report (GBR) were included. The GDR and GBR were included as Contract Documents while the GIR was included as a reference document.

The 72nd Street Station is approximately 302 m (990 ft) long and located between 69th and 72nd Streets and favors the west side of Second Avenue. The station complex

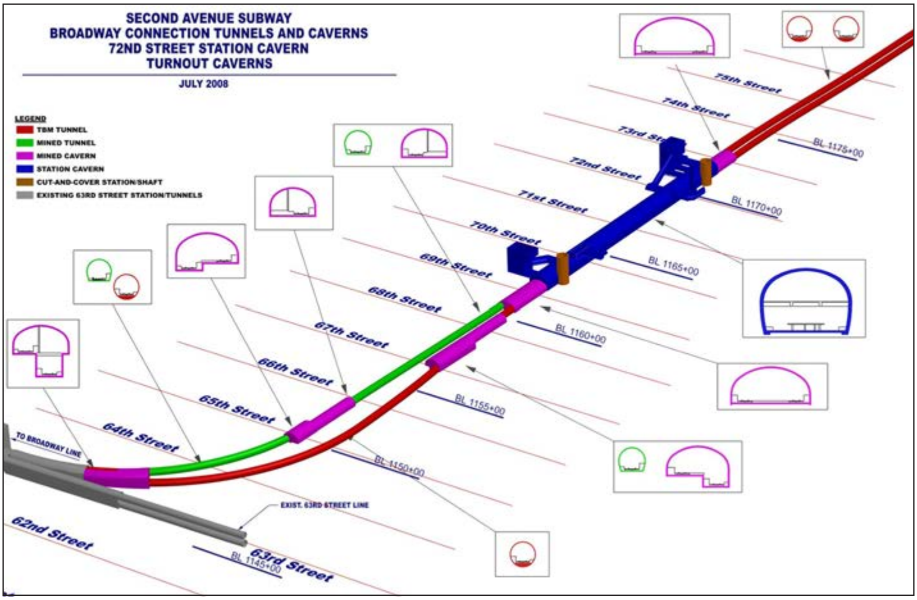

Figure 2. Contract C26007 isometric

consists of the main station cavern, cross-over caverns at both ends, two turn-out caverns to the south of the station to allow for future expansion of the 2nd Avenue line to the south, and a stub-cavern at 63 Street and 3 Avenue to connect to the existing station at 63 Street and Lexington Avenue, three entrances and two ancillary structures which will require modifications and underpinning of existing structures.

The station cavern is approximately 21.3 m (70 ft) wide and 15.2 m (50 ft) high with a minimum rock cover of 9.1 m (30 ft). The caverns are constructed by drill-and-blast and receive an initial shotcrete and final CIP concrete lining. The two track caverns south of 72nd Street range from 9.1 m (30 ft) to 14.6 m (48 ft) spans and make provision for the future extension south for Phase 3 without requiring major service disruptions for the construction. The configuration of the station and caverns is shown in Figure 6.

The running tunnels that connect the stations at 96th street, 86th street, 72nd Street through to the existing Station at 63 Street and Lexington Avenue were constructed under a separate Contract (C26002). As part of that Contract, temporary access shafts for the mining of the 72nd street Station were partially constructed between 69th and 70th street and between 72nd and 73 Streets. These two circular shafts are located on the east side of Second Avenue and is the primary access for the station cavern construction (Figure 2).

The connection to the existing stub tunnel south of 72nd Street station is a combination of TBM tunnels, single track mined tunnels and two track caverns. The two tracks at the station are side-by-side; therefore the geometry of the tracks to the station must take the tracks from the vertical stacked stub tunnel to a horizontal side-by-side position. This track geometry in addition to special track work required to connect to the future Phase 3 tunnels which take the T line south leads to some challenging cavern configurations.

The contract requires the installation of a reinforced concrete lining in the caverns and tunnels. The mezzanine structure and platform in the station were not included in this contract and will be installed in the subsequent contract for station finishes. The contract also includes the installation of a waterproofing system for all lined tunnels and caverns, an invert pressure relief system and a track drainage system.

The three shafts are required to be excavated under this contract for the future ancillary/entrance structures with the permanent structures built in the subsequent contract. Additionally and prior to the excavation of each shaft sites, brick and wood frame buildings acquired by the MTA required asbestos abatement, lead clean-up and demolition.

As this project was conceived as a mining type excavation, by drill and blast method, the full alignment was populated with a full range of monitoring devices including wells, inclinometers, manual survey ground deformation points, automatically monitored survey prisms on building facades, crack gauges, and seismographs.

A final component of the project was building remediation work. An allowance item of $1.5M was incorporated into the Contract to account for any repairs and/or reinforcement of buildings adjacent to the job site and within the blasting influence zone. The intent of the this item was for buildings classified as unfit to withstand blast vibrations, be brought to that level by structural repair or strengthening as specified by the Designer.

CONTRACT MANAGEMENT

The MTA Capital Construction Company engaged Parsons Brinckerhoff to serve as the Consultant Construction Manager (PB/CCM) in 2007 for the entire Phase 1 of the new Second Avenue Subway, encompassing all the construction contracts. The PB/CCM team provides construction management services to the MTACC, working within one of the most unique and complex urban environments in the world. Over and above administering the construction contracts, the major challenge is the coordination of the interfaces between adjacent contracts, coordination with a host of public and private utility companies, local entities, government agencies, and supporting the MTACC in their engagement of the surrounding community so as to minimize impacts on the quality of life along this busy thoroughfare. Included within the realm of prosecuting the Contract, the PB/CCM needs to conduct day-to-day interaction between the contractor, the design team, various departments within the MTACC organization, including quality, safety, procurement, and audits, various departments within the ultimate operator and owner, namely New York City Transit Authority, largely related to satisfying their technical requirement and obtaining their acceptance of as-built conditions, and finally assisting the MTACC in their reporting requirements to the Project Management Oversight Consultant team and the FTA, who have, in part, financed this project.

On the 72nd Street Station construction contract (C26007), staff from the PB/CCM team serve as Construction Manager (CM) and Resident Engineer (RE) and provides a full complement of office engineers, construction inspectors, estimators and schedulers to support the construction of the new station. The PB/CCM team works hand in hand with the contractor and Design team to ensure smooth workflow and communication so that submittal processes and interfaces run smoothly and keeps pace with the Contractor's construction schedule and operations, while at the same time addressing field conditions, constructability issues and value engineering proposals. Regular meetings are held between the contractor, PB/CCM and the design team to address technical issues on a timely basis. This assures that adequate resources are available within all entities in an effort to avoid any delays to the construction.

The construction site is in the heart of the upper east side of Manhattan, a very affluent and densely populated neighborhood, with boutiques, shops and a variety of restaurants at street level. The surface expression of the construction site for the 72nd Street Station contract extends from 73 Street to 68th Streets with the eastern side of Second Avenue through these city blocks dedicated to construction work zones,

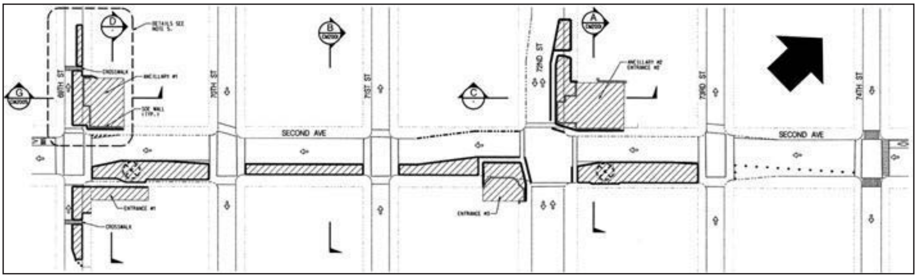

Figure 3. Layout of work zones

approximately two travel lanes wide (Figure 3), while at the same time maintaining four lanes of traffic though the street, as part of an agreement between MTACC and the Department of Transportation (DOT). Additional work zones are also maintained at the northwest and southeast corners of 72nd street and the northeast and northwest corner of 69th Streets to build ancillary and entrance structures for the station. Satellite work zones have also been created on the side streets and further down 2nd Avenue, as required, to facilitate staging areas and for concrete drop holes. The PB/CCM team has been instrumental in coordinating with the DOT and the local community in securing these work zones. Nevertheless, the total amount of surface area available to contractor is less than one-tenth the size of the excavation area underground.

Such a huge undertaking in the midst of a vibrant and busy residential community comes with a host of work restrictions related to limited work hours (especially surface operations), maintenance and protection of traffic, blasting restrictions, blasting and construction related vibration control, noise and dust control, environmental constraints, maintaining access to adjacent buildings, structural protection of adjacent buildings and installations and vector control. Following is a listing of the significant work restrictions:

- Surface work activities are limited to the hours of 7:00 AM and 10:00 PM, Monday through Friday, and from 10:00 AM to 7:00 PM on Saturdays, while underground operations can continue 24 hours per day
- Four lanes of traffic are to be maintained at all times and sidewalks must be maintained at a minimum width of 7 feet
- Muck hauling limited to 7:00 AM to 10:00 PM
- Blasting restricted to 7:00 AM to 10:00 PM, however further restricted to 7:00 PM in response to community concerns
- Blasting restrictions related to the concurrent construction of the station cavern and the TBM mining of the running tunnels under C-26002
- Blast vibration levels limited to 0.5 inches per second (ips) near fragile, sensitive and historic structures and utilities and 1.92 ips at all other structures
- Interior demolition and construction within buildings between 8:00 AM to 6:00 PM
- Various noise restrictions depending upon the type of adjacent areas, but generally background +5 dBA, the background levels being as measured or 75 to 80 dBA during daytime hours and 60 to 65 dBA during night-time hours, whichever is greater. Such restrictions are valid at a distance not more than 50 feet from the noise producing activities.

The PB/CCM team strictly monitors all such restrictions, and coordinates technical mitigation, as well as obtaining concurrence from the affected adjacent property owners

Figure 4. Main station cavern construction

Figure 5. Excavation of the TBM tunnel as part of cavern construction

and government entities, while at the same time remaining cognizant of the needs of the construction and the aggressive 37-month construction schedule mandated by the Contract. In addition to the above, the PB/CCM team manages technical interfaces between contracts, and other coordination interfaces that include site access, traffic congestion, impacts on the public, joint use of work areas. This document will focus on some of these issues.

Contract C26007 included "no-blasting" restrictions wherein no blasting was allowed when the TBM mining the running tunnels (Contract C26002) was directly under and south of the station top heading. The PB/CCM team, in conjunction with both contractors and the Design team, coordinated and implemented a plan where blasting in the cavern (Figure 4) could continue while the TBM was mining, including expediting the development of a MOU between the contractors. This involved shifting the TBM alignment below the cavern zone by approximately 4.5 feet to the east keeping the TBM at the edge of the overall cavern foot print thereby not increasing the rock removal quantities in the station cavern and yet placing a safe distance of more than a half TBM tunnel diameter of rock between the two contract areas of work and coordinating the timing of the cavern rock blasting daily, where the TBM tunnel below would be evacuated only at the time of the rock blast in the cavern. An emergency entry/escape hatchway was also provided from the cavern down to the tunnels for rescue operations, if it ever became necessary. Also, the two areas of the running tunnels below the cavern at the two cavern access shafts were reinforced using steel rib support with steel lagging over the top 120 degrees of arch and additional rock bolts, as necessary, to protect the tunnel, and at the same time, fiberglass rock bolts were installed from the invert of the cavern top heading to further reinforce the rock separating the two operations to prevent any rock fall out (Figure 5). Contractual details, which necessitated this coordination, are provided later.

Some of the other on-going coordination efforts with adjacent contractors, mostly related to access, air quality in the construction areas, ventilation, and safety, that the PB/CCM team deals with on a day-to-day basis, is the blasting being conducted to the north for construction of the 86th Street Station (C26008), the rehabilitation of the existing 63 Street Station immediately to the south west (C26006), which is being upgraded to accept the 2nd Avenue subway trains, and the systems contract (C26009) where all the appurtenances required to run the trains will be installed. Also, part of the way through construction under this contract, the northern third of the 72nd Street station cavern and the Ancillary 2 shaft is to be turned over to the station finishes contractor (C26011) and shared access is to be maintained for a period of approximately 6 months.

In terms of noise and dust mitigation, especially for muck hauling and material delivery, the contractor devised a plan to build an insulated enclosure around both access shafts, fitted with an electric gantry crane, to allow the storage of muck and truck loading within the enclosures. More details are provided later. The PB/CCM team supported the MTACC in securing concurrence from the surrounding community to build these structures, although they were not part of the original construction plans that had been communicated to the public. These structures were very effective in mitigating noise and dust during muck dumping and, in addition, allowed for a controlled release of blast fumes, as stated below.

One of the greatest challenges to prosecuting the work for the station excavation was the mitigation of community concerns related to blasting. Concerns were related to blasting continuing till 10:00 PM, the effect of blast fumes on air quality, and a perception of damage due to vibrations. The MTACC restricted the night blasting operations to 7:00 PM to mitigate the late night blasting concerns, and the muck enclosures were modified to contain the blast fumes and wet down any airborne dust while allowing a controlled release. The PB/CCM conducted a detailed study of the air quality, not only during blasting but also during the general construction operations, to show that the construction operations did not adversely affect the air quality. With respect to blast vibrations, there were strict limitations on allowable blast vibration that were followed with mitigation involving adjustment of round lengths, and overall volume of blasts. Control of blast vibration was a major coordination issue due to the presence of various historical, fragile, sensitive or landmark buildings along the corridor, each with different vibration criteria. Adherence to vibration criteria in the vicinity of such buildings required coordination with the buildings and the excavation sequence required based on the various excavation shapes and sizes (Figure 2) and the overall construction schedule. Finally, where an affected building contains sensitive equipment and/or hospitals where sensitive surgical procedures are carried out (e.g., Eye and Ear Hospital on 64th Street), protocols were established with these entities where prior notification was provided regarding the time of the blast. In other cases, especially with buildings adjacent to the ancillary/entrance structures, the blast vibration and other geotechnical instrumentation data were regularly forwarded to the buildings for their information. In addition, if and when, vibration exceedances were experienced, a team consisting of PB/CCM members and the contractor conducted an inspection of the building.

Given the various conditions of buildings along the construction corridor, the design team, in conjunction with the Department of Buildings and the PB/CCM, identified those structures that could be potentially vulnerable to blasting operations and the MTACC had a $1 million allowance in the contract to conduct building remediation to achieve "blast readiness." Such remediation was generally necessary in older masonry buildings within the blast influence zones and required shoring and reconstruction in basements. Coordination was required with the buildings to determine the specific scope and implementation of the remediation and the allowance was administered through a process similar to change order negotiations. A particularly unique situation arose in connection with the building demolition required for the construction of Ancillary 2 and Entrance 3. In the former case, building demolition revealed that the wall of the building immediately to the west was not tied back to itself and was potentially unstable without any work. The PB/CCM team, working with the design team, coordinated the development and implementation of a technical plan to support this wall with tie-backs (Figure 6) with the building owners and the Department of Buildings. In the latter case, the problem was similar, but restricted to a chimney that was shared by the building to be demolished and the building located immediately to the south (Figure 7). In case of Entrance 1, located at the northeast corner of 69th Street and within the high-rise building located here, the PB/CCM is supporting MTACC efforts to obtain access agreements with the building to relocate their utilities and underpin a wall so that an escalator entrance can be built.

Figure 6. Tie-backs at Ancillary 2

Figure 7. Tie-backs at Entrance 3

As may be evident in the foregoing, maintaining a significant public outreach program is essential for the successful and timely completion of a project of this magnitude. The MTACC, with support from the PB/CCM team has taken a proactive approach in informing the local officials and the affected residents and businesses regarding the various construction operations and sequencing, their durations, and addressing their concerns. The PB/CCM has a full-time community liaison person whose primary duties involve addressing the community concerns and coordinating with the MTACC/CCM/contractor to resolve the issues in a timely manner. An after-hours hotline has also been established where complaints can be received during non-working hours and can be addressed as received. In addition, regular meetings are held with the local Community Board (CB8), where the community is briefed on the overall progress of the work on the entire project. On a project specific basis, monthly "community advisory board" meetings are held with the local community members, businesses and residences, to provide more specific details on the progress of the contract. Subjects range from repair of irregularities on the sidewalks, requests for additional lighting in certain areas, to the layout of the work zones and MTACC plans for restoration of the area. These meetings are also attended by members of various City Agencies, including DOT, NYPD Traffic Department, the Mayor's office and elected officials. Written progress notes are also made available to the local residents through a monthly newsletter available on MTACC's website. Finally, quarterly "Public Workshops" are held on a project-wide basis, where the goal is to maintain an open two-way dialogue between the MTACC and community at large. Another factor that has helped in allaying some of the concerns is scheduling regular underground tours which allow the local people to personally view the enormity of the operations occurring right beneath their feet, the goal being to provide insight as to the impacts realized by the public at the surface. For the buildings that immediately adjacent to the ancillary shafts, monthly meetings are held with the building owners to address their specific concerns.

A "good neighbor initiative" has also been instituted where the construction work zones have been "dressed up" with "Street-Retail" fence wrapping depicting artwork designed by MTACC's graphic artist which contains signage for businesses along the work zones, wayfinding signs at street intersections, (Figure 8), pedestrian separators on crosswalks to better delineate safe passage across the streets, and a community bulletin board where project information is posted.

CONTRACTOR'S PERSPECTIVE

SSK Constructors Joint Venture was the selected low-bid contractor for Contract No. 26007. The Joint Venture is comprised of Schiavone, Kiewit and J.F. Shea construction companies, of which Schiavone is the sponsoring partner. The contract award and notice to proceed date was October 1st, 2010, with an overall duration of 37 months.

Figure 8. Street-retail fence wrap and wayfinding signs

There was an interim milestone (Milestone #1) of 30 months, which required the contractor to turn over the northern third of the Main Station Cavern for use by the following station systems and finishes contractor. The overall value of the contract was $447M, including a $16M option to excavate the Ancillary #1 site at the corner of 69th St and 2nd Avenue. The option was available until Notice of Award (NOA) plus 12 months. The MTA did exercise and award the $16M option to SSK.

Phased access was an important and pervasive consideration for this Contract. The MTACC was under high schedule pressure for the overall program to meet a revenue date (trains in service) of December 2016. In order to manage the project completion goal, all contractors were provided compressed time frames and expected to share and overlap operations at their work sites. These efforts were explicitly stated in various coordination clauses in the contract specifications. Successful bidders would have to carefully contemplate prosecuting the work with the full awareness that other contracts were ongoing nearby; i.e., adjacent and in this case below. The anticipated award date for Contract C26007 was made prior to the MTACC achieving full title for property acquisitions and arranging easements, both temporary and permanent, for various parts of the Project site. Two buildings at Ancillary 1 and 2 could only be accessed at NOA+3 months. Access to Entrance 1 was to be granted at NOA + 9 months. Access for the building at Entrance 3 was as late as NOA+15 months.

Besides the access requirements for the surface work, the underground portion of the Project had its own set of access and sequencing constraints. The scope of the first contract, C26002, was to perform two TBM drives the length of the Phase 1 alignment. When C26007 was awarded, SSK would have to start excavation activities with the understanding that the Contract C26002 TBM runs would be in progress. The MTA specified two non-blasting periods within the first 18 months, each coinciding with the expected arrival and departure of the TBM from the 72nd St Station work zone. To the best of their ability, the MTA specified three months NOA + 5 thru 7 for the passing of the west TBM drive and four months NOA +15 thru 18 for the passing of the slightly longer east TBM drive. This requirement presented SSK with a number of unique challenges. The first challenge was excavating above an active TBM requiring tight geometry control to maintain the structural integrity of the rock mass separation. In some instances, this separation was as small as 8 feet. The second challenge was that much of the rock mass to be removed was not available to SSK until after the second TBM drive had left the 72nd Street Station work zone. SSK projected rock spoil yields over the excavation schedule and found that a very high yield was required in a very short period to achieve the overall Project duration on 37 months. Ultimately, this recognition resulted in the need for a high speed rock spoil (muck) conveyance system. A conventional crane and bucket method could not meet the rigorous schedule demand; much less service all the other hoisting demands of an underground mining operation. Only a

Figure 9. Muck enclosure at the 72nd Street shaft

hoisting system with high line speed, and large load capacity, could remove the muck in a high enough volume and still service material, equipment handling needs. The only accesses to the underground portions of the Project were two 28 foot diameter temporary shafts. Thus, SSK designed and constructed two identical muck handling buildings over each temporary shaft (Figure 9). The buildings were pile supported/steel frame structures fully cladded with insulated panels that mitigated noise, dust and an overall visual impact to the highly residential neighborhood. The approximate dimensions were 160 feet long by 45 feet high by 32 feet wide. The system comprised of a gantry hoist and twelve dump stations oriented in two parallel rows of six, accommodated 25-yard muck buckets that hydraulically dumped into a waiting tri-axle trucks below them. Each muck building was designed to handle up to approximately 600 loose cubic yards per shift. With some normal operating inefficiency expected; actual daily yields were satisfactory to meet the schedule goal of the Project.

Additional challenges were related to the specific location of the Project on the upper east side of Manhattan with its mixed use high end residential neighborhood, was an unlikely setting for the alien nature of a massive infrastructure project.

This coupled with a very small surface footprint for staging, service and support operations tested the skill and savvy of SSK's engineering, planning and ultimately NYC experienced field crews. Staging and laydown areas were mainly relegated to the streets and narrowed sidewalks, while four lanes of through traffic was required to be maintained on 2nd Avenue during peak periods. The following aspects of the Project are the recipe for the daily juggling act required to prosecute the work in this dense urban environment: (1) removal of over 375,000 loose cubic yards of rock (Figure 10), resulting in over 18,000 haul truck trips over 18 months; (2) placement of over 8 million pounds of rebar; (3) placement of over 10,000 yards of initial rock support by shotcrete method (Figure 11); (4) 60,000 yards of cast in place concrete, (5) dozens of pieces of heavy mining equipment; (6) eight discrete custom forming systems with countless conventional forms. Truck movements were of particular challenge because of the high pedestrian traffic, thus necessitating dedicated flaggers and handlers. A particularly successful innovation came through the combined efforts of SSK/MTACC and NYCDOT to create a truck staging area just north of the construction zone. By taking a parking lane during specific work hours, SSK dedicated dispatchers were able to regulate the arrival of haul trucks, concrete trucks and other major deliveries as their need arose without unnecessarily congesting 2nd Avenue, a vital vehicular artery of the Upper East Side.

Additional challenges included the contractually mandated work restrictions. Restrictions on surface operations forced trucking of muck and delivery of concrete to

Figure 10. Main station cavern—waterproofing and concrete lining work

Figure 11. Shotcrete initial support

be performed during peak traffic periods. The contractor's preferred night time hauling period was off limits. Despite the fact that blasting vibration levels are difficult to keep below threshold levels in an urban environment, SSK was able to mitigate blasting effects and still maintain a good degree of production with close coordination between the Designer, Owner and their geotechnical monitoring subcontractor GeoComp.

CONCLUSIONS

An overall flavor of the complexities of building a subway station in the upper east side of Manhattan has been presented. It is shown that a fair and reasonable approach is essential from the perspective of technical expertise, effective communication and rational decision making on the part of the owner, designer, construction manager, and the contractor, working as a team, for the successful completion of the Second Avenue Subway Project. Just as important is a partnering approach between all stakeholders, including elected officials, the community residents and business owners, the traveling public along the alignment, as well as the multiple interfacing construction contractors.

ACKNOWLEDGMENTS

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PRECAST CONCRETE DESIGN CHALLENGES IN THE UNDERGROUND LIRR GRAND CENTRAL TERMINAL MAIN STATION CAVERN OF THE EAST SIDE ACCESS PROJECT

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ABSTRACT

The MTA Capital Construction's East Side Access Project (ESA) is the largest underground rail project in New York City. A new state of the art terminal located beneath Grand Central Terminal (GCT) will provide a new route for the Long Island Railroad (LIRR) to connect to the East side of Manhattan. This paper presents the challenges encountered and solutions developed in designing an underground structure located in bedrock 150 feet below ground. The Caverns have been designed as a unique and iconic underground structure built of precast concrete ranging from the cavern final lining and main structural system to the platforms and overhead smoke exhaust plenum. The design utilizes accelerated construction techniques including prefabrication, modular layout of the structural components, standardized connections, integrated architectural and structural elements, and built-in provisions for erection and fabrication tolerances. The design required collaborative coordination of the station's fit-out design with architectural, mechanical, and electrical systems. Additionally, the deep mined underground project site presents unique challenges, primary of which are restrictive access and confined space. As a result, the design and erection of the precast structure was influenced by the construction logistics due to the constraints on weight and size of the precast components.

INTRODUCTION

East Side Access Project

The LIRR currently transports commuters from Long Island into Manhattan, terminating at the already over congested Penn Station on the west side of Manhattan. Once completed, the ESA Project will provide LIRR commuters direct access to the east side of Manhattan underneath Grand Central Terminal. The ESA Project will help alleviate the congestion at Penn Station, which currently accommodates the NJ Transit, Amtrak, and LIRR lines; reduce travel time for LIRR passengers traveling to the east side of Manhattan and facilitate connections to the NYCT Subway System and Metro North Rail Road (Figure 1).

The ESA Project includes mining and lining of new tunnels and facilities under Manhattan and Queens. The tunnels run from Queen's Sunny Side Yard through the existing 63rd St. Tunnel, underneath Manhattan's Park Avenue until termination at 37th Street. Differing types of tunnel methods and final lining systems are being used dependent on the ground conditions, geometry and size of the excavation, site

Figure 1. Overall site plan

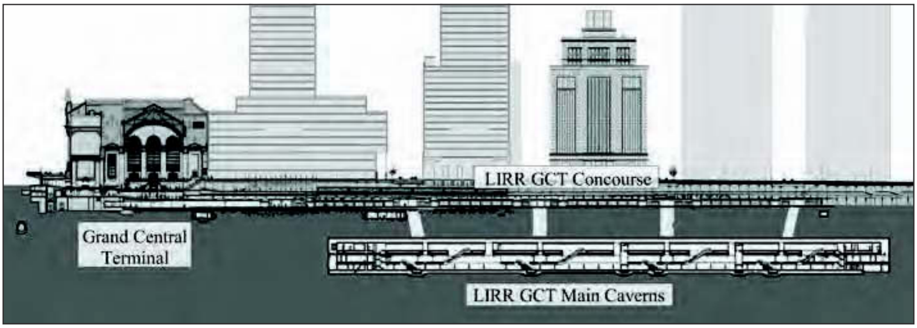

Figure 2. Longitudinal Section—the GCT main station caverns below Grand Central Terminal

constraints and functional requirements. Mining of the Queens segment of the ESA Project involves soft ground tunneling by pressurized face tunnel boring machine, cut and cover construction, and conventional tunneling. The Manhattan segment of the ESA Project involves mining construction in hard rock by tunnel boring machines, drill and blast, and road header. In Manhattan, cast-in-place concrete linings are being used mainly for the running tunnels and the multi-level ventilation and substation facilities; shotcrete final lining is used for the interlocking single level caverns and adits.

This paper focuses on the planning and design of precast concrete lining and precast interior structural system for the LIRR GCT Main Station Caverns, and the opportunity and challenges of implementing this system into a modern station 150' below Manhattan in hard rock (Figures 2 and 3).

The GCT Main Station Caverns

The new LIRR GCT Main Station Caverns will provide commuters a new terminal station which involves the construction of two parallel caverns approximately 59 feet wide, 66 feet high and 1142 feet long in hard rock located under Park Avenue and the active Metro North Railroad. Each of the LIRR GCT Main Station Caverns consist of an upper

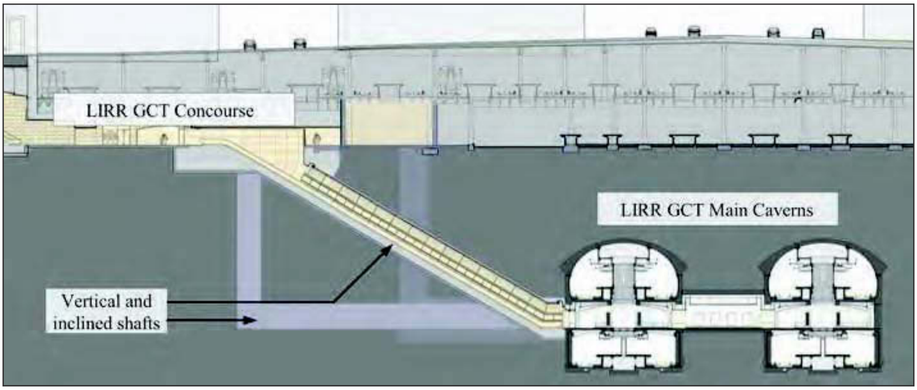

Figure 3. Cross section—the GCT main station caverns below Grand Central Terminal

and lower level platform with an intermediate mezzanine level and an overhead smoke exhaust plenum. At the mezzanine level, the two caverns are interconnected by four cross passages in the public area and two service cross passages at each end of the station where the back of house rooms are mainly located. The lower and upper level platforms connect to the mezzanine level by vertical circulation elements which include stairs, escalators and elevators. From the mezzanine, passengers are connected by high rise escalators to new LIRR Concourse located in the lower level of GCT. Exit to the street can be made through existing and new entrances at GCT located between 42nd Street to 48th Street.

The design and contract packaging of the GCT Main Station Caverns changed over time during the course of the ESA Project. Originally, the excavation and initial rock support, structural work and fit-out were planned to be constructed in three separate contracts. The structural final lining and main structural system consisted of cast-in-place construction. The subsequent fit-out contract included the architectural finishes, mechanical systems (HVAC, fire protection and plumbing and track drainage), electrical systems (lighting, power, grounding, and conduits for communication, fire alarm, security systems, signal, signal power, and traction power), cast-in-place platforms and an overhead smoke exhaust. In an effort to achieve schedule and quality benefits, a study was conducted to evaluate the feasibility of a precast concrete alternative for the construction of the GCT Main Station Caverns final lining and main structural system. The feasibility study covered the following aspects: types of precast concrete systems with bolted connections and cast-in-place concrete joints; types and shapes of precast members; tolerances and finishes; delivery and transportation of precast concrete members through the ESA Project alignment, erection of precast members in an underground environment, and an industry out-reach to potential precasters. This led to change the design of the cavern's final lining and interior structural system from a traditional cast-in-place concrete to a precast concrete alternative (Figure 4).

In addition to modifying the structural design of GCT Main Station Caverns, the structural work and fit-out of the GCT Station Cavern were combined into one contract. Combining the station cavern's architectural, structural, mechanical and electrical work allowed for the development of an enhanced integrated design approach.

The LIRR GCT Main Station Caverns are divided into two functional areas: public and the back of house areas (Figure 5). The back of house areas located at the north and south ends of the cavern contain electrical and mechanical rooms, tunnel ventilation and train operations. The vast majority of the cavern is dedicated to public areas for platforms and circulation between the different levels of the cavern. The structural

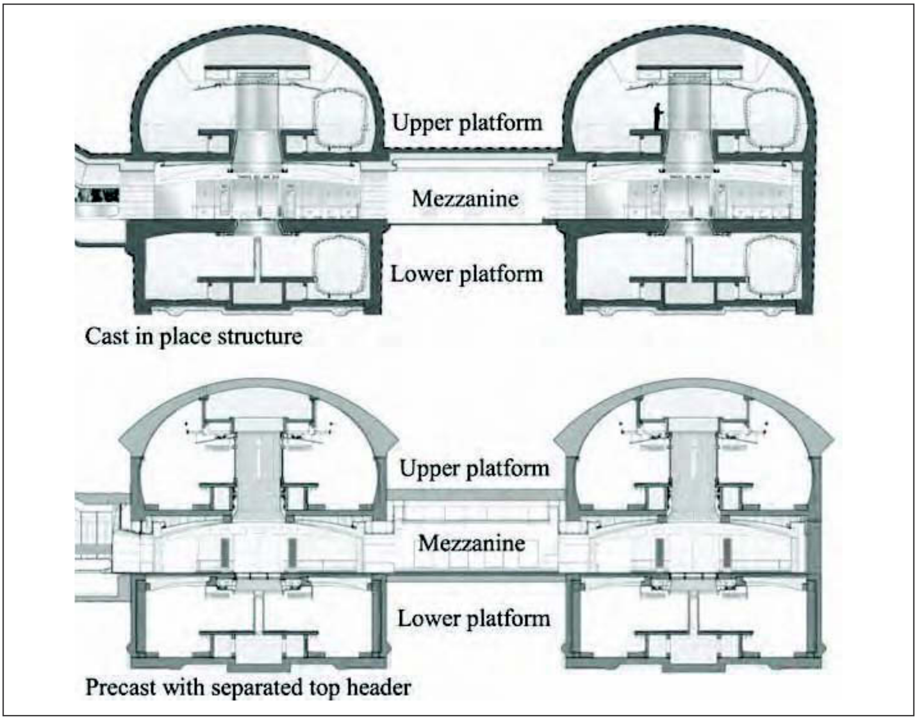

Figure 4. Main Station Caverns cross section with the cast in place design and precast alternative design

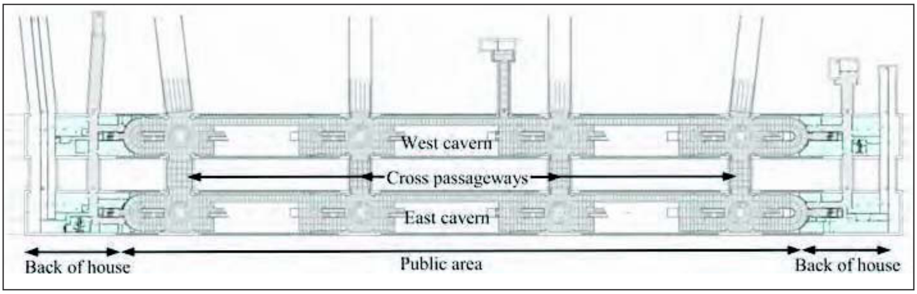

Figure 5. Mezzanine plan overall organization

framing system of the back of house areas consists of traditionally reinforced concrete construction composed of flat two-way slabs supported on rectangular columns. The structural system in the public area consists of precast concrete beams and deck panels The final lining is composed of precast wall panels and self-consolidating concrete which is to be constructed over a coordinated waterproofing system.

The configuration and layout of the caverns in the public area has been designed to repeat around the areas where the two caverns are interconnected by cross passages. This organization allowed for the structural subdivision of the caverns into three repeating modules: the Node Area, the Stair and Escalator Area, and the Elevator and Bridge Area (Figure 6).

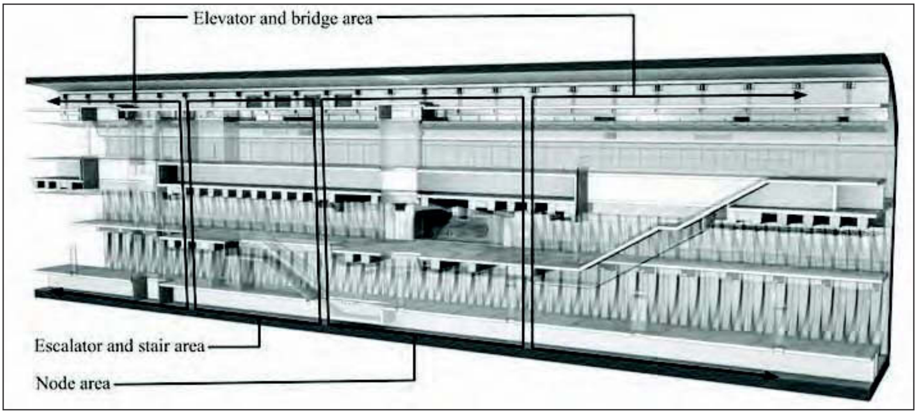

Figure 6. Structural modules—typical repetitive precast areas of the GCT main caverns

To take full advantage of an integrated prefabricated system design and with the vision of a linear construction process, a modular and fully coordinated layout was developed by realigning and integrating the structural, mechanical and electrical components, and architectural finishes at each horizontal level (invert slab, lower and upper platforms, mezzanine level and overhead smoke exhaust) of the station's cavern.

The advantage of a modular layout and precast concrete construction allowed the cavern structural system and final lining to be engineered as repeating precast concrete elements with typical connections. These elements include final liner wall panels, upper level and mezzanine level beams, mezzanine level corbels, upper and mezzanine level deck panels, platform wall and deck panels, smoke exhaust hollow core planks and wall panels, and node groin ribs. Through collaboration between the structural engineers and architects, it was determined that publicly viewable precast units would have a precast concrete commercial architectural finish taking into consideration the distinctive shape, color and texture of the precast members (Figure 7).

BENEFITS OF A PREFABRICATED AND INTEGRATED DESIGN APPROACH

Benefits of a Precast Concrete Design

The concept of prefabricated design to achieve accelerated construction has been in use in the construction industry for a great many years with major success in the building and bridge industries. Although it has not been used to any great extent in underground construction of transit stations where there is limited access and work is performed in confined spaces, the benefits of prefabricated design can be achieved by applying the proven techniques of above ground construction with some adjustments.

Advantages of prefabricated construction can reduce onsite construction time compared to conventional construction practices. Benefits applicable to the construction of the GCT Main Station Caverns include:

- Manufacturing of the prefabricated components can start before site access is granted, allowing the prefabrication process to begin before excavation of the caverns is completed.
- Prefabrication allows for earlier access for follow-on contracts.
- Prefabrication can reduce schedule risks through planned assembly techniques and assembly trials prior to construction

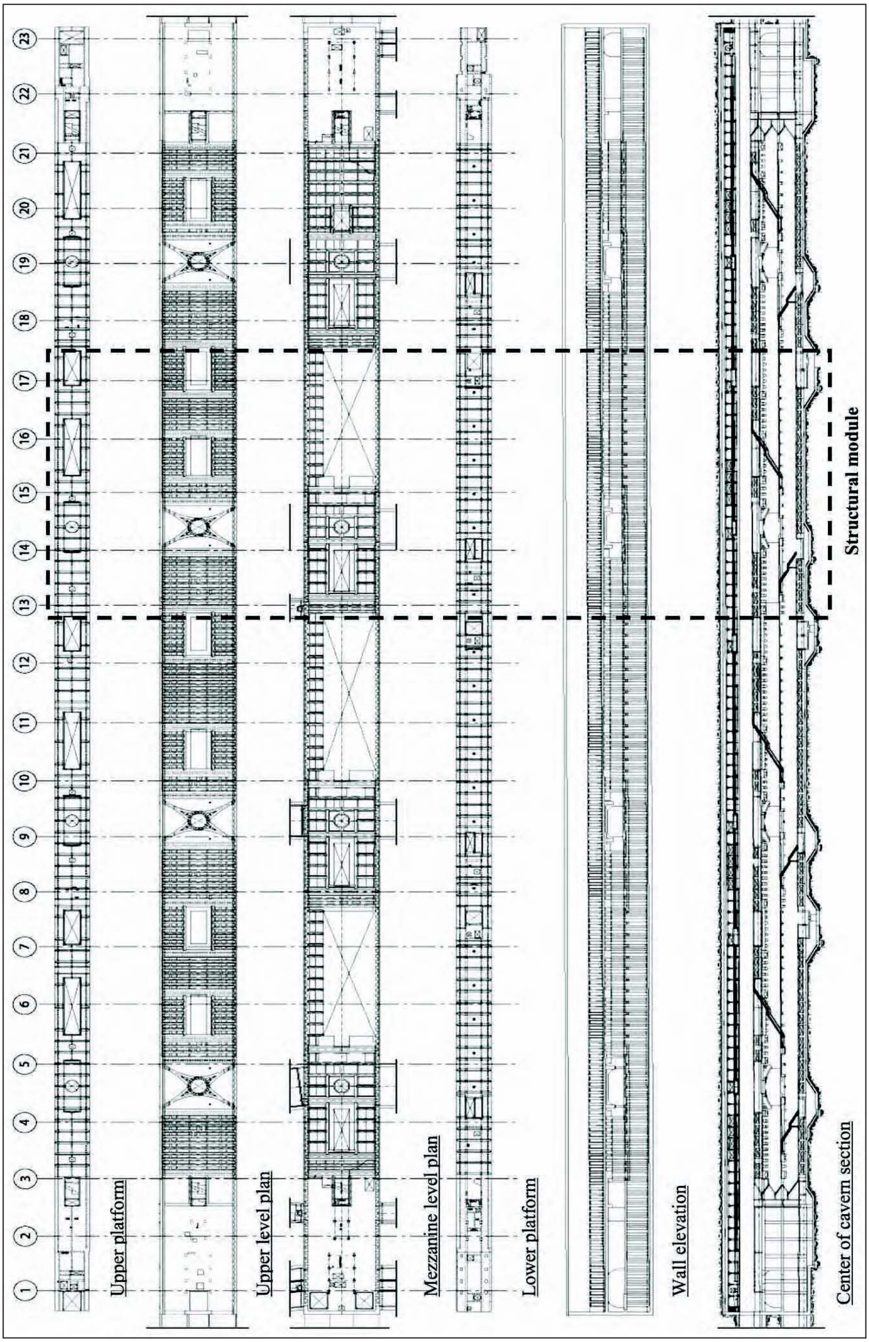

Figure 7. Typical precast layout of the GCT main caverns

- Construction Sequence—the cavern structure has been designed so that structural precast elements can be erected prior to and independent of cast-in-place concrete.
- Limited Job Site Operations—limits construction activities such as delivery and installation of rebar and formwork, stripping of formwork, and repairs typically required to surfaces damaged during stripping.
- Construction Schedule—decreases time that would be required for curing of cast-in-place concrete.
- Repetitive Shapes—repetitive element shapes reduces the number of molds and decreases the overall production cost.
- Quality of Finished Product—elements will be fabricated in a plant controlled environment with standardized production process.
- Durability—higher quality of materials and workmanship increase the life span of the structure.
- Architectural Finishes—wide variety of finishes, shapes and colors can be incorporated into the structural elements.
- Preplanned connections that in most cases eliminate major formwork for any cast-in-place closure pours.

The concept of prefabricated design and integrated design approach was extended to the architectural finishes and associated mechanical and electrical systems work in the GCT Main Station Caverns. As a result of this effort, most architectural systems were redesigned to provide minimal onsite installation. The Mezzanine perimeter walls were redesigned as shop made assemblies of steel frames over precast bases faced with demountable stone panels on a 5' and 10' module. This allowed for greater dimensional control, and easy access for maintenance and inspection. The Mezzanine glass wall assemblies were redesigned with fewer components to resemble preassembled curtain walls. Floor patterns and platform hatches were aligned with structural panels to minimize unnecessary panel variation and to control cracking of finishes. Ceiling systems and connection details were adjusted to the new precast design. All vertical elements such as smoke exhaust and elevator and escalator enclosures were modified to be modular with typical connections to the structure.

Mechanical and electrical designs were also affected. Pipes, ducts and conduits were rerouted to minimize precast penetrations through walls and floors. Details were redesigned to suit precast concrete construction. The location of the smoke exhaust dampers and HVAC spot cooling units were revised to fall concentric with the precast panels.

In addition precast construction made the installation of under platform mechanical and electrical services less restrictive by permitting these services to be installed in advance of completing the platform structure.

Construction Logistics and Advanced Planning

To suit the design and construction of the GCT Main Station Caverns towards a precast alternative design, advanced planning during the preceding excavation contracts of the ESA Project was required. The top heading of the station cavern was redesigned from a circular arch to an optimized parabolic shape to be supported on rock ledges (Figure 8). This permitted that the arch final lining be constructed in advance of the cavern's final lining walls and interior structure (Figures 9 and 10). The separated reinforced concrete arch was designed for full rock load, hydrostatic load, and the long-term lateral loads that will be imposed in the cavern's final lining walls. The arch design also included provisions for continuity of reinforcement and waterproofing at the interface

Figure 8. Separated arch header excavation

Figure 9. Cast segment of the separated arch final lining

Figure 10. Benching down to full cavern depth

between the arch's concrete pour (part of the excavation contract) and future cavern walls (part of the built-out and fit-out contract). The excavation below the initial top heading was performed after the construction of the separated arch. By not fully excavating the cavern to the invert level, this facilitated the concrete operation and waterproofing installation for the concrete arch.

With the introduction of a separated reinforced concrete arch, it was determined that an overhead gantry crane may assist in the erection of the precast concrete members. The design of the separated arch was revised to support a working load of 25 tons to account for the weight of the precast concrete units. Furthermore, the support structure for the overhead gantry crane was designed to be part of the permanent structure of the overhead exhaust of the cavern.

ACCESS RESTRAINTS

Several access routes were investigated for the delivery of the precast units and materials to the GCT Main Station Caverns. One primary access point was established via the Queens Bellmouth Structure adjacent to Sunnyside Yard.

The Queens Bellmouth structure allows for the delivery of the larger precast concrete units via the existing 63rd Street Tunnel and the new Manhattan tunnels. The geometry of the existing 63rd St. Tunnel, the size of the new circular running tunnels, and the curvature of the Manhattan alignment leading to the GCT Main Station Caverns created constraints for the delivery of the precast concrete units. Clearance envelopes were created to account for the possible transportation equipment, size and shape of precast concrete units and existing conditions. Via this route, the precast concrete units have the flexibility to be delivered on a flatbed, either by temporary rails, or rubber tired truck (Figure 11).

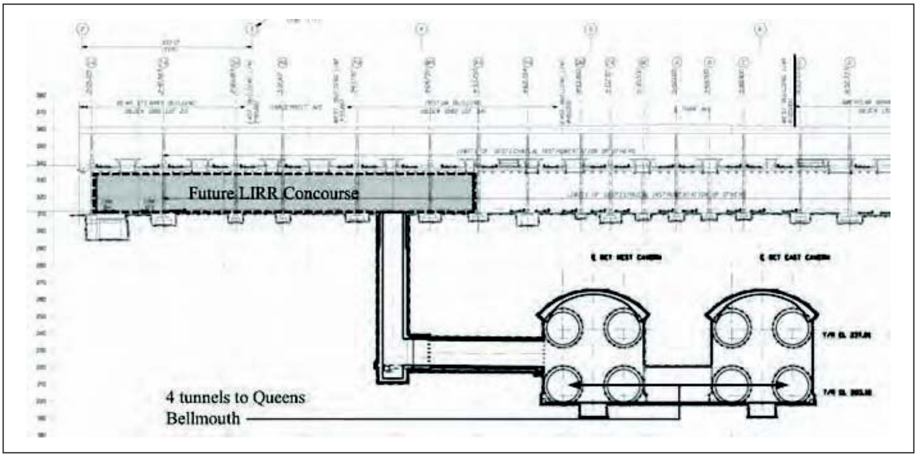

Figure 11. Material access—Manhattan alignment delivery

CHALLENGES OF CONVERTING CAST-IN-PLACE TO PRECAST

As with any precast concrete construction project, one of the major challenges in this project was the development of connection details best suited to the individual components, ease of construction and proper fit-up for an economical design. In this project, there were additional challenges including weight restrictions, maximizing repetitive components, geometric constraints and coordination with other disciplines for an integrated design.

Before the redesign to precast structure, the cast-in-place option had been developed to a 60% level of design. Converting it to a precast design often required reverting back to a conceptual level of design of some of the major components.

Geometric constraints were a driving factor in a number of the precast components. The geometry of the cavern was already established based on a cast-in-place concept. The intent of the design for the precast concrete alternative was to keep the same concrete lines and replicate the cast-in-place concept. Architectural geometric constraints required that the geometry of the precast elements not encroach beyond what was already established and not impact the architectural features and mechanical space which were highly compressed and with little clearance. The proposed delivery route also imposed constraints on the size shape and length of the precast components due to physical limitations of existing conditions of the tunnels through which the component would be delivered. Shape, size and layout of the precast concrete components were designed to optimize the fabrication process while satisfying the transportation and erection constraints, and performance requirements.

A major design parameter was a restriction on the overall weight of the components. The proposed gantry system supported from the overhead smoke exhaust plenum limited the component weight to 25 tons. This required creative solutions at the interfaces between component and development of specialized construction joints. The depth of the support beams at the Mezzanine and Upper Level beams were minimized in order to meet the weight restriction, which created an element with reduced section modulus.

To obtain the strength required, the beams were designed to act compositely with the precast deck panels and a concrete closure pour was used to connect the beams and panels together to achieve a monolithic design.

To take advantage of the economical benefits a precast design offers, the various components were designed using repetitive patterns. An effort was made to keep the

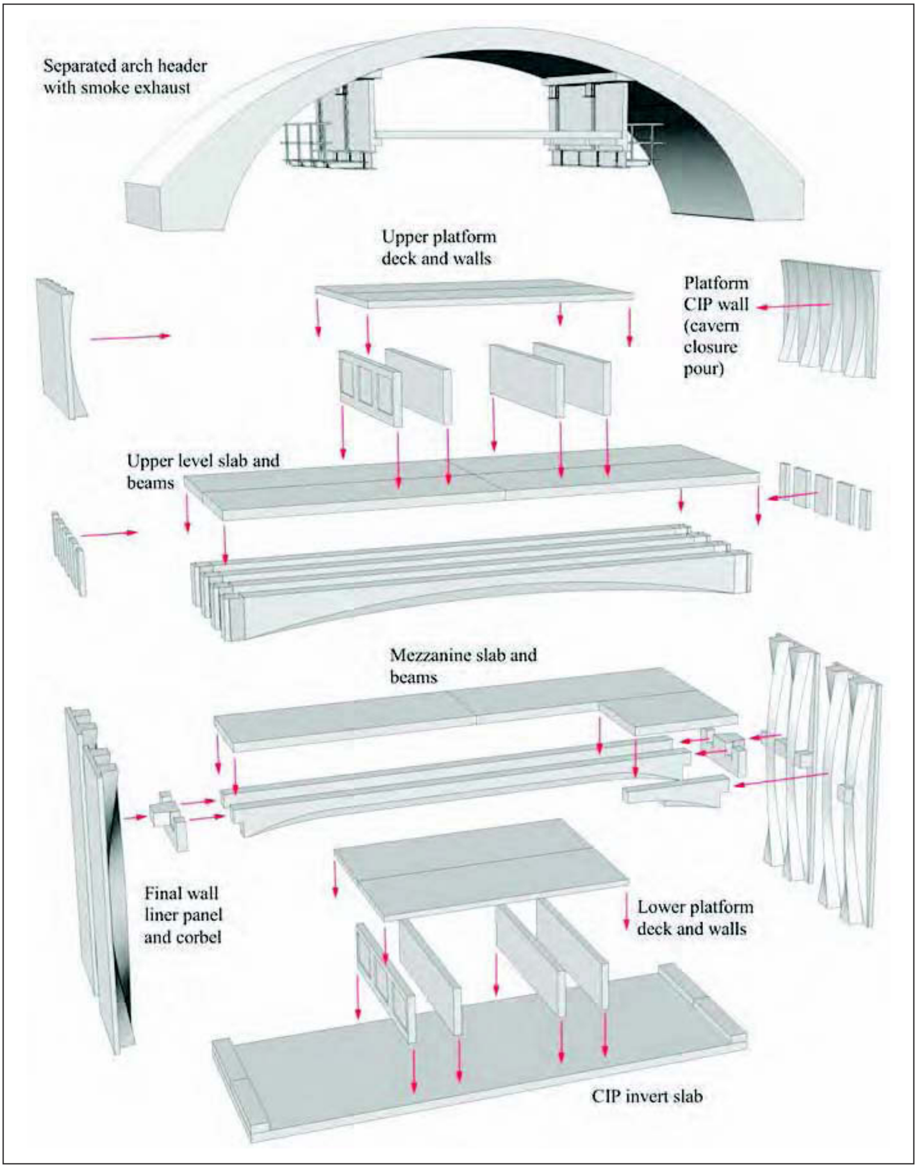

Figure 12. Typical precast units in the GCT main caverns

shape of the components simple for construction purposes yet sophisticated enough to meet the challenges of accelerated construction. The components were designed to be interchangeable where possible with standardized shapes, making as minimal number of different components as possible that often required minor rearrangements of beams, walls, and deck panels.

Connection of precast components where the alignment and location of embedded items is critical to the proper fit-up and connection of the components required that the embedded items to be set out using matching templates.

Connections and Pre-Embedded Hardware

Hardware used in the connection of the precast units as well as attachment of architectural finishes and other trade system components is a big part of the construction of the caverns. It was recognized early on in the design phase that there would be a large difference in the type and use of the hardware. Not only is there a difference in the various types of anchorage systems; the devices may be either embedded in the precast or post-installed, used in the permanent structure or for temporary construction, and pre-engineered or provided by the contractor. An effort was made to distinguish between the different uses of the anchorage devices and identify which items the contractor would be responsible for the design of. The precast hardware was categorized as follows (see also Figure 12).

Construction Hardware: Items to be placed on or in the structure in order to receive the precast concrete units in the field; e.g., anchor bolts, angles, or plates with suitable anchors. In addition, hardware to be embedded in the concrete precast units themselves, for connections to the structure or receiving other precast concrete units.

Plant Hardware: Hardware shown on the Contract Drawings or the Contractor's Assembly and Production Drawings to be embedded in the precast units themselves for other trades such as mechanical, plumbing, electrical, glazing, miscellaneous iron, masonry, etc. Final design and location of Plant Hardware shall be the responsibility of the Contractor.

Erection Hardware: Hardware designed by the Contractor for the fabrication, handling, transportation, and installation of the precast concrete units.

And the following for post-installed anchors.

Post-Installed Anchors: Hardware installed in the hardened cast-in-place concrete and in precast concrete units installed in their final location for other trades such as mechanical, plumbing, electrical, glazing, miscellaneous iron, masonry, etc. Final design and location of Post-Installed Anchors shall be the responsibility of Contractor.

Various types of devices were used in the pre-engineered connections of the precast units to provide for the intended behavior of the original design. The devices were selected to suit the design of the members and transfer of forces. The devices included cast-in-place anchors, cementitious grouted anchors, post-installed (drilled-in) mechanical anchors, post-installed (drilled-in) adhesive anchors, embedded channels, splice sleeves, headed rebar, and mechanical couplers.

Construction Tolerances

Due to the nature of precast construction and inherent tolerances in fabricating and installing the units, extra care was used in the setting out of the units to avoid an accumulation of tolerances. The cavern is 1,142 feet long and any accumulation of tolerances would cause misalignment of the units as construction progressed along the length of the cavern. A grid system was established based on a modular spacing of 5 feet and 10 feet with geometric control on true vertical and horizontal dimensioning. The precast units are to be fabricated and installed from the centerline of the grid lines. It was imperative that the units not be installed from the adjacent unit to prevent the unit from creeping into the location of the next adjacent panel. Construction joints between the wall panels and platform panels were made slightly larger than the tolerance for fabricating the panels to ensure that the panels did not encroach beyond the grid lines and prohibit the installation of the next panel. Closure pours were used between deck panels located at the mezzanine and upper level beams. The closure pours make up for any tolerances in the fabrication of the beams and deck panels.

The vertical alignment of the caverns followed the slope of the tracks, which is on a slope of 0.3%. Although the slope is considered very slight, it created a difference in elevation of ⅜ inch over 10 feet. The wall panels and support framing were installed to true vertical whereas some of the structural elements, i.e., deck slabs and platforms, along with the architectural finishes were installed parallel with the track. To achieve full benefit of the cost savings from the use of repetitive members, the wall panels were fabricated rectilinear and the framing beams set level on the wall panels. The ⅜" difference over 10 feet was accounted for in setting out the adjacent panel. Any unevenness in the layout of the deck panels to be obscured by the track slab, platform wall panels, or mezzanine topping slab.

Additional Modifications to Structural Specifications

One of the major benefits of specifying a prefabricated system is the increased quality that can be achieved through fabrication of the components in a plant controlled environment. However, to ensure a high quality product a number of provisions were inserted in the specifications.

Prototypes: Prototypes of the major prefabricated components were required to be constructed to represent the aesthetic effects and quality standard to which the production units were to be judged to. The prototypes were to be constructed full size with the same materials and the same construction procedures to be used for the production units. The units were to demonstrate the architectural features with the actual size and shape of the units and displaying the expected range of finish, color and texture variations allowed. The prototypes were to be maintained throughout construction as a reference for the production units. In order for the contractor to demonstrate proper repair procedures and hone his skills, the prototypes were to be purposefully damaged and then repaired.

Full Size Mock-Up: To simulate the erection and fit-up of the precast components, the specifications require the contractor to construct a mock-up of a section of the cavern using the construction techniques to install and connect the components, replicating the anticipated constraints in the existing caverns. The mock-up was to be constructed off-site and represent a 60-foot section of the cavern constructed with full size components including invert slab, wall panels, corbels, and upper and mezzanine level beams and deck panels.

Smoke Exhaust Plenum: The initial section of the smoke exhaust plenum was to be constructed as the first trial section of the permanent structure to demonstrate the erection procedure for the plenum and overhead crane to be used in the construction of the cavern.

Plant Inspection: The contractor is required to maintain full time inspection personnel at the casting plant responsible for ensuring specific quality is being achieved and proper procedures are implemented at all stages of the production process including the manufacture, handling, storage and protection of the precast units. The construction manager will verify the finish product and inspection process to guarantee quality control.

Reference Samples: An aspect of the architectural design concept is an open and brightly lit station. To that end, the concrete to be used for the exposed surfaces consists of white cement with buff limestone aggregate similar in color and texture to Sample No. 109 of the PCI Color and Texture Guide. During design, a trial mix was developed and tested to simulate the use of local materials. In order to control the color and texture of the concrete used in the final product, the contractor was required to submit samples of the concrete composed of the material he intended to use and demonstrating the standard of appearance, surface detail, color and texture to be achieved. Once approved, the sample was to be used as control sample to compare the production units to.

Erector and Fabricator Qualifications: Specific requirements were specified for the erector and fabricator including a specific number of years of experience and certification by the Precast/Prestressed Concrete Institute.

Delivery, Storage and Handling: The contractor was required to submit erection sequencing, handling and erection procedure along with calculation and reports on lifting inserts and devices to insure that the forces and distortions during lifting operations did not overstress or damage the precast units.

Repairs: The specifications require special requirements on the repair of the precast units. All repairs are to be performed in the presence of the Construction Manager. The Construction Manager has the sole right to approve or reject the repair of damaged or defective precast concrete units. The patching materials to be used shall be made with the same material of the concrete used in the production unit and the repairs shall not show any apparent line of demarcation between the original and repaired work (see Figures 13 and 14).

Figure 13. Architectural finishes in the GCT main caverns

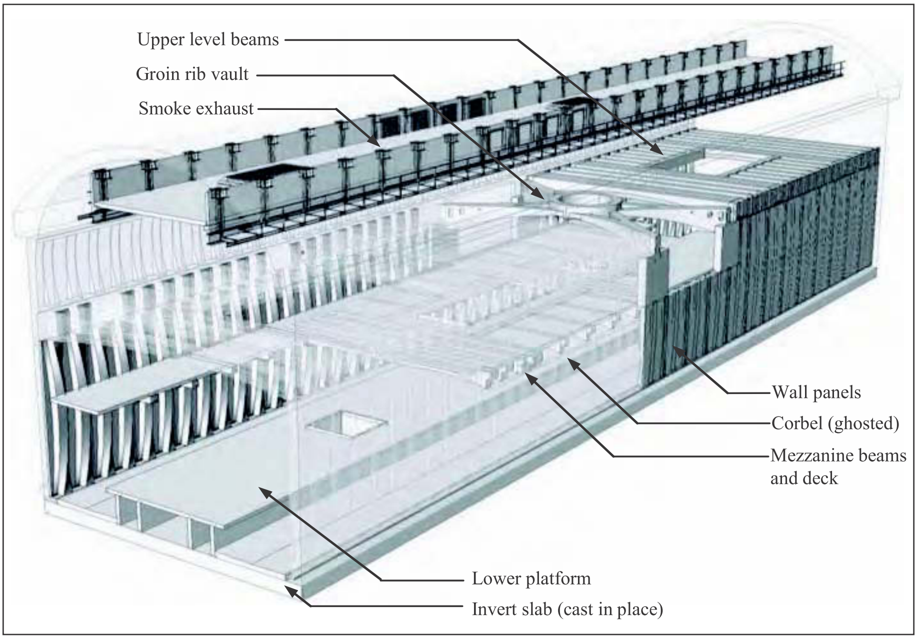

Figure 14. Typical precast units in the GCT main caverns

Table 1. Type and quantities of precast members

Precast Members	Types	Total Units
Smoke exhaust hollow core planks	9	498
Smoke exhaust wall panels	5	576
Smoke exhaust box	2	56
Wall panels	10	472
Drop-in and closure panels	1	508
Corbels	18	76
Mezzanine level beams	24	338
Node groin ribs	4	32
Mezzanine level deck panels	34	336
Upper level beams	15	524
Upper level deck panels	16	508
Platform wall panels	42	682
Platform slab panels	40	370
Precast stairways	9	72

DESIGN OF PRECAST ELEMENTS

The GCT Main Station Caverns is comprised of the precast elements listed in Table 1. As described earlier, typical challenges in the design and development of each component included weight limitations, simplification of connections and installation, and component repetition. Additionally, practicality in its fabrication and repeat uses of the same formwork for multiple types, were also kept in mind to maximize the advantages of precast design.

The design of the caverns was based on the provisions of the American Railway Engineering and Maintenance-of-Way Association (AREMA) Manual for Railway Engineering, the Building Code of New York State, and the design standards of the Metropolitan Transportation Authority of New York. Additional design requirements include NFPA 130 Standard for Fixed Guideways Transit and Passenger Rail Systems, standards from the American Concrete Institute (ACI) and the Precast/Prestress Concrete Institute (PCI).

The interior components of the structure were designed for a Cooper E50 train load at the Upper Level tracks and a pedestrian live load at the public areas in the Mezzanine Level and Upper and Lower Level platforms. The cavern lining was designed for rock loads and hydrostatic pressure. Other loads considered in the design included: a collision load applied to the platforms, seismic forces on the inter components of the caverns, and pressure loads from the piston effects from train movement in the tunnels.

Concrete specified for the precast units called for 6,000 psi strength with ground granulated blast-furnace slag, white silica fume and metakaolin supplementing the standard Portland cement. For cast-in-place concrete and self-consolidating concrete a concrete strength of 4,000 psi was specified. Self-consolidating concrete was specified in areas were access to vibrate the concrete was restricted and in thin concrete pours; such as behind the concrete wall panels and closure pours respectively.

The design developments for each precast member would extend beyond the scope of this paper. However, major components of the structure that posed significant challenges during design included the smoke exhaust, wall final liner, beams, corbels, decks, and platforms.

Smoke Exhaust

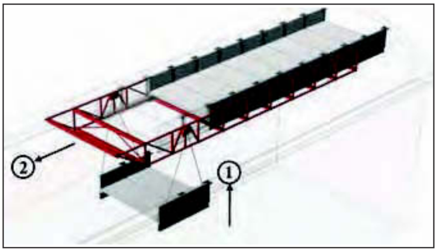

Figure 15. Self-launching gantry crane

The overhead smoke exhaust structure was designed to temporarily support a gantry crane to assist in the construction of the cavern (Figure 15). The smoke exhaust is connected to the underside of the caverns arch, and its primary steel framework provides for the running rails of an overhead gantry crane assembly. Construction of the smoke exhaust structure was designed to be self launching in which the gantry crane would essentially build sections of the structure by hoisting preassembled segments in front of it, and proceed forward once all elements were bolted into place. This self launching construction is similar to self launching construction often seen in segmental precast bridges and vertical steel cranes in buildings.

Wall Final Liner Panel

The final lining of the cavern walls are designed to resist the external rock and hydrostatic loads. Additionally, it is designed to resist the internal shear and moment loads transferred from the fixed connections of the mezzanine and upper level beams, as well as loads transferred from the existing separated arch (Figure 16).

One of the primary constraints on the precast wall panels was the 25 ton weight limitation. The original cast-in-place design of the wall called for 3'-4" in thickness by 41'-4" in height, with reinforcement evenly distributed as in typical wall design. Sectioning this into vertical strips for its full height, and in widths equal to the 5 foot center-to-center spacing of the beams, exceeded the weight capacity by nearly twice. Additionally, a wall panel of such width would result in too many pieces to make up the entire length of the caverns.

To reduce the weight of the panel various modifications to the panel were considered. The geometry was revised to a 10 foot double T-beam shape, with its web oriented outwardly towards the rock surface, and its flanges serving as the final formwork. The space between the precast panel and the rock layer was to be filled with concrete, resulting in a total wall depth of 3'-4". This significantly reduced the weight of the precast wall panels. The height of the wall was also reduced to terminate at the upper level beams. This would allow for a key closure pour connection at the upper level track to accommodate for the varying as-built conditions of the underside of the separated arch structure.

In an effort to reduce the amount of cast in place concrete fill cast behind the wall panels, the panels were flipped resulting in the web of the double T-beam facing away from the rock face creating a composite T-beam section. The effective depth of the wall remained unchanged, however the space between the webs no longer require concrete fill. In close collaboration with the architects, the geometry was furthered altered to introduce a twisted geometry.

The connections of the precast walls to the invert, mezzanine beams, and upper beams proved to be an equally challenging task that affected the design of the wall panel. The connection to the invert must resist the significant end shear force resulting from the external rock and hydrostatic load (Figure 17). At the same time, it would have to be simple enough to allow for the installation of the 25 ton precast wall unit. The resulting connection consisted of a structural steel square tube, placed into a socket

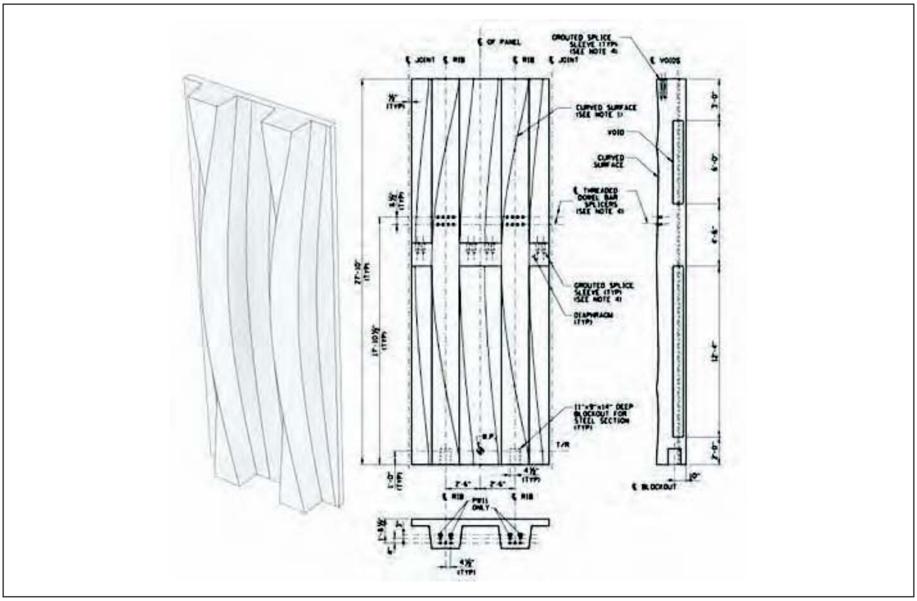

Figure 16. Precast wall final liner panel

cast at the underside of the wall panel at predetermined locations in the invert. The sockets assist in aligning the wall panels during its installation. The cavities within the steel tubing as well as the annulus around the perimeter of the tube would be grouted to assure full bearing contact. The connection of the mezzanine beam required an additional corbel component to be attached to the wall panels. To accept and support the corbel during construction and to assist in transferring the loads, a diaphragm was cast integral with the wall panel. Pre-embedded sleeves in the diaphragm and corbel allow for installing grouted dowels after the corbel and mezzanine beams were installed, simplifying the connection.

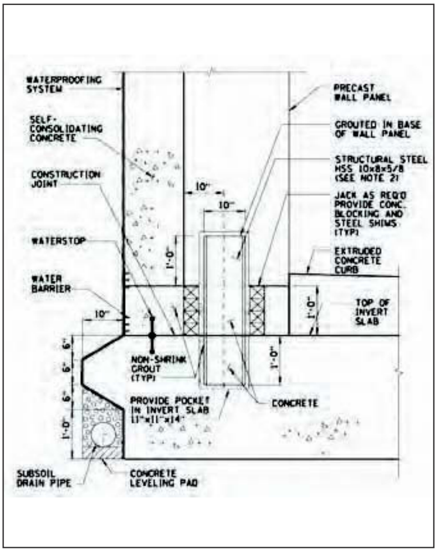

Figure 17. Typical precast final liner panel invert connection detail

Mezzanine and Upper Level Beams

Converting the mezzanine and upper levels from a cast-in-place design into a precast design was initially envisioned as a series of T-beams placed side-by-side. However, it was quickly realized that providing for a transverse connection at its flange to allow for longitudinal load transfer would be complex. Furthermore, some of the wider beams would also exceed the weight restrictions. Thus, the T-beam configuration was revised to be composed of precast beams (web portion only) with deck

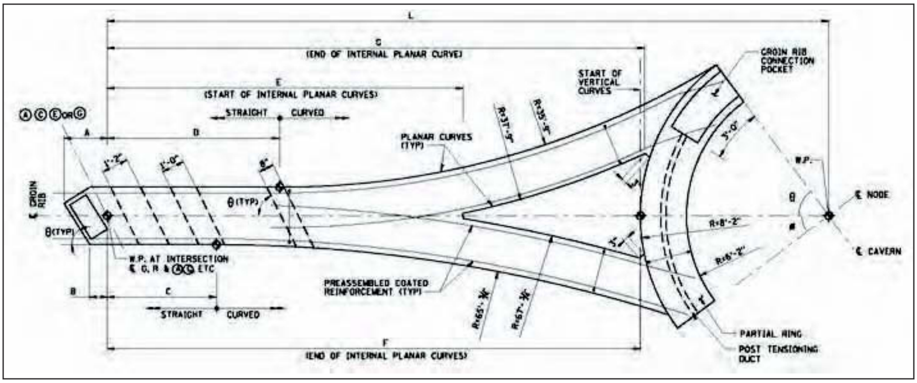

Figure 18. Precast groin rib component

panels spanning from the edges of each beams instead with a cast-in-place closure pour between the deck panels; and thus, would provide a rigid connection to the pre-cast beams.

In their final condition, the mezzanine and upper level beams were designed to act compositely with the concrete deck panels with its end restrained to develop negative moments. However, in the initial stage of construction, the beams were non-composite and simply supported. In keeping with the sleek aesthetic vision for the architectural design of the station and maintaining sufficient head room clearance, the beams were developed with a parabolic profile and very slender throughout most of its length; which in the initial stage of construction resulted in large deflections. To account for the large deflection expected from the initial dead load, the beams were required to be cambered.

The calculation of the deflection was complicated due to the staged construction, differing member properties and boundary conditions, and the time dependent properties of the concrete. Deflections were developed considering the two stages and time effects of construction; and evaluated for code compliance, constructability, functional operation, passenger comfort and effect on architectural finish. The loading conditions included the initial dead load, sustained dead load, train and station passenger live load, impact load, and creep and shrinkage effects.

Groin Rib Vault

At the mezzanine level where the caverns are connected by perpendicular cross passages and escalator banks, a different type of upper level structure was required that differed from the beam structural system. A large vertical 12' diameter exhaust shaft was required at the center of the space for mezzanine emergency smoke evacuation. The mezzanine and cross portals also required column free and open spans for passenger circulation. Additionally, mechanical and electrical conduits required substantial space for routing utilities.

This structure was required to support the train loads of the upper level tracks, electrical chases and air ducts located below the upper level platform, and the architectural finishes of the mezzanine level ceiling. Various structural options were considered which included: a cast-in-place dome, a series of variable depth precast concrete panels, and a groin vault. The groin vault consisted of four oblique precast concrete ribs attached to a central compression ring, supporting a cast-in-place two way slab. The groin rib vault structure was selected because it best transferred the loads to the cavern final lining walls through an arch effect while providing the significant clearance for the utility systems (Figure 18).

In order to maintain the central ring of the groin vault structure in compression under different load cases, post tensioning strands were used along the center of the ring. In addition, the upper level platform wall panels were designed to span over the node and transfer upper platform and passenger load to the upper level beams adjacent to the node area. This allowed the reduction of the long term deflection of the groin rib vault structure.

Upper Level Beam Connection

The final connection of both the upper and mezzanine level beams into the wall panels were designed to develop full shear and moment transfer. The upper level beams extend beyond the faces of the wall panels and are supported directly on top of the wall panels. The beam's primary reinforcement extends beyond the wall panels and bends into the cavity behind the wall panels to fully develop the reinforcement. Corrugated metal sleeves were embedded in the beams to allow for the wall ribs' primary reinforcement to remain continuous by passing through the ends of the beams. The vertical reinforcement from the upper wall closure pour would thread through the sleeves, and insert into the aligned grouted splice sleeves at the top of the precast wall panels. Once grouted, this connection would be a rigid moment transferring connection between the upper wall, the upper beams, and the top of the precast walls (Figure 19).

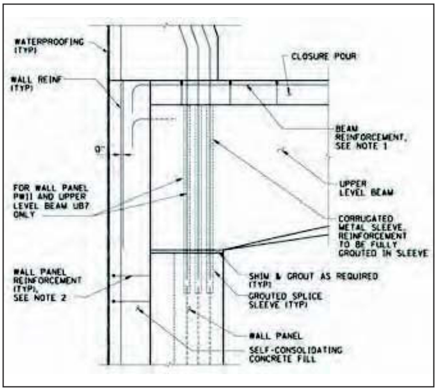

Figure 19. Upper level beam connection detail

Mezzanine Level Beam Connection

The mezzanine level beams, unlike the upper level beams, do not extend beyond the face of the wall panels. Therefore, they rely on a corbel structure as a supporting ledge for the beams to rest on during its installation and final condition. During installation, the beams are carefully positioned into place between the precast sidewalls of the caverns, and then lowered onto the corbel structure. To assist in this erection procedure, construction tolerances were built into both the corbel unit and mezzanine beams to allow for a 1" gap (Figure 20).

The mezzanine beams are designed to be simply supported on the corbels during its construction state. However, in its final state, the mezzanine beams are designed to be part of a fixed end moment frame with the cavern walls. Such fixity must be assured in the detailed connection between the beams and the corbel; however, the built-in construction tolerance of the 1" gap creates a challenge. Relying on post grouting procedures does not fully assure the bearing contact required in the compression side (bottom region) of the beam. The solution was to pre-embed a threaded rod/nut assembly

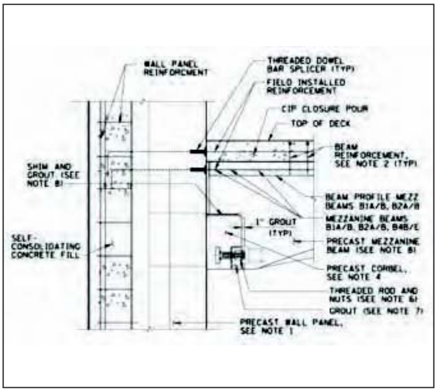

Figure 20. Mezzanine beam connection detail

into the corbel structure and a steel bearing plate into the ends of the mezzanine beam. Once the mezzanine beams are positioned onto the corbel, the threaded rod/nut assembly will be adjusted and torqued in the field to fully bear against the bearing plate in the beam. The contact between the nut and bearing plate will assure full transfer of the compression forces and restrict any rotation from the beams end. The negative (top) reinforcements of the beams are then connected into the pre-embedded sleeves in the precast walls. Once the closure pours and grout are set, the fully embedded negative reinforcements and the bearing contact on the compression face results in a rigid moment connection between the mezzanine beams and walls.

Multiple Beams

Framing around openings at the Mezzanine Level required large beams to support the heavy loads. With the restriction on the weight of the beams at 25 tons, a concept was developed to use three beams to share the loads. The triple beams were designed to share the applied load based on their relative stiffness; essentially, designed as a one piece homogenous member with full support at their ends. Transverse post-tensioning was provided along the length of the beams to tie them together, so that they would deflect together and behave as a single member. A finite element analysis was performed to confirm that the beams would behave as one.

Due to the configuration of the wall panels at the beam supports, the beam connections were designed with the exterior beams restrained from rotation and the interior unrestrained. The ends of the exterior beams are dapped to sit on a corbel and the dapped end of the interior beam extended to rest on both the diaphragm between the ribs of the wall panel and on the corbel–such that a portion of the load from the multiple beams would be transferred from the interior beam directly into the diaphragm and the remaining load applied to the corbel, which transfers the load to the diaphragm on the other side of the wall panel ribs.

To evaluate the flexibility of the corbel, a second model was developed using beam members representing the beams and corbel, and with fully restrained shear links modeled at the locations of the post-tensioning bars. The results of the analysis indicated that the rigidity of the diaphragm supporting the interior beam relative to the flexibility of the corbel has the tendency to shift the shear forces at the ends of the beams from the exterior beams to the interior beam; however, the amount of load transferred is dependent on the location of the shear links. The closer the shear links are to the end of the beams, the more load is transferred to the interior beam. To obtain an even distribution of the support reactions, the location of the post-tensioning rods were adjusted.

The beam analysis showed that the shear forces in the shear links compared favorably with the shear forces from the finite element analysis with smaller shear forces at mid span of the beams and increasing towards the ends of the beams.

Overall, the analysis confirms the assumption that the three beams act together in resisting the applied loads, that the transverse post-tensioning is adequate to transfer the shear forces between the members, and that the three beams share the load based on their relative stiffness.

Corbel

The purpose of the corbel component was to provide for a horizontal supporting ledge for which the mezzanine beams can rest on during its erection stage. This expedites the field installation process and aids in the accelerated construction concept by eliminating the need for temporary shoring to erect the full spanning mezzanine beams. It is envisioned that the lifting crane will install a series of mezzanine beams, followed by the mezzanine decks, thus creating an immediate working platform for which the next work activity to begin.

The development of the corbel geometry was a challenge. Typical precast design often embeds a steel angle into the walls to be used as a shelf to receive the precast beams. For this case, however, embedding any protruding steel angle into the finish face of the precast final lining wall would impede in the fabrication process. Pre-embedding inserts into the walls to allow for a steel angle to be bolted in the field was a considered option; however, the design would have required a significant number of connections to properly transfer the entire shear load expected from the mezzanine beams, thus, being impractical for accelerated construction. The corbel shape, and the development of the integrated diaphragm in the precast walls, was a solution that simplified field installation and fully transferred the reaction loads from the beams. Additionally, the corbel structure allowed for multiple beams to be immediately installed, hence, expediting the construction work flow (Figure 21).

The corbel acts essentially as a transfer girder supporting the mezzanine beams. It is designed to transfer the loads from the mezzanine beams into the cavern's walls. The corbel is attached to the face of the precast wall panels by being "hung" from the built-in diaphragms of the walls. The corbels' connection to the walls is simplified with the use of headed rebar dropped into place through the corbel and into pre-embedded sleeves in the diaphragm. This connection resists the horizontal shear caused by the rotation of the corbel due to its eccentric loading. Design of the corbel structure required closely spaced shear stirrups to resist the reaction loads from the mezzanine beams. Furthermore, due to the corbel being "hung" from the wall's diaphragm, proper caged reinforcements were designed to resist the internal torsional rotation. Exposed rebar cages protrude through the top of the corbel and into the closure pour regions to assure monolithic behavior with the mezzanine beams and decks.

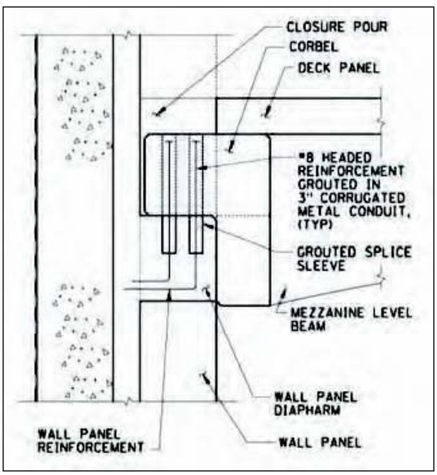

Figure 21. Corbel detail

Mezzanine and Upper Level Decks

The design of the precast concrete mezzanine and upper level deck panels was consistent with the design of the supporting beam elements (Figure 22). The design accounted for the temporary condition during construction and act compositely with the beams during the permanent conditions. The detailing of the deck panels minimized the amount of shoring and formwork required for the construction of the mezzanine and upper level floor system. This provided a sealed working surface and reduced the amount

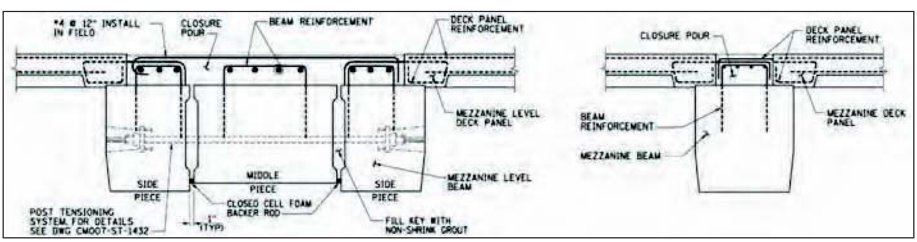

Figure 22. Upper level (left) and mezzanine (right) beam details

of cast-in-place concrete required for the closure pours. Welded wire fabric reinforcement was used for ease of fabrication of the precast members. Provisions were made in the design of the deck panels to inlayed acoustic material into the precast panels to comply with acoustic requirements of the station and reduce the need for on-site sprayed application. In addition the design of the deck panels accounted for the required openings to accommodate track drainage, electrical and mechanical services.

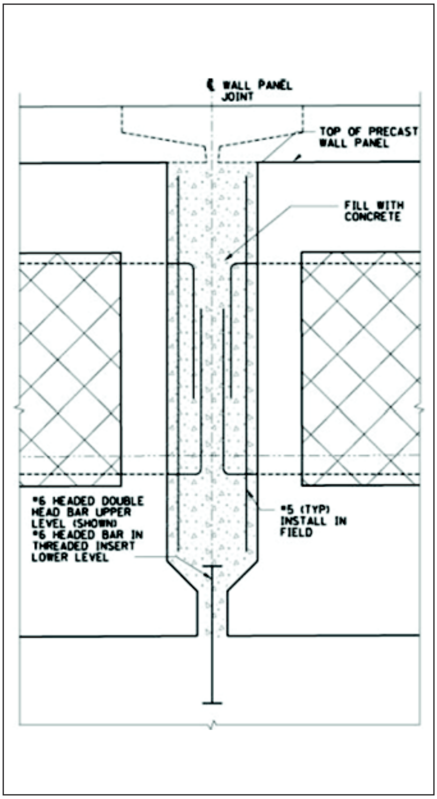

Figure 23. Platform closure pour connection detail

Platforms, Walls, and Decks

The upper and lower level platform structure in the public area of the GCT Main Station Caverns consist of precast concrete wall and deck panels connected by vertical and horizontal cast-in-place closure pours (Figure 23). The lower level platform rests on the cast-in-place invert slab and the upper level platform is supported by the precast concrete upper level structural framing. The platform structure was designed for a pedestrian live load of 150 PSF and for a live load of 250 PSF for mechanical and electrical areas. In addition, the platform structure was designed to resist a collision load of 225 kips at a 10 degree angle to the train direction. Seismic loading was also considered as part of the platform structural design. The lateral loads imposed on the platform are resisted by a series of shear walls mainly located around the platform openings. Structural continuity in the reinforcement of the deck panels was provided so the deck panels will act as diaphragm when resisting the lateral loads and also be able to transfer the loads to the shear walls. Based on the functional requirements and layout of the station, the design of the platform structure was further subdivided into four areas: typical pedestrian area, escalator/stair area, electrical/mechanical area and elevator area. The design of each platform area required the development of various types of platform deck and wall panels due to interfaces with: vertical circulation elements (escalators/stairs/elevators); HVAC (air ducts, smoke evacuation shafts and air supply pylons); electrical (under platform conduits and pull boxes); fire protection (under platform fire standpipe and deluge system) and architectural (terrazzo finish floor, hatches and acoustic insulation in the platform wall panels). As a result, the locations of the cast-in-place closure pours were closely coordinated and strategically located to allow for repetition of the platform wall and deck panels. Standardized structural connections were designed for the various areas of the platform structure which required the use of double head bars, embedded plates and bolted steel connection plates.

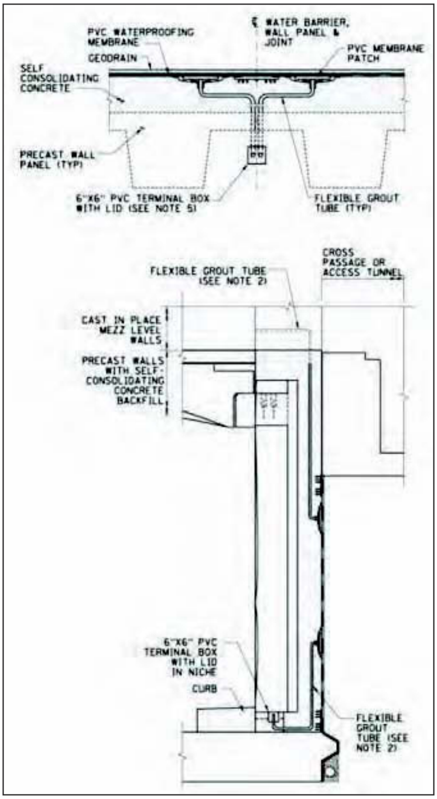

Figure 24. Waterproofing section and plan detail

Waterproofing

The waterproofing design of the GCT Main Station Caverns had to be revisited from the system used in the tunnel structures in order to accommodate for a precast final liner design (Figure 24). The waterproofing system consists of a layered geodrain, geotextile, and PVC membrane. The waterproofing membrane is further sectioned into rectangular compartments by water barriers welded to its surface. These compartments allows for future leak repair efforts to be isolated to only the regions where the leak damage occurs. Grout tubes installed from the surface of the waterproofing membrane extends out to the surface of the concrete liner. Leak repairs to the damaged area are done so by injecting grout through these grout tubes, sealing up the section where water is infiltrating from.

With the cast-in-place design, the water barriers are often laid out in grids defined at every construction joint and at practical equidistant spacing that provide for a manageable sized compartment for repairs. The grout tubes are typically installed at the corners of each compartment, allowing the grout to seal the compartments starting from the corners and working its way towards the center. However, with the final liner converted into a series of precast wall components, the water barrier had to be relocated to coincide with the joints between the precast components. Furthermore, placement of the grout tubes was relocated to be accessible in a safe location and penetrations through finished precast surfaces avoided where possible to simplify the connection and erection process.

The solution was to utilize flexible grout tubes rather than the rigid grout tubes typically used in cast-in-place construction. The flexible grout tubes allow for access to repair locations to be rerouted to a more desirable location. For the case of the of the wall panels, the grout tubes were routed from the waterproofing surface to the access location by running underneath the precast wall panels and terminating into terminal boxes installed in the cast-in-place invert. This method avoided any penetrations through the surface of the wall, the need for any pre-embedded connection, and additional steps during construction to connect the grout tubes; and kept the tubes in the cast-in-place regions only.

CONCLUSION

The design of the GCT Main Station Caverns used construction technologies and techniques such as precast concrete, prefabrication of components, or preassembly of units that may expedite the installation time during construction. The design used proven techniques developed in other construction industries but modified to suit

Figure 25. Lower platform with bridge and elevator

Figure 26. Mezzanine with groin vault

Figure 27. Upper platform

highly restrained conditions encountered in underground conditions 150' below ground. The advantage of prefabrication was further extended to the architectural finishes and associated mechanical and electrical systems creating a multidisciplinary integrated design. For the GCT Main Station Caverns, techniques included precasting the primary structural members to significantly reduce the volume of concrete that would have otherwise been cast-in-place, and prefabricating the reinforcement to reduce on-site labor (Figures 25–27).

ACKNOWLEDGMENTS

The authors would like to thank the Metropolitan Transportation Authority Capital Construction, the General Engineering Consultant (GEC)'s Project Director Jerome Forman and Chief Engineer Jesus Schabib for their support and guidance; Colin Barratt for motivating a successful team of designers to engineer the vision of a signature structure by the use of innovative designs; the GEC's design team for their endless support and commitment to the ESA Project with special attention to structural engineers Victor Shey, Michael McBrearty, Ravi Jain, Xiaojun Wang, Wo Chen, Artur Kasperski, Mario Valenti, and architect Elton Elperin. Graphics are courtesy of the MTA. Finally our most and sincere gratitude goes to the families of the designers that supported them to complete the design to meet our client's demands.

PART 2

Contracting Practices

Chairs

Brad Cowles
Jacobs Civil

Dan McMaster
Hatch Mott MacDonald

DELIVERING VALUE AND RISK MANAGEMENT THROUGH A COMPETITIVE ALLIANCE PROCUREMENT PROCESS

Peter Spies ▪ New Zealand Transport Agency

Tom Ireland ▪ Aurecon

ABSTRACT

The selection of the Procurement method for major projects often has a significant outcome on the ultimate delivery of a project. The $1.2B Waterview Connection project is the largest and most complex project yet undertaken by the New Zealand Transport Agency and consideration was given to many contract forms. This paper describes the NZTA decision process in adopting the competitive alliance model, and more particularly measures taken during the procurement process to achieve certainty with respect to outcomes and minimize project risks whilst also achieving value for money.

Refinements to the procurement process to ensure compliance with the Code of Practice for Risk Management of Tunnel Works prepared by the International Tunnel Insurance Group are described along with the other risk mitigation measures applied to achieve a successful procurement outcome. Also described is the alliance commercial model which serves to both facilitate collaboration between the Contracting consortia and Client to achieve "best for project" outcomes as well as managing commercial risks for all participants.

INTRODUCTION

In March 2009 the New Zealand Government announced its Transport Policy through a Government Policy Statement (GPS). A key component of the GPS was the announcement of the Roads of National Significance (RON's) programme of 7 key projects. This involved a commitment to upgrade transport infrastructure, particularly where such projects would promote economic development and efficiency by removing bottle necks and assisting mobility.

Auckland is New Zealand's largest city, with a population reaching 1.5 million by December 2012 and projected to grow to 2.5 million within 30 years. As the economic gateway to New Zealand, 61% of the countries imports and 32% of the countries exports pass through either Auckland Airport or Seaport. Accordingly 3 of the 7 RON's projects are located within Auckland, these include the recently completed Victoria Park Tunnel, the planned Puhoi to Wellsford motorway and the Waterview Connection project currently under construction and the subject of this paper.

This paper describes the competitive alliance procurement process followed, and more particularly measures taken during the procurement process to achieve certainty with respect to outcomes and minimize project risks whilst also achieving value for money.

PROJECT DESCRIPTION

The Waterview Connection project once completed will deliver a massive increase in the capacity of the Auckland motorway network. The city, suburbs and gulf islands cover an area of 637km^2, sitting across an isthmus less than 2km wide at its narrowest point resulting in transport links running through a congested central corridor. Upon completion the Waterview Connection together with three other adjacent projects will fully open the Auckland Western Ring Route (WRR) progressively under construction over the past 15 years, providing an alternative to State Highway 1 (SH1) ensuring greater resilience and reduced congestion through central Auckland. Refer to Figure 1 for details of the Western Ring Route.

The project is 4.5km in length and includes 2.5km of twin three lane motorway tunnels (14.53m OD) constructed using a EPB TBM. The tunnels pass under a built up residential area and a major local arterial carrying over 50,000vpd before surfacing and connection into a full motorway to motorway interchange linking SH20 to SH16. Refer to Figure 2 for the layout of the project.

Figure 1. Western Ring Route location

SELECTION OF CONTRACT FORM

The New Zealand Transport Agency (NZTA) uses a range of contracting models to deliver projects including Measure and Value (M&V) for small projects, and Design and Construct (D&C) where there is considered to be some opportunity for contractor innovation and/or to design risk out of the project.

Alliancing is the NZTA's premium contract model used where projects are highly complex and have inherent risks that are best managed through a collaborative contracting relationship. While a number of tunnels have been built in

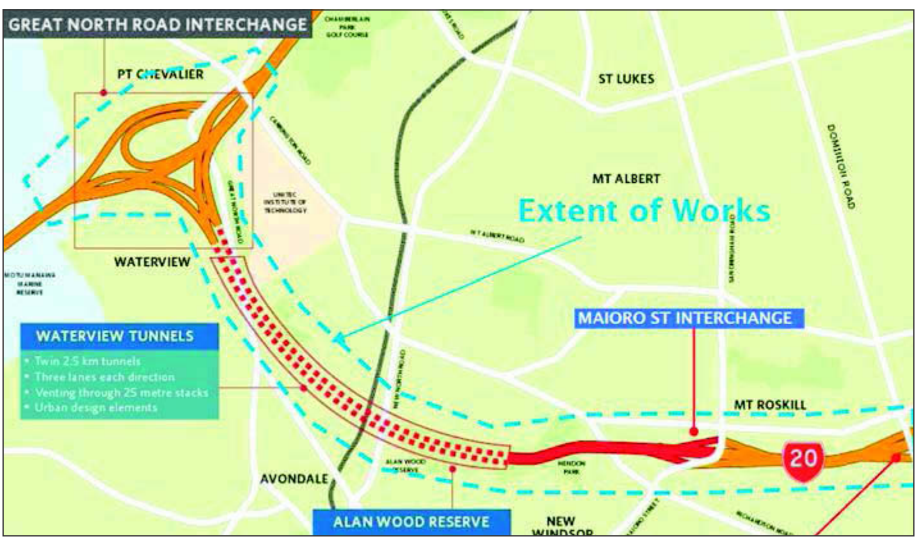

Figure 2. Waterview connection project scope

New Zealand none were of the size or length required for the Waterview Connection. The length of the tunnels and their operating environment also required specialized fire and life safety features while construction would take place in a built up residential area with major traffic issues and a sensitive environment.

A project Alliance is where an owner forms an alliance with one or more service providers (designer, contractor, suppliers etc) for the purpose of delivering outstanding results on a specific project. Project alliancing was first used in the UK oil and gas infrastructure procurement in the early 1990s and then became widely used in Australia.

The key differentiators of alliances from other contracting models are:

- Performance obligations are generally stated to be collective (i.e., the Alliance Participants commit to work together in a manner so as to achieve the successful delivery of the Work under the Alliance and to act in Good Faith).
- Reimbursement to the non-owner Participants is 100% open book subject to verification by audit and can be described as a 3-limb compensation model:
  - 100% of what they expend directly on the work including project-specific overheads (Limb 1).
  - A fixed lump sum Fee to cover corporate overheads and profit (Limb 2).
- An equitable sharing of gain/pain depending on how actual outcomes compare with pre-agreed targets in cost and non-cost performance areas (Limb 3).
- The project is governed by a “Project Alliance Board” (PAB) with representatives from all parties who carry full authority to bind the party. All decisions of the PAB are required to be unanimous.
- There is an express commitment to resolve issues within the alliance without recourse to litigation except in the case of “willful default.”
- The Alliance Participants develop and commit to work within an agreed set of “Alliance Principles.”

The NZTA became aware of the benefits of this risk-sharing approach and first used the model in 2000 on the Grafton Gully–Freeflow alliance a major connection between the Auckland Port and SH1. Since then eight State Highway projects have been successfully delivered or are being delivered as alliances.

For the Waterview Connection project the relevant characteristics and desired outcomes that led to the decision to use the project Alliance model can be summarized as follows:

- Risks could not be adequately defined or dimensioned prior to tendering—a common feature of tunnel projects
- The cost of transferring risk would be prohibitive under a D&C model—Alliancing provides for risk sharing
- A collective approach to assessing and managing risk will produce a better outcome—the alliance model allows for the project solution to be progressively refined and developed to reflect emerging risk.
- A Whole of Life approach—The operating costs of tunnels are high and it was felt that combining the D&C phase with a 10 Year Operate and Maintain phase would allow optimal decision making across the two phases.
- Value for Money—The commercial arrangements in the alliance model strongly incentivizes parties to achieve Value for Money. The competitive alliance procurement model also relies on the competitive process to drive innovation to achieve a lower initial TOC and this has been demonstrated across a number of projects in Australia and NZ.

- The NZTA had acquired the skills and capacity to influence or participate in the development and delivery of the project—the model allows for the combination of skills from all parties to be applied to the collective outcome.

MEASURES TO ACHIEVE COMPLIANCE WITH THE ITIG CODE OF PRACTICE FOR RISK MANAGEMENT OF TUNNEL WORKS

The procurement process was refined to provide compliance with the Code of Practice for Risk Management of Tunnel Works (TCoP) prepared by the International Tunnel Insurance Group (ITIG, 2006). The principle of shared risk within an Alliance incentivizes a common understanding of project risks prior to finalizing the Alliance Agreement and therefore should enable superior risk management through construction and beyond. As best practice risk management is a primary objective of TCoP this procurement method should be beneficial to all concerned.

One of the main objectives of the TCoP is to set minimum standards for risk assessment and ongoing risk management procedures for tunnelling projects whereby compliance with the TCoP should minimize the risk of physical loss or damage and associated delays to a level 'as low as reasonably practicable' (ALARP). The formalized risk management procedure proposed by the TCoP is used as a means of documenting formally the identification, evaluation and allocation of risks. Significantly there are three sections addressing activities in predesign stages including: client role and responsibilities, project development stage and contract procurement stage. This is an acknowledgment that risk management practices need to be instigated well in advance of commercial phases, so that commercial competition does not lead to a significant elevation of project risk. In particular the practice of gaining commercial advantage by taking on a potentially unmanageable level of risk is one of the outcomes to be avoided by the TCoP.

Client Role and Responsibilities

The Alliance concept is considered to maximize the potential for full integration of the traditional roles of Client, Constructor and Designer.

In addition to key NZTA staff integrated within the Alliance, the NZTA has elected to introduce the function of an Owner Interface Manager (OIM) who is accountable for the delivery of WRR projects and delegated to making project decisions on behalf of the Client. The OIM sits within but "separate" to the Alliance to represent the Owner (NZTA) through the delivery of the Alliance contract. This role has been created to meet the following requirements:

- Coordination of consent requirements across the entire WRR
- Coordination with adjacent Contracts
- Manages NZTA risk across WRR
- Ensure that cash flow expectations and budgets are managed and coordinated
- Ensure that whole of life (WOL) approach meets NZTA expectations and includes for a 100 year life
- Technical confirmation that NZTA objectives are being achieved including meeting the Requirements and Minimum Standards
- To ensure the necessary interfaces and interactions in order to achieve efficient advice to the Alliance
- OIM has the power to direct changes

The OIM is supported by internal NZTA technical experts and external experts including an Owner Verifier (OV) with relevant tunnelling and M & E experience to fill the skill gap in NZTA. The 'Owners Verifier' provided independent technical advice on behalf of the NZTA both during procurement and the delivery phase of the project i.e., they are 'outside' of the Alliance. These technical advisors include experienced tunnel engineers familiar with construction of road tunnels, Auckland conditions and international best practice for risk management including the TCoP.

Project Development Stage

One of the key requirements of TCoP during the project development stage is the assessment of project options and this includes:

- Geology and hydrogeology
- Tunnelling methodologies
- Ground movements and surface settlements including 3rd party impacts

Geology and Hydrogeology

The tunnel is to be excavated predominantly through East Coast Bays Formation (ECBF) which occurs throughout the Auckland region and which comprises shallow dipping alternating beds of extremely weak to weak sandstone and siltstone. Within the ECBF, there are occasional interbedded lenses of Parnell Grit, a weak to moderately strong sandstone. The Waitemata Group sediments were deposited during Miocene times (around 26 million years ago) during the mid-Tertiary submergence when Auckland was entirely underwater. This sandstone/siltstone was deposited underwater in a shallow basin, and has been uplifted and subjected to faulting since.

At the northern end the tunnel passes through mixed face conditions including the Pleistocene material derived from the ECBF rocks that was deposited in a shallow marine environment to form the firm to stiff clay of the Tauranga Group.

Extensive geotechnical investigations were carried out for the project over a 10-year period, and all of these investigations were collated in a Geotechnical Data Report for use by the tenderers. A summary of the investigations undertaken is shown in Table 1.

All test results from all investigations were collated in a database allowing graphical presentation of geotechnical parameters. Tenderers were also invited to review the geotechnical information available and request additional investigations that were carried out by the NZTA during the tender period, resulting in the drilling of an additional 58 cored drillholes, 18 CPT's and 10 Dynamic Probes, and a suite of additional testing including specialist rock testing.

Extensive hydrogeological investigations and interpretation was undertaken prior to the commencement of the tender period including the establishment of 315 peizometers and conducting 3 pump tests. An area of high water ingress predicted to be up to 60l/s was identified, and modeling was undertaken to determine the extent of drawdown for various tunnelling methodologies. All this reporting was made available to tenderers.

Tunnelling Methodologies

Project development phase investigations included a study of both a Sequential Excavation Method (SEM) combined with a cut and cover tunnel, and an Earth Pressure Balance (EPB) TBM, with the viability of both methods confirmed. There had been prior experience with EPB tunnelling within Auckland, with 6km of small bore

Table 1. Waterview connection geotechnical data report summary

Item	Number
Factual Reports Sourced	24
Investigation positions registered	1,367
Length of logged geology	12.7km
Number of Insitu tests	4,086
Number of laboratory tests	1,983

wastewater tunnels construction over the last 4 years, however nothing approaching the scale of the proposed Waterview tunnel.

The major identified risk with this tunnel is the low (9m) cover to the arterial road at the northern end. This section also has mixed face conditions with soft ground in the crown. The SEM solution utilized a 500m long cut and cover tunnel to mitigate the risks associated with these conditions. This however imposed significant disruption associated with service and traffic diversions, and additional cost due to the physical works and longer program.

For the EPB option the face stability risk in the soft ground conditions is mitigated with closed mode EPB operation.

The tender documentation left the selection of the tunnelling methodology to the tenderer, however Minimum Requirements were specified for each of the tunnelling methodologies including mandatory forward grouting for the SEM methodology and a detailed EPBM specification for the TBM option.

Ground Movements and Surface Settlements Including 3rd Party Impacts

The project has been subject to a detailed and comprehensive statutory planning process. The '*technical viability'* of the project was established through the Board of Inquiry (BOI) into the Waterview Connection which is a statutory planning approval process. A key principle involves the definition of broad envelopes of environmental effects within which 'resource consents' are to be agreed between the NZTA and regional and local stakeholders. These resource consents define environmental effect limits for construction and operation of the Works that must be complied with. The Assessment of Environmental Effects (AEE) included a detailed assessment of settlement associated with face loss and consolidation settlement related to drawdown. A detailed noise and vibration assessment was also undertaken.

The tenderers had to comply with the consent conditions that included prescriptive requirements for monitoring and management of settlement effects. Additional Minimum Requirements were included by the NZTA within the tender documentations in order to mitigate the risks associated with 3rd party impacts.

Contract Procurement Stage

This is a key aspect of the risk reduction sought by TCoP as procurement methodologies used on previous projects internationally have led to the elevation of project risk. The detailed measures used to reduce risk during the procurement are described in detail in the following sections of the paper.

Ground Reference Conditions

During the procurement phase the NZTA placed high value on design and construction mitigation measures proposed in order to address ground risks, as is appropriate for a major tunnel project. In order to achieve alignment on the level of geotechnical risks an Alliance Geotechnical Baseline Report (AGBR) process was developed. The purpose of the AGBR is to provide a comprehensive discussion of geotechnical risks and to pose a series of detailed questions of the proponents' design and construction methodologies in respect of how they will address all geotechnical risks. Baselines of low probability but high consequence geotechnical risks that may otherwise impact the Target Outturn Cost (Contract Price) and which may also have the potential to skew the competitive tender process are defined by agreement. The AGBR contains definitive statements about the nature, form, composition and structure of the ground (both artificial and natural) and groundwater, together with geotechnical properties of the ground. The wording of the AGBR and the baselines have been agreed by the NZTA and the Alliance Proponents prior to submission of Tenders in order to develop a

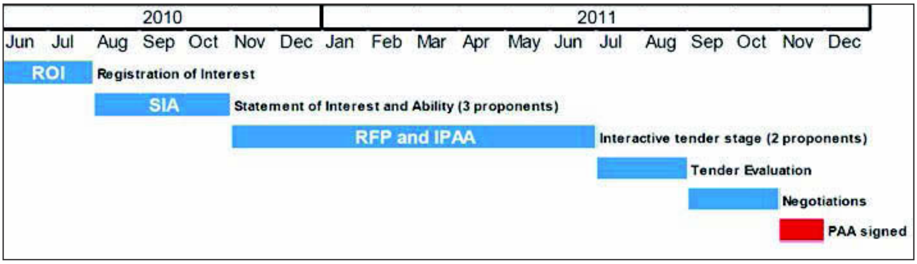

Figure 3. Procurement program

common understanding of the risks included within the tender and those excluded. The final agreed revision of the excluded risks was incorporated into the Project Alliance Agreement (PAA).

The final AGBR document meets the requirements of ground reference conditions as defined by TCoP.

DESCRIPTION OF PROCUREMENT PROCESS

The NZTA used a three stage procurement process over a 19 month period as shown in Figure 3. In July 2010 advertisements were placed seeking Registrations of Interest (ROI's) from Consortia having an appropriate combination of local and international expertise and best practice to deliver the Waterview Connection project. Three Consortia registered interest and an Inception meeting was held on 4 August 2010 where the NZTA's procurement process and requirements was explained to the Applicants and the Statement of Interest and Ability (SIA) documents issued. To further assist Consortia in their understanding of the NZTA's requirements, two interactive meetings with the Tender Evaluation Team (TET) were held separately with each Applicant before submissions closed on 4 October 2010.

The TET reviewed the submissions and met to complete their assessment resulting in two Proponents being selected to proceed to the next Request for Proposals (RFP) stage. Significant effort was put into optimizing the RFP documentation and the 7 month interactive tender process and was based on what the Project team referred to as the Four P's. Product, Process, People and Price, as it was recognized that to successfully procure a Proponent that would be working with the NZTA to deliver the project, we needed to ensure that we were being offered and could build the right Product through a robust Process delivered by the right People for the right Price. Key features of the interactive tender process are described in the next section with strict probity protocols established and followed to ensure that commercial-in-confidence information provided by one Consortium was not divulged to the other.

MANAGEMENT OF PROJECT RISKS DURING PROCUREMENT

Background

The background context to the increasing importance placed on risk management is the losses suffered by tunnel insurers over the last decade. The insurance industry (Wannick, 2006) has reported a general trend towards high-risk type construction methods, often delivered using design and build contracts with one-sided contract conditions, in an environment of fierce competition.

There are many guidelines on risk management strategies (Eskesen et al., 2004) and it is not proposed to outline this material in this paper. Some of the key points as they relate to the TCoP include the recommendations:

- to include in the tender documents specific technical requirements such that risks are managed in accordance with the risk strategy, with explicit allocation of responsibilities for risks;
- tender evaluation procedures should include an evaluation of the contractor's ability to identify and control risks by the choice and implementation of technical solutions; and
- systematic assessment of the differences in risk between the project proposals by different tenderers. Generally risk should be allocated to the party who has the best means for controlling them.

This section of the paper describes how a competitive procurement process was conducted to achieve value for money but without elevation of project risk.

Tender Cost Reimbursement

The NZTA has a policy of reimbursing tenderers a fixed sum equivalent to 60% of expected tendering costs. A pool of $18M was allocated for the Waterview Connection Tender. This was undertaken in an openbook way using the same audit procedures as the Alliance agreement. In exchange for the tender contribution, the NZTA owns the Intellectual Property generated by each of the Consortia including all tender design material and risk mitigation methods. The benefits of owning the tender IP is that the project risk and cost can be reduced by incorporating good ideas from the losing tender. In the case of Waterview, alternative designs for the ventilation fans and lighting from the losing tender were incorporated in the final Alliance Agreement. These savings were greater than the tender cost reimbursement to the losing tenderer.

Maintaining a Competitive Price Without Elevating the Project Risk Profile

Through the tender evaluation procedures, Tenderers are incentivized to maximize value through their design and construction methodology whilst retaining the ALARP principle with respect to the Design and Construction of the Works and impacts upon Third Parties. This is achieved through an interactive tender process comprising a combination of Alignment workshops and Technical/Consent meetings held separately with each Proponent.

Eight weeks before tender close the tenderers were also required to submit their tender design drawings in a process known as Certificate A. The designs were reviewed for compliance against the 'Requirements and Minimum Standards', and feedback on the designs including all identified non-conformances was provided at an Alignment workshop. Whilst there are some probity risks with this process, that need to be carefully managed, they are not significantly different to any interactive tender process, and there are benefits for both the Owner and the Tenderer. Due to the two-envelope evaluation process adopted where Tangible Cost Adjustments are made to the tender price for non-conformances, Tenderers are very keen to ensure that their tender design conforms to the project requirements and will not attract a price penalty during the evaluation. The benefits for the Owner are that this process ensures that there will be no non-conforming tenders which then makes evaluation difficult. It also allows the risk profile of the tender design to be evaluated, and further alignment with the Tenderers on the risk profile can be achieved prior to the final tender submission.

Alignment Workshops

Alignment Workshops were attended by relevant Proponent and NZTA representatives both from the core project team as well as practice area specialists and covered most project areas including the following topics relevant to risk management:

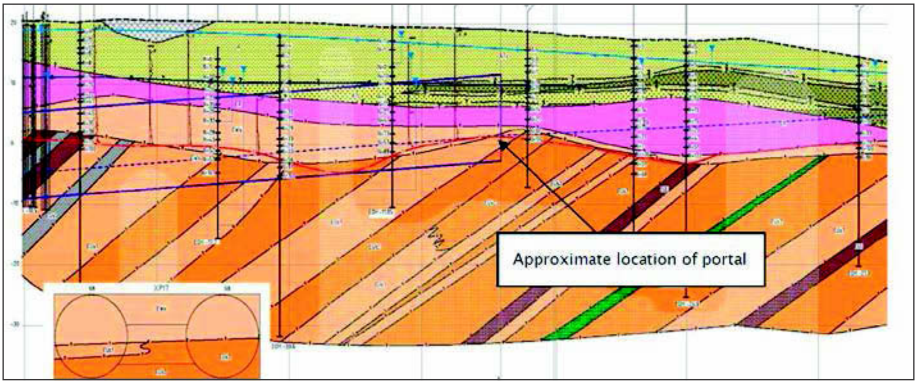

Figure 4. Geological longsection at Great North Road Crossing

- Tunnel operation and maintenance requirements/whole of life considerations;
- Tunnel optioneering;
- Project risks (including three Alliance Geotechnical Baseline Report (AGBR) workshops;
- Value Engineering;
- Variation Benchmarking.

These meetings were a key part of the interactive tender process in managing the risk profile of the project. Each tenderer is looking for ways to achieve a competitive advantage, with the main goal of winning the project and this focus inevitably leads to some alternatives that may be cheaper, but have much higher risk. As discussed above, each Tenderer compared SEM methodologies with an EPB-TBM alternative, but both tenderers identified that the long cut and cover tunnel crossing a major arterial road at an oblique angle was going to be disruptive and expensive. One of the tenderers considered replacing the cut and cover tunnel with a SEM tunnel under Great North Road. The cover at this location is less than 1 tunnel diameter, and the ground conditions include a mixed face with soft ground in the crown, all below the water table. The NZTA view was this alternative elevated the project risk, and this was communicated via the Alignment Workshops. As a result both Tenderers elected to base their tender on an EPB-TBM methodology which minimized the overall project risk but also provided a value for money solution.

Another good example of alignment on risk profile was the mitigation for face instability during the crossing of Great North Road at low cover. The geological long section for this area is shown on Figure 4. Whilst it is proposed to mitigate the face stability risk in the soft ground conditions with closed mode EPB operation, it is also proposed to temporarily divert the traffic while excavating the tunnel in order to reduce the consequences of any face control issues. In a competitive environment a contractor may elect to rely on construction procedures to achieve a stable face, however the road diversion is a simple solution that eliminates the risk associated with tunnelling under live traffic. Both Tenderers adopted this solution following alignment with NZTA.

Technical Consent Meetings and Departures

Technical/Consent Meetings were used to focus on technical matters such as the Requirements and Minimum standards (MRs) which were framed to encourage innovation, but also included specific technical requirements to ensure risks are managed in accordance with the risk strategy. Tenderers were encouraged to challenge the Minimum

Requirements where opportunity existed through the submission of Departures. Each Departure had to be accompanied by a 'Risk and Opportunity Statement' whereby the Owner and their technical advisors could ensure the Departures from the Standards and Minimum requirements did not result in the elevation of project risk. In addition to the Departure process, these meetings also proved useful in ensuring that the emerging Conceptual Designs would be acceptable and the risk of a Tangible Cost Adjustment (TCA) being imposed reduced.

Certificate A

As noted above, the "Certificate A" concept from NZTA's D&C procurement process was also adopted. This involved each Proponent submitting their Preliminary Conceptual Design Reports eight weeks prior to close of tender for review by the Project team. These Certificate A submission consisted of the following:

- Statement of compliance with the Requirements and Minimum Standards and Draft consent conditions;
- List of approved Departures incorporated into their Preliminary Conceptual Design;
- Road Safety Audit of the Preliminary Conceptual Design;
- The Design Statement associated with the Preliminary Conceptual Design, listing all referenced design standards, manuals, guidelines and specifications;
- List of key assumptions and risks;
- Description of construction methodologies consistent with the design intent and draft consent conditions;
- Operations and Maintenance Plan.

Feedback was then provided 6 weeks prior to close of tender to give the Proponent comfort that their submission would meet NZTA's requirements.

Tender Evaluation—Tangible Costs Adjustments

Evaluation was undertaken in accordance with NZTA's procedures which use a two-envelope system. Envelope One contained all non-price information including the following attributes: Relevant Experience (10%), Relevant Skills (30%), Resources (20%) and Methodology (40%). Further information evaluated included the detailed Conceptual Design and associated information such as Construction Methodology, Risk Register and Operations and Maintenance Plan. The non-price attribute scores were converted to a Supplier Quality Premium (SQP) based on $100m per 10% difference in overall attribute grades for 'quality of service' or 'how' things are delivered, while Tangible Cost Adjustment's (TCA's) were applied to quality of product differences or 'what' is being delivered, resulting from a detailed review. TCAs are identified in one of three ways.

- Quality of Product—whole of life costs, benefits and risks with respect to the quality of the product offered by one Proponent, relative to that of the other Proponent, including operation and maintenance regimes;
- Key Result Areas of the Project Alliance—risk associated with actual out-turn cost likely to result from the non-cost performance parameters offered by one Proponent, relative to that of the other Proponent, during the design and construction period; and
- Risk Exceptions—perceived proposed changes to the risks transferred to NZTA under the AGBR and/or the Variation Benchmarking Register, as well as residual risks, including those arising from construction methodologies, key plant resources proposed and mitigation strategies detailed in the risk register.

This evaluation method is described in detail in the Request for Tender document, and is well understood by the NZ contracting industry following its implementation on several previous projects. The ability of the Evaluation Team to adjust the Tendered Price for evaluation purposes means the Tenderers take the Interactive Tender process very seriously as their goal is to avoid any price adjustments as part of the evaluation process. This is a key mechanism in avoiding the elevation of project risk during a competitive tender process.

OUTCOME OF THE PROCUREMENT PROCESS

With both Proponents finding the "sweet spot" through the tender process the result of the tender evaluation was a slight advantage to the Tuhono consortium, but this was negated by a lower tender price submitted by the Well-Connected consortium who became the Preferred Proponent on 18 August 2011. Of particular satisfaction to the NZTA was that both Proponents had lower than expected allowances for Risk and Contingency. Feedback at the tender debrief sessions was that:

- The interactive tender process allowed Proponents to fully understand the NZTA's requirements;
- The extensive work done in preparing the Conceptual Design and feedback received through the Certificate A process had minimized design uncertainty;
- The AGBR risk allocation and commercial principles negated risk associated with tunneling uncertainty.

CONCLUSION

The success or failure of a project can often be traced back to decisions made prior to and during the procurement of the Contractor when the potential consequences of these decisions are not well understood. Effective risk management decision making is key to ensuring the selection of an appropriate procurement model while also ensuring that all relevant information is sourced and shared to ensure a common understanding of residual risks through the construction phase. These risks can then be managed using appropriate Processes managed by the right People however there remains the danger that commercial pressures can lead to decisions that elevate construction risk profiles. The commercial model and Alliance Principles inherent in an Alliance Agreement, while not substitutes for effective risk management practices, are considered to be effective catalysts in ensuring project risks are managed in accordance with the ALARP principle.

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DEVELOPING THE MIDTOWN TUNNEL PPP PROJECT

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Dusty Holcombe ■ Office of Transportation, Public-Private Partnership

ABSTRACT

After nearly 4 years in development, Financial Close for the Downtown Tunnel/Midtown Tunnel/Martin Luther King Expressway Extension PPP Project was achieved on April 13th, 2012. This paper describes the procurement and development stages of this $2.1 billion DBFOM project that included the detailed planning, preliminary design and costing of the first immersed concrete tunnel in Virginia. It describes the partnering approach taken by the client, the developer and the contractor in delivering this much-needed project for the people of Hampton Roads.

VIRGINIA'S PUBLIC PRIVATE TRANSPORTATION ACT

The Commonwealth of Virginia has had a long history dating back to the 17th century in development of the business environment including the original public-private venture with the Virginia Trading Company to first explore the Virginia coast. In its modern history, Virginia has utilized the Public Private Partnership Act (PPTA), to promote an "Open for Business" environment for private sector investment in transportation infrastructure. The focus of the legislation and Virginia's Public-Private Partnership (P3) program is to facilitate the identification, selection, development and procurement of complex transportation projects, promote private sector investment to leverage public funding and create a transparent environment for the development of projects that bring value to the citizens of the Commonwealth. Since its enactment in 1995 by the General Assembly, Virginia has received numerous unsolicited proposals and initiated several solicited projects. The program has successfully constructed nearly $2.5 billion of major transportation projects and has approximately $5 billion worth of active construction projects.

PROJECT BACKGROUND

One of the projects currently under construction in the southeastern portion of Virginia is the Downtown Tunnel/Midtown Tunnel/Martin Luther King Freeway (MLK) Extension, commonly referred to as the Midtown Tunnel Project. The Midtown Tunnel Project is being developed under a long term P3 between the Virginia Department of Transportation (the "Department") and Elizabeth River Crossings OpCo, LLC (ERC).

The concept of improving the Midtown Tunnel had been discussed in the Hampton Roads region for decades and had been included in the Metropolitan Planning Organization's (MPO) constrained long range plan since 2001; included in the 2002 transportation referendum for Hampton Roads; and in 2007 was part of the Hampton Roads Transportation Authority package of projects. With a history of being a priority project for the Hampton Roads region, the Department, in May 2008, advertised a Solicitation for Proposals (SFP) requesting firms to provide a proposal identifying their team's qualifications and experience related to the development of the Midtown Tunnel Project (Figure 1).

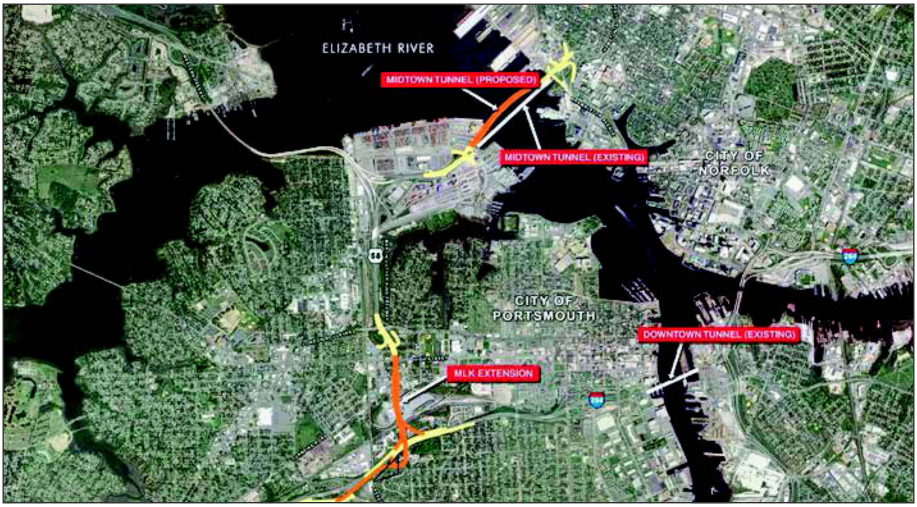

Figure 1. Project map

Additionally, in late 2010 the Hampton Roads Transportation Planning Organization (HRTPO) developed a *Prioritization of Transportation Projects–Project Evaluation and Scoring* study, which identified a prioritization methodology for transportation projects that was based on three components: (i) Project Utility, (ii) Project Viability, and (iii) Economic Viability. The HRTPO staff evaluated over 150 projects that were currently under consideration as part of their 2034 Constrained Long Range Plan and a score was provided for each of the noted project components. The Midtown Tunnel Project attained a score of 242 out of 300 and was the highest scoring out of all projects scored. This is significant because it meant that the Hampton Roads MPO and TPO rated the Midtown Tunnel Project the highest priority project in the region and provided additional creditability for advancing the procurement and development of the project.

PROJECT SCOPE

The scope of the Midtown Tunnel Project as defined in the SFP included the development, design, construction, operations, maintenance, and tolling of:

- the new Midtown Tunnel which is an immersed concrete tube tunnel approx. 1250 meters (4,100 feet) long that will run under the Elizabeth River between the cities of Norfolk and Portsmouth and be adjacent to the existing Midtown Tunnel;
- the MLK Extension, which is located in Portsmouth and is an extension of the existing Martin Luther King Freeway from its current terminus at London Boulevard to a new interchange at I-264 that will involve construction of approx 1311 meters (4,300 feet) of limited access elevate concrete structure;
- the rehabilitation of the existing Midtown Tunnel which is a single tube with two-lanes of bi-directional traffic and the existing Downtown Tunnels, which consist of two tubes and 4-lanes of traffic; and
- modifications to the interchange at Brambleton Avenue/Hampton Boulevard in the City of Norfolk.

PROJECT SCREENING AND DEVELOPMENT

Despite the initial enthusiasm expressed by the private sector about the project, the SFP was answered by only one private sector proposer. That team consisted of a joint venture with Skanska Infrastructure Development, Inc. and Macquarie Financial Holdings Limited as the lead firms. The joint venture was joined by a design-build team that had significant experience in the Hampton Roads construction market with Skanska USA Civil Southeast, Inc, Kiewit Construction Company and Weeks Marine, Inc. Together, these team members made up Elizabeth River Crossings and were led in the procurement by Chris Guthkelch.

As part of the procurement process, ERC participated in a series of Independent Review Panel (IRP) meetings, which is a panel of diverse transportation officials, local representatives and citizens that review the proposals and make recommendations to Virginia's Commonwealth Transportation Board (CTB) and the Commissioner of Highways on what can be improved about the project scope, proposals and which entities, if any, should be advanced to the next level of procurement.

The IRP and CTB's main recommendation was to advance ERC to the next stage of the procurement and because they were the only proposers on the project, to recommend to the Department to enter into negotiations for an Interim Agreement, which is allowed under the PPTA, so that the financial and technical feasibility of the Project could be better defined and project development activities in key areas of risk could be mitigated with additional studies and information. The Department and ERC entered into the Interim Agreement on January 7, 2010.

As part of the responsibilities of the parties to the Interim Agreements, both parties conducted several feasibility studies for both technical and financial project components, held public hearings on the design elements and completed additional analysis to define better the key risk features of the Midtown Tunnel Project. Once both parties agreed to the project's feasibility, the Department and ERC initiated negotiations of a Comprehensive Agreement. Those negotiations were completed in December 2011 with the execution of the Comprehensive Agreement; Financial Close was attained, including execution of the Transportation Infrastructure Finance Innovation Act (TIFIA) Loan, in April 2012.

PROJECT ORGANIZATION

Once the Interim Agreement had been executed, VDOT and ERC established a 3-tiered management structure to jointly manage the Development Stage of the Project to execution of the Comprehensive Agreement.

Key Principles

Key principles for ensuring effective management of this very complex stage were:

- Collaboration—enabled by setting up joint working groups for technical, commercial and financial work-streams under VDOT-ERC co-chairs.
- No Surprises—achieved through regular updating of Sponsors and Key Stakeholders.
- Empowerment—devolution of decision-making to the Project level best able to manage these.
- Common Goal—both sides recognized the critical importance of delivering a Project within tight parameters of affordability.
- Flexibility—it was recognized that the management structures had to be able to adapt as new challenges emerged during the Development Stage. For example, Context Sensitive Design became a critical issue for one of the

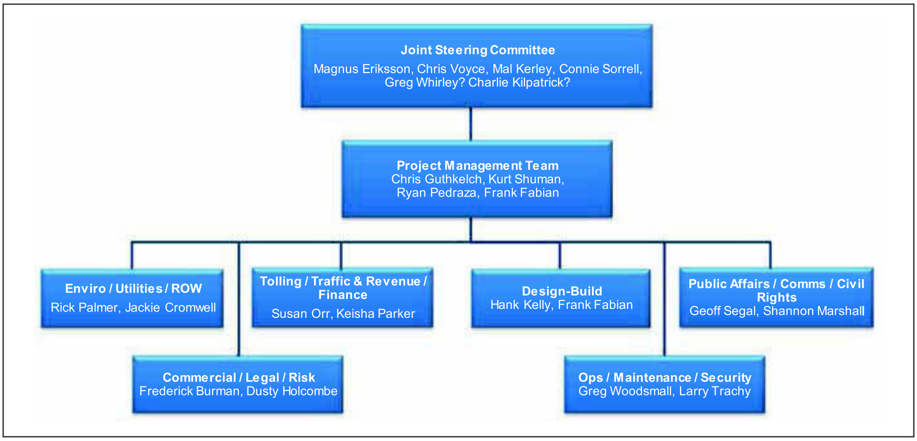

Figure 2. 3-tiered management structure diagram

localities and had to be managed carefully at a late stage of the preliminary design process.

3-Tiered Management Structure

The 3-tier Management Structure (Figure 2) comprised:

- A Steering Committee comprised of VDOT/ERC senior executives that provided guidance to the Project team and recommendations to the VDOT Commissioner of Highways and the Commonwealth Transportation Board.
- Project Management team comprised of the OTP3 Program Manager, ERC Project Managers and VDOT Hampton Roads District Project Manager.
- Joint Working Groups, each with VDOT/ERC co-chairs, covering:
  - Environmental, Utilities and Right of Way
  - Commercial/Legal/Risk
  - Financial/Tolling/Traffic and Revenue
  - Operations and Maintenance/Traffic Operations/Security
  - Design—Build
  - Public Affairs/Communications/Civil Rights

The Management Structure proved highly successful and enabled very close working relationships to be developed between VDOT and ERC, with a high level of trust and understanding of the other side's position being achieved.

Development Stage Work Plan

During Phase 1 of the Development Stage, ERC and its design-build contractor, SKW constructors developed a work plan comprising [224] separate work packages to enable delivery of a preliminary design covering all aspects of the proposed Project solution including mobilization, permitting, ROW/utilities, detailed design and construction schedule, operations and maintenance, tolling and handback at the end of the 58-year contract term. Cost of development was shared 50/50 between VDOT and ERC and the work plan was managed by a discrete team from VDOT, ERC and SKW who delivered a $32 million Work Plan on time and within budget.

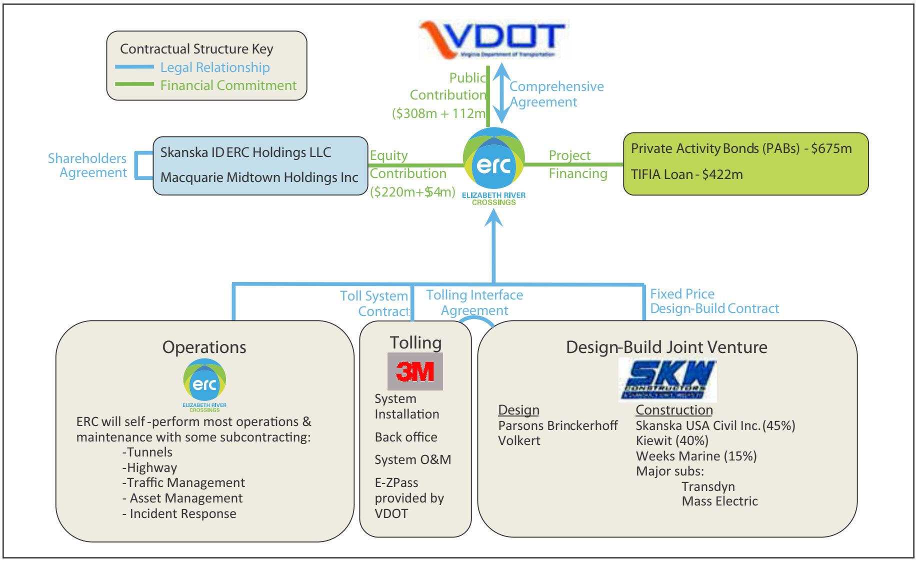

Figure 3. Contractual structure diagram

PROJECT SOLUTION

The Midtown Tunnel Project features an open road, all electronic tolling system and the tolling scheme employs congestion pricing as required under the Federal Highway Administration's Value Pricing Pilot Program, which is the mechanism that provided authority to toll the facilities. Tolls at the tunnels for cars will be $1.59 in the off peak and $1.84 in the peak (peak is defined as Monday through Friday from 5:30 a.m. to 9:00 a.m. and 2:30 p.m. to 7:00 p.m.). Tolls at the MLK Extension will be $1.00 ($0.50 if a trip includes both a tunnel and the MLK Extension). Tolling of the new facilities will begin following substantial completion of the new Midtown Tunnel which is currently scheduled for 2018.

Contractual Structure

The Project's contractual structure matched the usual arrangement expected for a PPP project, with the exception of additional parties needed for managing the more complicated funding arrangements arising from a Federal loan, State subsidy and private activity bonds (Figure 3).

Key Components

Skanska Infrastructure Development Inc and MIP II, a Macquarie Fund, provided equity in equal proportions and established a special purpose vehicle, ERC Opco LLC, for implementing the Comprehensive Agreement on behalf of the Client, VDOT, represented in Hampton Roads by its District Office.

On July 17, 2012 the existing Elizabeth River Tunnel staff transferred seamlessly from VDOT to the ERC Opco to provide construction management oversight, operations and maintenance, tolling operations and public outreach for the 58-year concession period (Figure 4).

SKW Contractors, through its Design-Build Agreement with ERC Opco LLC, will design and construct the new Midtown Tunnel and Martin Luther King Expressway Extension, as well as rehabilitating the existing Downtown and Midtown Tunnel facilities

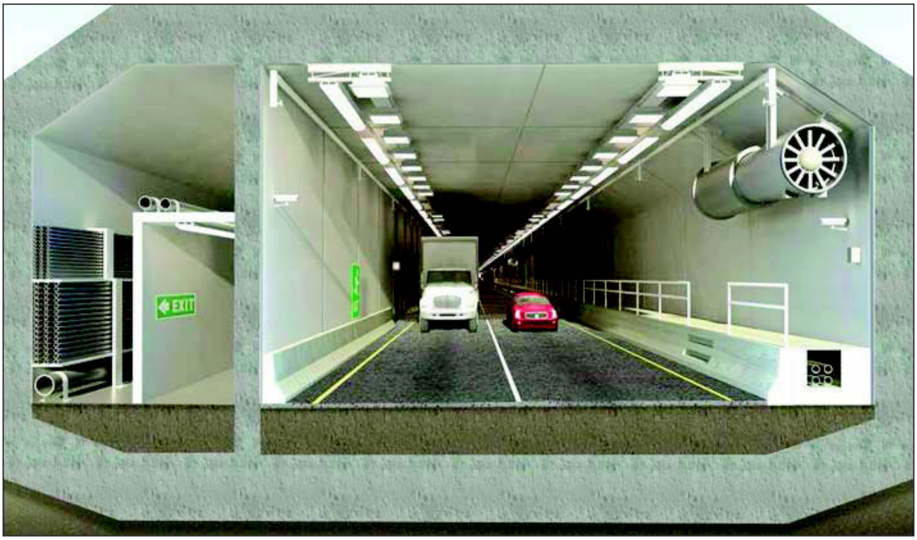

Figure 4. Tunnel cross section

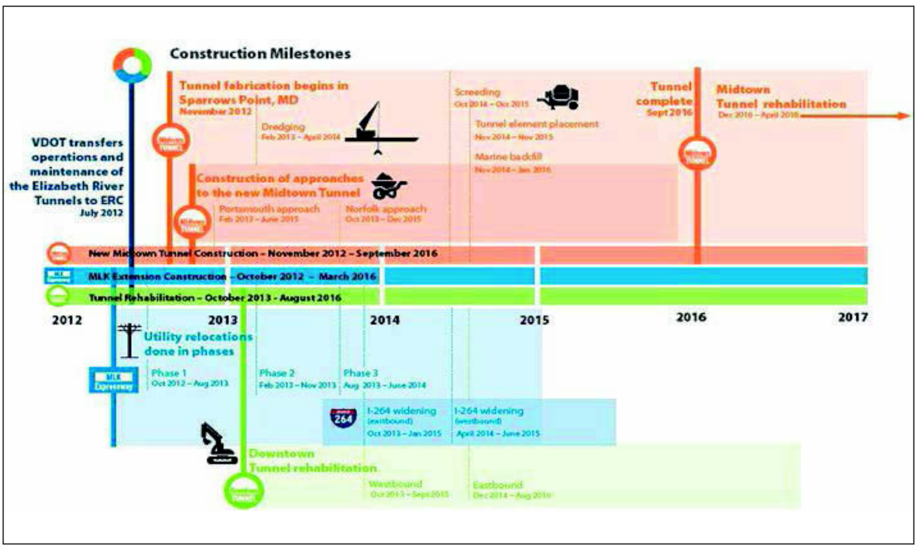

Figure 5. Outline construction schedule

and provision of the intelligent transportation system within the Project area. Its obligations under the Design-Build Agreement are undertaken on a lump-sum, fixed price, turnkey basis (Figure 5).

Federal Signal Technologies has contracted with ERC Opco LLC to design, install, test and commission the electronic tolling collection system for the Project and to operate and maintain the electronic tolling system during the tolling operations and maintenance period. Its design-build obligations are similarly provided under its Tolling Contract on a lump-sum, fixed price, turnkey basis.

Project Details at a Glance

Project details at a glance are as follows:

- New Midtown Tunnel and approaches: 6,550 feet long; 2-lane, 4,198 feet long portal-to-portal; reinforced concrete, divided single cell, immersed tube approximately 3,760 feet long laid in 11 segments, varying in length between 332 and 356 feet; segments to be constructed at Sparrows Point, MD and towed 200 miles to site; longitudinal ventilation using 19 jet fans; tunnel can cope with 100 MW fire.
- MLK Extension: 4,800 feet elevated roadway and ramps.
- Rehabilitation of existing Downtown and Midtown Tunnel facilities to NFPA 502 standards.
- Capex: $1.45 billion
- Construction and rehabilitation period: 67 months
- Dredged material: 1.5 million cubic yards
- All electronic tolling

Risk Allocation

All Project risks contained within the Comprehensive Agreement are flowed down, on a back-to-back basis, to the parties best able to manage them. This has enabled best value to be obtained by not driving large contingencies but also by allowing the best value funding terms to be obtained during the Financial Close stage of the Project.

Project Value and Funding

The total value of the Project is $2.1 billion and is funded through:

- Tolling revenue
- Issue of $675 million of Virginia Small Business Financing Authority Senior Lien Revenue Bonds
- $463 million TIFIA Loan
- $421 million VDOT Subsidy to reduce tolls
- $272 million private equity investment by Skanska ID and MIPII.

LESSONS LEARNED

Inevitably for a Project as complex as this and with a procurement and development period extending over several years, there are many lessons learned both at an individual and organizational level. All projects are unique but the Midtown Tunnel PPP Project had a number of challenges specific to its environment:

- ERC was the sole bidder.
- The Project procurement started with no preliminary design work having been done.
- Immersed concrete tunnel technology was new to Virginia and as VDOT had not built an immersed tunnel for many years, public sector experience was thin on the ground.
- Hampton Roads is a volatile political environment, due in part to the differing demographics of localities either side of the Elizabeth River.
- A change in Administration resulted in the disbandment of VDOT's Innovation Project Division which had been handling the procurement and the setting up

of the Office for Transportation Public Private Partnerships (OTP3) under a publically-recruited Director.

Lessons learned that can be applied to any other P3 project are as follows:

- Relationships Matter—there's a saying that the process doesn't fail, only the people. Early recognition of this by VDOT, ERC and the SKW teams resulted in an outstanding collaborative effort. The relationships formed during the Development Stage are already having a beneficial impact as the Project progresses into its Implementation stage.
- Expect the Unexpected—the longer the procurement and the more complex the Project challenges, the more likely that issues will emerge. Project development is deeply messy since so many stakeholders are involved and, if it's P3, the procurement and development is being conducted in the public eye.
- Partnering Works—both sides recognized that in order to partner, they had to be transparent and honest with each other. Each respected the other's point of view and frequent dialogue occasioned through the 3-tier management structure resulted in timely delivery of a value-for-money solution.
- Megaprojects Don't Happen in a Vacuum—the Key Stakeholder Diagram illustrates the number of public-private, business and non-business, internal and external stakeholders that needed to be managed during the Procurement and Development Stages. Effective Stakeholder Management at Federal, State and local levels proved to be vital to successful execution of the Comprehensive Agreement and Financial Close.
- P3 is Different—P3 projects require fully joined up technical, commercial and financial solutions that are interdependent on each other but deliver fixed price, date certain, turnkey solutions. It is vital that contracting parties understand the dynamics of this type of deal before getting into the negotiating room.
- Be Ready for the Long Haul—typically P3 procurement and development takes 3–4 years, with another 3–5 years of design and construction before handover to operations. While continuity is essential to maintain the developer team's knowledge and experience, managers must consider individual and team development, succession planning and enabling the team to pace itself.

FOLLOW ON ACTIONS

This section describes the follow on actions undertaken by the OTP3 after the Project achieved Financial Close.

Creation of the Office of Transportation Public-Private Partnership (OTP3) Office

The OTP3 was created during the active procurement and negotiations of the Midtown Tunnel Project. The creation of the OTP3, was in part generated by the need to have a centralized focused group within the Commonwealth that will actively identify, select, develop and procure the complex types of projects that are defined as P3's. The office was able to consolidate the financial, commercial, legal and technical resources to create continuity in the development of the contractual and technical documents. This allowed for a more productive negotiation session between the two parties and created an environment of greater competitive negotiations which ultimately lead to a more competitive total cost even though there was only one bidder for the Project.

Greater Transparency for PPTA Projects in Virginia

After achieving commercial and financial closure on the Midtown Tunnel, there was an outcry from the local residents and elected officials that they were unaware of the specifics of the Midtown Tunnel Project. While the OTP3, VDOT and ERC participated in numerous public hearings, briefing and meetings with local organizations, elected officials and citizens during the development of the Project, the OTP3 felt it would be beneficial to develop a *Public Involvement Guide for PPTA Projects* (the "Guide"). The Guide is currently under development and is expected to be available for public review and comment in early 2013. The Guide will identify the different public touch points during the identification, selection, development and procurement of a PPTA Project and provide guidance to both citizen and elected officials as to the level of transparency that will be provided on PPTA Projects.

Creation of a Risk Analysis Guidance Document

As part of the pre-procurement and procurement activities for the Midtown Tunnel Project, the VDOT conducted several risk workshops to identify, qualify and quantify the numerous risks associated with the development, procurement, construction and operation of the Midtown Tunnel. Because there was only one bidder advance to the negotiation stage of the procurement, VDOT and ERC representatives held a joint risk workshop. This was an essential two-day meeting facilitated by an independent party that identified the key risk elements of the Project from each party's perspective, potential mitigation actions that could be taken prior to signing the Comprehensive Agreement and open discussions on who is best able to manage the risk element.

GBR—TO USE OR NOT TO USE?

Andy Thompson ▪ Hatch Mott MacDonald

ABSTRACT

The Geotechnical Baseline Report (GBR) is becoming a standard tool for delineating the baseline conditions for the purpose of risk allocation and management. However a GBR is not the only tool that can be used for this purpose. This paper explores these alternatives, with examples from completed projects, and attempts to identify whether there are common items that need to be incorporated into a contract, irrespective of the baseline form used, to aid the successful implementation of the chosen method.

INTRODUCTION

The construction of underground infrastructure occurs in conditions that are unlikely to be fully understood at the time of Bid. Encountering ground conditions during construction that are considerably different than those anticipated during the project design phase, is a significant possibility. These events can and do give rise to significant impacts on the ability of the project teams to deliver the project on time and within budget. The effective identification, assessment and management of the risks associated with changed ground conditions are therefore critical to the successful management of the Project.

The use and establishment of some form of baseline is increasingly recognized as a means of allocating the risk associated with ground conditions. One of the common tools for identifying this baseline is through the use of a Geotechnical Baseline Report (GBR). It is the author's contention that there are other methods that can be used to identify the baseline conditions and that there are other critical elements that need to be incorporated into the contractual framework to properly manage changed ground conditions and achieve a project outcome that satisfies all the relevant stakeholders.

Examples from recent tunnel projects will be used to demonstrate how these strategies can be used and how project outcomes may be affected by the strategies adopted.

RISKY NATURE OF UNDERGROUND CONSTRUCTION

Almost uniquely in the construction industry, the creation of underground space occurs in an environment where there is no certainty about the conditions to be encountered, which may inevitably affect the final outcome of the project. For example in the mid-1980s US completed tunnel costs frequently exceeded bid prices by more than 25%. Although the industry track record has improved recently significant cost overruns continue to occur. In a study of 258 transportation projects worldwide undertaken by Flyvebjerg et al. (2003) the average cost increases for fixed link projects including tunnels was 34% and that 9 of 10 projects suffered from cost increases. Given the costs involved in tunnel construction projects a cost overrun of this magnitude can and will have significant impacts on the project stakeholders. For example the Channel Tunnel project came in 80% over budget, the Storebaelt Tunnel a 54% cost overrun and the Jubilee Line Extension in London, 67%.

The figures quoted above are global cost overrun figures and it must be recognized that not all the cost and schedule overruns are due to changed ground conditions.

Ground conditions however, exercise the minds of the industry due to the fact that uniquely at the start of the project the physical conditions that will be encountered during construction are not precisely known and that the risks associated with unanticipated subsurface conditions may represent the greatest risks to the project from a cost and schedule perspective. These costs will at some stage have to be apportioned between the relevant stakeholders. As noted by Salvucci (2003) “delay is the most significant driver of cost increase and reduced project benefits.”

Cost or schedule overruns of this magnitude have sparked various initiatives to improve project delivery methods, so that Clients will have an increased expectation that the final outcome price will be as close as possible to the bid price.

As noted in the International Tunneling Association’s Recommendations on contractual sharing of risks (ITA, 1998), the management of the risks associated with changed ground conditions is a major element in achieving a project outcome that satisfies all the relevant project participants.

In order to improve cost certainty the risks associated with the ground and especially changed ground conditions are realistically considered and suitably managed from the outset. So how to do this?

RISK MANAGEMENT

What is risk? The Institute of Risk Management (IRM, 2002) defines risk as “the combination of the probability of an event and its consequence.” Smith (1999) states that risk falls into three categories known risk, known unknowns and unknown unknowns with the unknown unknowns being events that cannot possibly be foreseen.

Applying this to underground construction it is obvious that tunnel construction will always involve a lot of unknown factors that neither the contractor nor the employer can be absolutely sure of and these can affect the project outcome in terms of time, money and quality.

To develop the risk management strategy the following process is typically undertaken: (1) Identify the risk sources, (2) quantify the risk, (3) develop management responses and (4) make provisions for residual risk.

The results of this process should be a comprehensive risk register that identifies all risks as well as indicate how the risks are to be managed, their potential severity from a cost, schedule and quality perspective and to whom they should be allocated. Increasingly the risk register is being used throughout the project lifecycle rather than purely as a design and planning tool and should wherever possible be carried through into the operations phase as a living document.

This formalized risk management process was captured in the UK, by the “Joint code of Practice for the Procurement, Design and Construction of Tunnels and Associated Underground Structures” published in 2003 by the BTS. While not universally accepted the risk management template provided in this document is a useful starting point.

RISK ALLOCATION

There are four approaches to the allocation of project risk; ignore, transfer, share or assume it. The method chosen will depends on the Client’s risk tolerance with most clients being risk averse and will be reflected through the procurement method chosen. Whichever approach is taken it must be recognized that there is a cost associated with each potential risk, which even if not explicitly assigned is borne by either the owner or the contractor through the contract. As such a payment mechanism must be included in contracts to allow a contractor to price the risks he is responsible for and ensure that the Client receives a realistic cost proposal.

The apportionment of risk need not be fair as long as it is clear. In the UK this was highlighted in the decision of Stent v Gleeson in August 2000, where Judge Bowsher concluded:

> *"In all projects, the allocation of the risks of negligence and the duty to insure against those risks is a matter to be considered. Clear allocation of risk may reduce the likelihood of litigation or arbitration. The decisions of the courts, including this decision, should not be seen as being opposed to such allocation of risk. All that is being decided in this case, as in others, is that the parties should be clear and explicit in their contracts so that parties start a project with clear knowledge as to where the risks lie rather than disputing the allocation of risk when the project goes awry. There is so much guidance in the decided cases on this topic that it would be easy for any lawyer for a contracting party to draft clear words excluding liability, if that is what his client wants, and the other party could then decide with informed consent whether he wants to accept that exclusion*" (Lovells, 2000).

It is not the apportionment of risk but rather that the apportionment is clearly stated that is a key factor in determining how successfully the risk will be managed and its effect on the project outcome.

INTERNATIONAL TUNNELING ASSOCIATION RECOMMENDATIONS ON CONTRACTUAL SHARING OF RISKS

The first guidelines were published by the ITA in 1978 and have been updated on a regular basis. A number of specific guidelines have been identified with the ones most relevant to this discussion summarized below:

Changed Conditions Clause: Should be included to induce contractors to avoid including large contingencies in their bids for dealing with unforeseen ground conditions.

Full Disclosure of Available Subsurface Information: All information, both factual and interpretive, should be fully disclosed to contractors at bid time to assist in determining whether a changed condition subsequently exists.

Ground Support: Contract documents should define

- The assumed character of the ground
- Parameters for the design of ground support
- Bills of quantities for ground support cover a reasonable range of site conditions
- Methods to take account of changes in quantity of ground support, dictated by actual site conditions when they differ from those assumed.

Ground Characterisation:

- Definitions of the ground characteristics
- Estimate of the extent and occurrence of each characteristic, as a uniform basis for bids.
- Procedural provision for how the owner and the contractor agree to changes as a result of encountering actual site conditions differing from those understood to exist at the time of tender.

These reinforce the need to share, allocate, or assume risk but never ignore it.

GEOTECHNICAL BASELINE REPORTS (GBRs)

One of the prime elements of the ITA's recommendations is that the Client clearly states the ground conditions assumed as the basis for preparing the Bid and Contract

Documents. Although no specific direction is provided by the ITA a GBR would provide this.

The "Joint Code of Practice for Risk Management of Tunnel Works in the UK" makes the use of a GBR is mandatory to "*provide the basis for comparison with ground conditions encountered in relation to those assumed and allowed for at the bid stage by the Contractor*."

The GBR provides a framework that enables differing conditions to be determined and the associated Differing Site Conditions (DSC) clause relieves the contractor of assuming the risk of encountering conditions differing from those indicated and provides a method within the construction contract so it can be handled as an item of contract administration.

It is usually assumed that risk associated with conditions consistent with or less adverse than the baseline are allocated to the contractor, and those significantly more than the baseline are accepted by the owner. As Essex (1997) notes the baseline is a contractual baseline and may not represent geotechnical fact but is dependent upon the level of risk and price certainty that the owner wishes to accept. It reflects whether the costs of differing site conditions should be allocated to the contractor and built into the construction cost (leading to higher bid price) or whether those risks and costs should be addressed by the change order process (leading to a lower bid price but less certain outcome cost). The GBR has proven to be a valuable risk management tool and a second edition was published in 2007.

This author believes however that the GBR as its is currently used is not necessarily the best means of allocating the risk on all projects and without consideration of the other factors needed to properly allocate risk may actually be counterproductive. Examples from recent projects are now considered.

GREATER ISTANBUL WATER SUPPLY SYSTEM, ISTANBUL, TURKEY

A 100 mile pipeline bringing water from the Melen River on the Asian Side of the Bosphorus and delivering it to existing waster treatment facilities on the European Side. 20 miles of tunnel were constructed, seven using drill and blast with the eighth tunnel crossing under the Bosphorus Channel constructed by TBM.

Procurement Strategy: Design-bid-build awarded to the low bidder.

Contractual Provisions: Based on FIDIC with changes for Turkish Law. Changed ground conditions clause and a mechanism for payment were included.

Baseline: No interpretative geotechnical document was provided, only the Geotechnical Data Report.

To provide a baseline for bidding and for payments the expected ground conditions were split into 6 different classes based on an analysis of the expected "Q" values, and each class had a specific support design. Separate items are included in the Bill of Quantities for the support classes, together with individual support elements to supplement the design, so that each excavation and support type can be priced. Support classes are indicated on the Drawings as a % of each tunnel length, although no specific indications of where along the tunnel drive the different classes would be expected. No details of the calculations or design criteria used by the Engineer in identifying the classes and the percentage probabilities were provided. This Contract Drawings therefore indicated the baseline conditions.

Risk Management: Although no formal risk assessment was used the contract clearly showed where the risk lay through the use of re-measurable bill items for payment related to six predetermined rock types. The Client, therefore, carried all risk related to variations in the estimated quantities and for any delays and increased costs that could accrue.

Changed Conditions: After each blast the contractor's geotechnical engineers mapped the exposed rock and calculated the "Q" value. The Engineer would then review the contractors assessment and agree or otherwise with any additional support measures indicated. The actual ground conditions were worse than expected leading to increased quantities of the most expensive support being used, which also caused delays. However the predetermined support types were suitable for the ground conditions although at times additional support elements were required and paid for.

Payments for excavating and supporting the ground conditions encountered were made based on the agreed records although manipulation of factors in determining the "Q" value did cause minor disagreements. The changes in quantities of the various ground types were dealt with using the stipulated contractual mechanisms. At the site level this ensured that the focus was on solving problems rather than arguing about payment.

Summary: It is considered that the absence of a GBR on this project did not affect the contractual management of changed ground conditions as an alternative form of baseline was provided and the various contractual tools identified were provided.

MERSEY KINGSWAY TUNNEL UPGRADE, LIVERPOOL, ENGLAND

This project consisted of the construction of three new cross passages between the twin tubes of the Mersey Kingsway Tunnel undertaken on a nighttime possession basis when one of the tubes was closed to allow access to the cross passage locations.

Procurement Strategy: Design-bid-build utilizing a target cost contract and partnering. Split into two distinct Stages, Stage 1 saw the Contractor undertake site investigation prior to the final design being finalized with Stage 2 being the actual construction. This was necessary due to the constraints imposed by the nighttime possession requirements.

Contractual Provisions: Engineering and Construction Contract, Option C Target Cost with activity schedule. Changed ground conditions were dealt with as a compensation event with the changed ground conditions assessed against the site investigation results and the design criteria developed from them. Once accepted as a valid change the Contractor would submit his valuation and this would be added to the target cost. Actual costs were then paid.

Baseline: No geotechnical baseline document was used but this was mitigated by the fact that the contractor undertook the site investigation and interpretation to agree the geotechnical parameters to be used in conjunction with the engineer, prior to the final designs being prepared. Given that the permanent and temporary works designs were developed on the basis of the investigation results, the ground conditions had effectively been baselined.

Risk Management: A formal risk assessment and risk register methodology was used.. The risk related to changed ground conditions was clearly allocated between the Client and the Contractor. The use of the two-stage strategy allowed the Contractor to refine his methods and pricing prior to the start of construction.

Changed Conditions: Ground conditions encountered were not significantly different than those envisaged prior to the start of Stage 2 of the contract. As a result of the probe drilling undertaken during Stage 1 holes making significant amounts of groundwater were grouted to refusal. During construction it was found that this grouting had successfully sealed the groundwater inflow routes leading to stable conditions during tunneling.

Summary: The approach utilized ensured that ground condition risk allocation was clear despite the absence of a GBR.

Early contractor involvement and the use of a formal risk assessment process ensured the client had a realistic understanding of the risk and allowed the project team to effectively manage those risk leading to a high degree of cost certainty.

WATER TUNNEL, NEW JERSEY, USA

This project involved sinking two 45ft. deep shafts together with mining 1200ft of 10ft diameter tunnel beneath a canal and a river with minimum cover to the riverbed of 20ft.

Procurement Strategy: A two-stage target cost procurement. Once a preferred bidder had been established a constructability and value engineering review of the outline design was undertaken. During this phase the GBR was finalized. Once this was agreed the owner, engineer and contractor agreed the revised target cost and schedule based on the finalized design and GBR to enable construction to commence.

Contractual Provisions: The Institute of Chemical Engineers Green Book, target cost contract modified to comply with local regulations. The contractor was to be reimbursed for his actual verified costs and a pain/gain formula was agreed which included a cap on the amount the contractor would be responsible for should the target be exceeded. A changed ground condition clause was included.

Baseline: At bid time an outline GBR had been prepared by the Engineer. After Contract award and as part of the first stage constructability review the Contractor requested that an additional three boreholes be drilled. The results were then incorporated into both the Geotechnical Data report and the GBR. The final GBR was developed jointly between the Engineer and the Contractor and defined the basis on which the Contractor priced the final design and allocated the risk.

Risk Management: A risk register was developed that allowed the client to make an informed decision, concerning the tunnel option and his choice of procurement strategy. It was also used as a tool to determine tunneling methods and in finalizing the tunnel alignment and final designs. The original risk allocation indicated in the GBR was rendered irrelevant when the client later took responsibility for all risk.

Changed Ground Conditions: Immediately after mining started the TBM struggled with the encountered ground conditions. Higher than expected water inflows caused the shale to plasticize leading to changes being required to the cutterhead tool configuration. The changed ground conditions were addressed as per the contract procedures with the actual conditions compared to those in the GBR. The contractor and the engineer brought in independent experts who separately concluded that whilst the rock was behaving differently than expected, the rock types were as per the GBR.

At this point the Client made a decision that the Project had to be completed by a specific date and thereby reallocated all risk to himself. At this stage the contract ceased to be a target cost contract and essentially became a cost plus contract. The client paid the Contractors actual costs irrespective of the original contract intent.

Summary: The GBR established the contract baseline and the changed conditions were identified using established process in the Contract.

The Client's actions in reallocating risk during the contract negated the risk assessment findings that had influenced the procurement strategy. An example of how Clients actions can affect the outcome irrespective of planning etc.

SLURRY TBM PROJECT, USA

This project involved the construction of 10,000ft of 23ft diameter segmentally lined tunnel in mixed glacial till, fill and rock in an urbanized environment using a slurry TBM.

Procurement Strategy: A negotiated contract with prequalification. The main thrust of the negotiations was the allocation of risk during the TBM mining phase.

Contractual Provisions: The Contract used was a standard agency style contract with specific provision added as a result of the negotiations.

Baseline: A GBR was not used. A Geotechnical Interpretive Report (GIR) was included that described the assumptions at time of design and the parameters used by the Engineer to develop the final designs. Given that the ground conditions encountered during mining with a pressurized face TBM cannot realistically be measured or recorded the Contract established a baseline based on TBM operational cycles and parameters, advance rates, cutterhead tool replacement rates etc, that were to be submitted by the Contractor as back up to the Baseline Schedule. Deviations from the operational parameters etc. would be the first indication of ground conditions adversely affecting the TBM. These parameters and Contract wording were negotiated prior to Contract award. A capped amount of hours was included for cutterhead interventions and a very detailed set of parameters agreed to for the decision-making process related to need for and duration of any cutterhead intervention. Crew hour rates were agreed for hours in excess of this as well as a daily impact cost should the hour pool be exceeded.

Risk Management: A risk register was used and risks were clearly allocated through inclusion of separate bid items for soil, mixed soil and rock and pure rock mining in addition to the other contract provisions used to establish the baseline.

Changed Ground Conditions: No changed ground conditions issues were encountered that were not dealt with in a straightforward manner through the various contract provisions. It should be noted that all TBM performance records were available to the Client and were compared against the calculated values, which had been submitted and approved by the Client prior to mining commencing.

Summary: The absence of a GBR was not a factor in the outcome of this project. Indeed for pressurized face TBM projects the author questions the efficacy of using a GBR that cannot realistically be measured against due to the lack of access to the cutterhead. In the authors view the approach to be adopted for similar TBM projects needs to be established as a result of a detailed risk management exercise such that all parties understand their potential exposure.

The approach adopted, effectively baselined and provided a framework for the proper management of risk and the project outcome was satisfactory for both the Client and Contractor.

CONCLUSIONS

The uncertainty of tunneling and challenges provided by the ground conditions provides a risky environment, which if managed poorly can lead to delay and increased costs. A GBR is only one of the methods to establish the baseline and in some cases is not necessarily the best.

The author believes that the items below should be addressed to properly manage the risk associated with changed ground conditions:

- A documented risk assessment and risk register that lives throughout the feasibility, design, construction and operation of the project.
- This risk assessment shall be used to ensure the client appreciates the risks he will be required to allow for and assist in determining risk allocation and procurement strategy.
- Risk should be allocated to the party best able to deal with the consequences. For changed ground conditions this will commonly be the client. However it is divided the allocation of risk must be clearly stated and a tool provided to enable this to be established.
- Ground condition risk should be allocated through the use of a baseline that clearly defines this allocation. This can be achieved using a GBR or other suitable alternative and be decided on a project specific case.

- A changed ground conditions clause and associated payment mechanism must be included.
- Insurance should not be considered as the sole mitigation measure in risk assessment for tunnel works.
- Any procurement strategy/contract can be used provided they allow for the above.
- The use of a collaborative project environment strategy such as partnering is encouraged but is not considered essential.

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SUBSURFACE PUBLIC-PRIVATE PARTNERSHIP PROJECTS: BRAVE NEW WORLD OF RISK ALLOCATION

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ABSTRACT

For owners, Public-private partnerships (PPP) represent an opportunity to not only secure private funding for needed infrastructure projects, but also an opportunity to transfer significant risk to the private sector, especially the Contractor and its Engineer. The selection of the Concessionaire is influenced by its appetite and ability to accept risk, including significant subsurface condition risk. The Financier will evaluate whether the project is "bankable" based on its own assessment of risk assumption and allocation. Both the Concessionaire and the Financier seek to transfer significant risk to the Contractor and its Engineer.

Over the last four decades, the underground design and construction industry has made significant progress in developing improved contracting practices that promote fairness in risk allocation. These practices do and should apply in the PPP context. This paper addresses the subjects of risk in major subsurface PPPs and provides recommendations for fair and efficient risk allocation.

INTRODUCTION

Public-private partnerships ("PPPs") are gaining in popularity at the federal, state and local levels as the public sector seeks private capital to finance infrastructure and other projects which traditionally had been funded with public monies. In general, a PPP involves a Development Agreement between a Public Owner and a private sector Concessionaire. Under the Development Agreement, the Concessionaire is responsible for financing, designing, constructing and (typically) operating and maintaining the completed project for a concession period (often multi-decades in duration). The Concessionaire typically enters into an agreement—the Design-Build Agreement—with a Design-Builder to design and construct the project. The Concessionaire may secure financing from banks, investors or from a combination of these and other sources (collectively the "Financier"). While the PPP experience in the United States has been modest to date, an increasing number of projects involving tunneling or other significant subsurface work are already being delivered in the PPP mode.

Public Owners have significant options to address risk allocation for subsurface conditions on PPP projects. The same sound and fundamental principles of fairness in risk allocation that apply in other major subsurface projects should be applied and adopted in the PPP context.

SUBSURFACE CONDITIONS RISK ALLOCATION

The North American heavy construction industry has learned a number of fundamental risk management lessons over the last 30 to 40 years of underground construction practice. One key lesson is that risks associated with subsurface conditions on underground construction projects are significantly different, and much more difficult to pre-determine, than risks associated with surface construction. This is due, primarily, to the inability to predict the nature of the subsurface conditions and ground behavior prior to the actual construction. Over the years, attempts by Owners to unilaterally transfer subsurface risks to the Contractor have been met with commercial, financial, and political fallout for all parties involved. Owners have failed to have their facilities delivered on time or within budget; Contractors have lost money, their bonding capacity, or their existence altogether; engineering consultants have faced financial exposure because their professional service agreements expect them to be able to “see” underground; and insurers have suffered losses when their contractor and engineering clients could not financially absorb the financial risks. To improve its worsening health and reputation, the tunneling industry developed a number of improved contracting philosophies and practices to cope with these unique risk challenges. These principles have been set out in publications by ASCE’s Underground Technology Research Council, and have been endorsed on an international level by a number of British and international publications (see References at the end of this paper).

Improved Contracting Practices for Subsurface Construction

Central to fair risk allocation is that the Owner ultimately “owns the ground” through which it wants its facility built. This responsibility comes with a number of distinct elements. Firstly owners are responsible for carrying out a thorough site exploration program, and documenting the results in a Geotechnical Data Report (GDR) that is included in the Contract. Secondly, the Owner prepares an interpretation of the anticipated conditions and how those conditions will influence the construction. This interpretation is presented in a Geotechnical Baseline Report (GBR), also included in the Contract. The Contractor is responsible for the construction-related risks associated with the conditions presented in the GBR, but is not required to carry contingencies in its bid for conditions more adverse than those presented in the GBR. If the Contractor encounters conditions that differ materially from and are more adverse than those presented in the GBR, it is entitled to additional compensation attributable to the differences.

A number of other improved contracting practices have also been developed, such as Escrow Bid Documents and Dispute Review Boards, to aid the parties in meeting their contractual obligations under this risk management approach. The objective of these practices is to provide efficient, expeditious resolution of disputes that may arise during the course of the construction, thereby avoiding costly litigation and protracted dispute resolution processes using other dispute resolution techniques.

Nuances have evolved over the years relative to format and content for these different provisions, to reflect differences in the contracting methods, e.g., Design-Bid-Build (DBB) vs. Design-Build (DB). One nuance that will be discussed here is with regard to how the GBR is prepared.

Geotechnical Baseline Reports

Having a GBR in the Contract serves two primary objectives: to clearly allocate risks to the Contractor for anticipated subsurface conditions; and to provide a basis by which the Contractor may obtain additional compensation if it encounters unanticipated subsurface conditions. This is carried out in conjunction with a Differing Site Conditions (DSC) clause in the Contract. If there is no DSC clause, or if the Owner does not intend to provide additional compensation for unanticipated conditions, the reasoning for inclusion of a GBR in the Contract should be questioned. Assuming that the Owner does intend to share risks as recommended by the improved contracting practices, there are a number of different ways that a GBR can be developed, depending on the form of contract used.

For DBB contracts, the Owner prepares a 100% design (through its engineering consultant). The design may preclude the use of certain equipment and construction approaches, such as a non-pressurized face TBM or shaft supports consisting of dewatering with soldier piles and lagging. In this instance, the GBR is fully developed by the Owner in a unilateral fashion, taking into consideration the Owner's design and construction constraints and preferences. There is no opportunity for pre-construction bidder input other than through the standard Request for Information process.

Modifications to this approach can and should be implemented when the work is to be delivered using a DB approach. The same modifications are appropriate whether the project is being financed by the Owner (the DB approach) or by a private financier/concessionaire (the PPP approach). The key is that in both instances, someone other than the Owner is responsible for completing the design, not just the construction.

Whether DB or PPP, the Owner will typically prepare a preliminary design as a reference design for bidding purposes, but the entire design and selection of construction means and methods will lie substantially with the competing Contractor/Engineer teams, not the Owner. In either instance, the Owner may impose the same limitations on design and construction approaches in its reference design and bid package. Relative to the GBR, it is recommended [ref. 2007 ASCE Guidelines for Geotechnical Baseline Reports for Construction] in this instance that the Owner follows a collaborative rather than unilateral approach to finalizing the GBR as follows:

- The Owner prepares an initial version of the GBR for bidding purposes that presents baselines of the relevant physical conditions to be encountered at the site. These physical conditions are independent of the different bidders' designs. The Owner's GBR for bidding would leave gaps in the text, with guidance to the bidders on information that is to be provided in those gaps.
- In association with their bid preparation, each bidding team would answer the questions posed in the initial GBR, including their interpretation of the relevant physical conditions and behaviors consistent with their design and construction approach. This would include explanations for how specific risks are to be addressed during the work.
- In its evaluation of the different proposals, the Owner has the latitude to question or seek clarification of certain statements or assumptions contained in each bidder's GBR responses. There might be a difference of opinion about the relative risks to be addressed, or it might just be a matter of clarifying a bidder's position on a certain approach. In either case, the Owner would have the opportunity to discuss differences with each bidder to a point of mutual acceptance.
- Upon completing its review of the bidders' responses, and obtaining revised documents to reflect a common understanding of the matters at issue (sometimes referred to as a "cure" period) the Owner could then solicit financial bids for the project. In this manner, the Owner would be in a better position to make an "apples to apples" comparison of the bids.

RISK ALLOCATION FOR SUBSURFACE CONDITIONS ON PPP CONTRACTS

Risk Allocation Perspectives and Dynamics in PPPs

Most Public Owners resort to the PPP approach because they lack sufficient funds to undertake the cost of designing and constructing a desired or needed project. As a general matter, many Public Owners perceive their objectives as best achieved by

(a) minimizing retention and (b) maximizing transfer or allocation to the Concessionaire, of risk typically borne by the Public Owner. Put another way, the Public Owner seeks to significantly contain its risk exposure for cost overruns or schedule delays due to design and construction defects, events or conditions, such as the encountering of differing or unanticipated subsurface conditions ("DSCs").

The Concessionaire may be willing to assume risk (such as for DSCs) as a component of its base compensation. Presumably, the Concessionaire's ability to price that risk derives from and should depend upon an adequate degree of reliable subsurface investigation available at tender. The Concessionaire will plan to transfer down to the Design-Builder, on a back-to-back basis, risk (including subsurface conditions risk) that it assumes in the Development Agreement.

The Financier will conduct some degree of due diligence to satisfy itself that the Concessionaire is not assuming imprudent levels of risk that may impair the Concessionaire's ability to complete the project on time and budget. The source of payment of the Concessionaire's loan obligation to the Financier typically comes from a revenue stream from the completed project. As such, the Financier will want to gain a reasonable degree of confidence that the Concessionaire will "get to the finish line" and that the Concessionaire's acceptance of imprudent types or degrees of risk do not unduly imperil or jeopardize that objective. The Financier will also be keen to insure that the Concessionaire passes its design and construction risk downstream to the Design-Builder.

The Public Owner, Concessionaire and Financier perspectives on risk allocation and their aversion to cost overrun exposure together combine to exert substantial downstream risk transfer pressure to the Design-Builder.

PPP Characteristics Influencing Subsurface Conditions Risk Allocation

The previously-discussed risk allocation perspectives and dynamics in PPPs present a challenging environment within which fair and balanced subsurface conditions risk allocation can exist, especially for the Design-Builder. Similar dynamics exist where Public Owners seek a DB delivery approach—they are increasingly adopting aggressive subsurface conditions risk allocation approaches on DB projects that would not be deemed acceptable by the industry under a DBB approach.

Beyond the foregoing considerations, there are distinguishing characteristics of PPPs that need to be taken into account relative to subsurface conditions risk allocation.

First, because PPPs are authorized by special enabling legislation, Public Owners often are exempt from otherwise governing risk allocation approaches (such as statutorily-mandated inclusion of differing site conditions provisions). As such, Public Owners in PPPs generally have a broader range of discretion and judgment relative to risk allocation approaches than in more conventional delivery approaches. This can be good or bad depending upon the risk allocation decisions that are made.

Second, Public Owners, Concessionaires and Financiers—all keenly aware of the need to maintain cost and schedule control—view DSCs as a potentially major source of cost overruns and schedule delays and, as such, will seek to contain that risk exposure through contractual provisions that ultimately transfer substantial conditions risk to the Design-Builder.

Third, the Public Owner in a PPP Project may not have the funds to commission an adequate subsurface investigation program prior to tender and/or may seek to eliminate any risk for subsurface conditions data or reports which it furnishes in a RFP by disclaiming the accuracy or completeness thereof and negating any right of the Concessionaire or Design-Builder to rely upon those materials.

Fourth, since most Public Owners in PPPs do not have sufficient funds for design and construction, there are likely to be no contingency funds available to address the economic consequences of DSCs identified during design or encountered during construction.

Fifth, many Public Owners in PPPs rationalize more aggressive subsurface conditions risk allocation to the Concessionaire and Design-Builder based upon the latter's responsibility for (a) defining the scope of and performing their own subsurface investigation program; and (b) the development and finalization of design and construction approaches consistent with anticipated subsurface conditions. Further, Public Owners reinforce their more aggressive risk allocation positions on the recognition that there is a direct correlation and interdependency between the character of anticipated subsurface conditions and the achievability of project design and contemplated means and methods (including equipment selections) to be utilized in the construction process.

APPLICATION OF IMPROVED CONTRACTING PRACTICES TO PPP PROJECTS

Improved contracting practices—including the hallmark and fundamental principle of fairness in risk allocation for subsurface conditions—should be applied and adapted in PPP projects.

There are several major PPP and DB projects presently in progress in which a wide range of risk allocation approaches to subsurface conditions have been adopted. These projects include:

- Port of Miami Tunnel (PPP)
- Virginia Midtown Tunnel (PPP)
- Evergreen LRT in Vancouver (DB)
- Ottawa LRT (PPP)
- Ohio River Bridges East End Project (PPP)
- Eglinton-Scarborough Crosstown LRT (PPP)
- Alaskan Way Viaduct Replacement Tunnel (DB)
- Niagara Tunnel (DB)
- Lake Mead No. 3 Intake Project (DB)
- Trans Hudson Express Tunnels (DB) *project terminated

Each of these projects has approached risk management aspects of subsurface conditions in a different manner:

- Use of a Geotechnical Data Report—some have included the document but through exculpatory language have denied the right to rely on the information as a basis for a DSC.
- Use of a Geotechnical Baseline Report—some have engaged bidders in a collaborative process (e.g., Niagara Tunnel), whiles others have solicited comments and suggested modifications from bidders in their proposals. Others have included a GBR purely as a means of allocating risks to the Designer-Builder, with no responses accepted and no DSC clause in the Contract. Some have sought to limit the risks of a DSC by limiting the baseline conditions to a narrow zone around the planned excavated opening, thereby disavowing the accuracy and behavioral influence of any ground beyond (above and below) the baselined zone. The latter approach appears to be the horizontal equivalent of limiting the accuracy of a borehole to the cylinder of ground through which it was drilled.

- Disclaimers that shed disproportionate risks to the Contractor or PPP team, either through words or odd baseline geometrics such as those described above.
- Use of Escrow Bid Documents (EBDs)—some have included EBDs, some have not.
- Use of Dispute Review Board (DRB)—some have included a DRB, others have exclude this or any other form of disputes resolution.
- Financial Risk Sharing—some projects have acknowledged that conditions more onerous than the baselines are compensable by the Owner, whereas others have asserted that all risks associated with unforeseen conditions are to the bidder's account. Several have adopted a ladder-rung form of financial responsibility, where an initial dollar volume is to the bidder's account, a second dollar volume is to the Owner's account, and amounts above those two are to be shared by the bidder and Owner according to a specified percentage split.

Project Owner decisions in these respects are having a material impact on subsurface conditions risk allocation—in some cases dispensing with risk sharing altogether.

CONCLUSIONS AND RECOMMENDATIONS

With the growing number of tunnel projects being delivered through DB and PPP contracts, the doctrine of fair contracting through an equitable sharing of risks associated with subsurface conditions is quickly being eroded. Project Owners are driving the tunneling industry in a backward direction by utilizing unfair and unbalanced risk allocation approaches. In several cases, we have returned to the "You bid it, you build it" standards of the 1970s.

The concern is that unanticipated subsurface conditions during construction will lead to delays and disputes that will find their way to the newspapers and industry magazines, and controversies will build once again relative to the ability of the tunneling industry to "deliver." That DB or PPP contracting methods are being utilized will be irrelevant to the impacts of bad press.

The authors recommend that organizations such as the Underground Construction Association of the Society of Mining Engineers (UCA of SME), ASCE's Construction Institute, and the International Tunnelling Association engage in dialogue with Project Owners, financiers, insurers, contractors and engineers to help steer the industry back to more appropriate means of managing risks associated with subsurface conditions on PPP (and DB) projects.

Those familiar with what happened to the Australian tunneling industry in the 1980s–1990s will appreciate what is in store for the US tunneling industry if the trend away from fair contracting practices continues.

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PART 3

Design and Planning

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A REVIEW OF PORTAL DESIGN CONCEPTS FOR MOUNTAIN TUNNELS

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ABSTRACT

Tunnels located in mountainous terrain have to enter and exit the mountainside through portals. A tunnel portal, being at the transition zone between surface and underground construction, must therefore consider the technical aspects of both slope stability and tunnel stability. Since the portal area is near the surface, the ground is usually more weathered, requiring more intense ground support than in deeper less weathered zones. This paper reviews the influence of topography, climate, rock type, and rock structure, upon tunnel portal stability and design. In addition it considers tunnel variables such as tunnel size, tunnel opening geometry, pillar width for parallel tunnels, as well as the risks of landslide, rockfall and avalanche at portals. Case histories are reported to illustrate practical portal design solutions for both initial and final ground support, for both the surface and underground aspects of the portal zone.

INTRODUCTION

The demand for new tunnels is growing each year, especially for transportation (roads, railways), water transport, and mining. Many of these tunnels are relatively long (extending several kilometers), and engineers devote much effort to the design of these tunnels in order to create the optimum balance between size, shape, function and cost.

Unless the tunnel is being driven from a shaft into an existing underground network, access will be via portals, especially in mountainous terrain. Portals usually represent a relatively small percentage of the overall tunnel length, so the cost of portal construction is often small compared to the cost of the main part of the tunnel. Sometimes little attention is paid to the design and construction of the portals unless a problem develops.

Portal costs are usually more expensive on a per meter basis than the cost of the main part of the tunnel. In addition, they often pose unexpected problems. These problems can cause serious delays in tunnel construction. Even after the tunnel is constructed, portal stability problems may affect tunnel usage and require ameliorative measures which may put the tunnel out of service for several months.

Too often the tunnel alignment is fixed before anyone takes a close look at the geology and site conditions at the portals. It is better, however, to consider several portal locations, conduct relevant site investigations, and choose the portal locations and tunnel alignment to take advantage of good ground conditions. This approach can minimize portal problems, as well as the overall cost of the project including any necessary remediation.

Tunnel portals vary greatly in size, geometry, engineering complexity and their cost of construction. Thus it is impossible to deal with all aspects of portal design in a single paper. Consequently, the authors start with an overview of tunnel failures and problems (Figure 1), and discuss the technical approaches to be used to optimize portal design, with an emphasis on portal stability and geotechnical design. References are made to

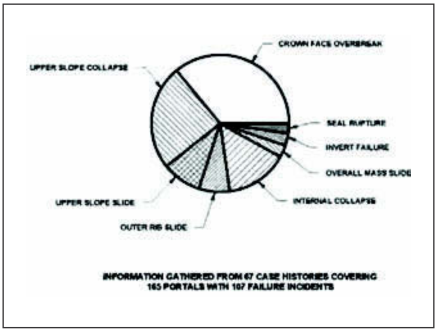

Figure 1, Analysis of portal failures (Rogers et al.)

Figure 2. Concrete canopy, bolts, and shotcrete (Madeira)

Figure 3. Rock blocks held with bolts (Madeira)

Figure 4. Sinnberg Portal at oblique angle (Courtesy of ILF Consulting, USA)

Figure 5. Box cut supported with shotcrete (Mexico)

Figure 6. Brow repaired with mass concrete (Mexico)

illustrative case histories. An extensive bibliography is included. Photographs illustrating different types of portal design and construction are shown in Figures 2–8.

SOME LESSONS FROM REVIEWING PORTAL FAILURES

It should be noted that portal failures are quite common. The history of civil engineering includes analyzing and understanding ‘failures’ that have occurred, and then improving

Figure 7. West Elk portal (USA)

Figure 8. West portals of Donnersbergtunnel (Austria) (Courtesy of ILF Consulting, USA)

design methods to try and prevent the same mode of failure from occurring again. This has been true for cathedrals, buildings, bridges, and dams. So too with tunnel portals; we can learn from past failures. The experiences of just two sets of authors are summarized below; they highlight the most common causes of failure and set the scene for a more general discussion of the approaches to be taken in design.

Portals in Hard Rock

Most portal failures are due to blocks or wedges of rock moving into the excavation. Rogers & Haycocks (1988) studied 166case histories, mostly associated with mine entries, and found 107 incidents of portal failure (64% incidence of failure). In 67 of these cases the authors were able to classify the main failure into one of two categories (internal to the tunnel, and external to the tunnel) and eight failure modes; their analysis is reproduced in Figure 1. Note that more than one-third of the failures were due to 'crown face overbreak' (i.e., collapse of the brow), and more than one-quarter were due to 'upper slope collapse.'

Their descriptions and sketches of these eight failure modes provide a good checklist for understanding portal design. Based on their experience, Rogers and Haycocks recommended a four-step sequence of investigation and analysis, which has been expanded by the authors as follows:

1. *Characterize the geology* in the immediate portal area. In structurally incompetent rock, this includes using standard rock classification systems such as RMR (Bienawski 1974, Bienawski 1989) and Q (Barton et al. 1974, Barton 2002). In structurally competent rock, this includes measuring the orientations of geologic weaknesses and discontinuities, and analyzing various types of sliding and toppling failure modes that could occur around the portal slopes.
2. *Check the stability of the overall slope*. This involves site investigation and slope stability analysis over the full height of the slope(s) using commercially-available software, including site conditions both before and after portal excavation is complete, and under the worst anticipated in-service groundwater conditions.
3. *Check the stability of the side slopes* of any box cut in front of the portal.
4. *Check carefully the stability of the first 50 m of tunnel*, using standard tunneling methods of design, and apply appropriate support.

Portals in Soil and Weak Rock

The portal failures discussed by Rothfuss et al. (1995) were civil engineering tunnels for hydroelectric projects and highways. All five of their main case histories involved overlying soils and weak rocks, and each case involved at least one slope failure. The contributing factors included weak zones, reactivation of an ancient landslide, high water table, period of high rainfall, inadequate dewatering and drainage, washout of the toe of the slope, inadequate support of the slope or tunnel, and support not applied in a timely manner. This makes a good checklist of problems to be aware of when designing portals, particularly in the presence of 'soft' cover.

A variety of remedial measures were needed to repair these portals. Among the lessons learned by Rothfuss et al. are the following:

1. Make a thorough geological and geotechnical investigation before and during construction of the tunnel portal.
2. Write clear specifications explaining the intent of the portal design elements and configuration, and the delineation of responsibilities, and maintain good communications with field and support team members during construction.
3. Observe the slopes carefully for anomalous behavior. Instrumentation can be very useful, provided that the data is reduced and made available in a timely manner to decision makers.

Needless to say, the remedial works in each case resulted in project delays (from one to three months), as well as additional costs. Of course it is easy to be wise after an event. Nevertheless, the main lesson to be learned is that many of these problems might have been avoided by better site investigation and initial design.

APPROACHES TO DESIGN

General Considerations

There are some considerations that apply generally to all portals, and it is as well to bear these in mind when designing and constructing any portal:

1. The portal needs to be designed to be stable for the life of the tunnel.
2. The portal is usually in rock (or soil) that is more weathered and fractured than the rock in the main part of the tunnel.
3. Usually the portal is within a slope that can collapse, sometimes triggered by the actual construction of the portal. Often the portal will unload the toe of the slope. So slope stability is a concern that must be addressed. Often this requires reinforcement of the slopes above and around the portal entrance.
4. Because portals are often located in soils and weathered rock, it is often necessary to approach the portal via a box cut (which also needs to be supported) so that the portal brow can be in located in relatively competent ground. One of the trade-offs in design is the determination of how far into the hillside the portal brow should be located.
5. Stress relief will weaken the rock mass immediately above the portal making it difficult to support the brow of the portal; it may also induce a slope failure which otherwise would not have happened.
6. In some situations the soil or rock mass will need to be reinforced by bolting or grouting from the surface prior to starting portal excavation.
7. Tunnel reinforcement is usually more intense in the vicinity of the portal, and progressing inwards until the effects of the portal excavation are no longer apparent and more competent ground conditions are encountered.

8. The portal will be excavated at the beginning of underground development before the crew has obtained much experience of working together and working in this particular geological environment.
9. Often construction techniques have to be modified to allow more gentle excavation in the portal area, including the avoidance of heavy blasting.
10. Seismic waves from earthquakes are amplified around portals, particularly in soils and soft rocks, and can trigger collapse of the portal and surrounding slopes.
11. Portals are open to the atmosphere and can be affected by the environment and weather patterns; issues are heavy rainfall, flooding, ice build-up, and fogging.
12. Sometimes there are visual problems for humans approaching a tunnel portal. This may require physical re-design to 'ease' drivers into and out of the tunnel.
13. Of special concern to high speed rail tunnels, is the configuration of the portal structure with respect to shock wave development from the speeding train.
14. Since the portal will be heavily used, a conservative design approach is needed because any instability in the portal area can adversely affect tunneling and/or mining operations, and could be costly in down time and remedial measures.

Given the various problems that can occur, it is wise to consider several portal locations in the planning stage. Each will require a preliminary site investigation followed by preliminary design, in order choose the best option. Then, a more thorough site investigation and engineering design can be done to finalize the design at the preferred location. In cases where a pre-existing highway or railway alignment exists, or in mountainous terrain where alignment options are limited, there may be less flexibility in portal site selection.

Since the portal provides access to the tunnel, the portal area has to be as stable as possible, since any future problems with the portal will likely lead to downtime and a serious loss of use. Hence, the authors' strategy is to provide redundant methods of support, with back-up systems ready to install should problems develop in the initial stages of portal and tunnel development.

Tunnel Size and Choice of Twin Tunnels

As a general rule, the larger the tunnel the more difficult and expensive it is to support. Typically, once the diameter exceeds about 5 m, the cost of the tunnel increases at about the square of the diameter. So there is a strong financial incentive to prevent tunnels becoming any larger than they really need to be, providing that they fulfill their function and meet safety standards. For this reason two smaller tunnels are often preferred to one large-diameter tunnel, and this applies also to the portals. A trade-off study is often needed to evaluate the one versus two tunnels option for a tunnel system requiring two traffic lanes in each direction.

When twin tunnels are utilized for two-way highway traffic in a mountainous environment, or when a new tunnel is driven parallel to an existing one due to increased traffic demands, or when existing twin tunnels are widened, the width of the central pillar becomes critical. This can and will influence portal stability if not properly accounted for in the design or the re-design process. Under "ideal" conditions, a minimum of two tunnel diameters should be left between adjacent tunnel openings to reduce stress concentrations in the pillar, but under the constraints of mountainous topography and/or the location of existing tunnels, this is not always possible. Consequently, it is sometimes necessary to reduce this pillar width to one diameter or less. This then requires a careful evaluation of both the portal and the turn-under section of the tunnel. The result is often the requirement for pre-supporting the central pillar from one tunnel before the second tunnel is constructed, or installing pre-support from one tunnel during the excavation widening process, so that when the second tunnel is itself widened, the pillar is not over-stressed leading to partial or complete failure. These measures are currently being employed in the I-70 Twin Tunnels near Idaho Springs, Colorado, as well as making an evaluation of the ground stability implications in the portal zones.

In soft rock and/or soft ground tunnels, the pillar stabilization process may require ground improvement in addition to additional ground reinforcement. Depending upon ground conditions around a highly stressed central pillar, the proper selection of excavation and ground support methods and sequences in the portal zone may become extremely important.

Flow Chart for Portal Design

There are several good flow charts for portal design in the literature, including the flow chart published by Pelizza et al. (1998). It is summarized in Table 1. It can be readily seen, that portal design, especially for large transportation tunnels (roads and railways) can require a substantial investigative and engineering effort.

During the design process, problems and solutions associated with the tunnel portal zone should be evaluated in a comprehensive risk assessment program. This should include all perceived natural and man-made hazards, a construction term evaluation, and an in-service term evaluation. Each risk should be identified, rated for both magnitude and consequence of risk, and mitigation measures identified. For each risk and mitigation measure, the post-mitigation magnitudes and consequence of risk should be re-evaluated to determine the influence and effectiveness of the mitigation measure considered. Part of this process should also consider the construction cost and schedule for each mitigation measure so that the one with the best cost/benefit ratio can be selected. This “portal” evaluation must also consider the influence upon tunnel construction and tunnel performance in service, along with how those construction and in-service term conditions may influence portal stability and performance.

The types of risks to be considered are basically those issues in Table 1, but for each project, the specific risks must be project specific and the mitigation measures the same, as a function of topography, geology, weathering, drainage, climate, size of portal cut, geometry of portal cut, size, shape and layout of tunnel(s), size and payout of available working space at the portal, etc.

Portal Stability

Portal stability for the life of the tunnel is the key issue of portal design. Generally, the following tasks need to be carried out.

Site Investigation

Site investigation is usually the process that precedes the design for any engineering structure built on or within geologic materials. Hence, the site investigation techniques for a tunnel portal are similar to those used everywhere. The objective is to obtain a good understanding of the following:

- *Geology* at various scales, using field mapping, geophysics and drilling to determine material types and their location
- *Weathering* extent and depth
- *Geologic structure*, particularly faults, major joints and joint patterns (including their orientations)
- *Groundwater conditions*, by mapping seeps, determining phreatic levels and flows, and understanding the overall hydrogeology of the area, for both surface and subsurface water.

Table 1. Flowchart for tunnel portal design (after Pelizza et al., 1998)

Problem	Consideration
1. Landscape and environmental analysis	i. Evaluation of landscape situation, general architectural environment. Local laws and special constraints, social aspects ii. Evaluation of the technological needs (ventilation etc.) (from the tunnel project) iii. Architectonic choice between: ■ Portal inscription ■ Multi-criteria analysis among the solutions iv. Evaluation of the environmental impact of the preliminary chosen solution
2. Site investigation	i. Geological and geotechnical investigation with special attention to geotechnical properties that are important for: ■ Land slide ■ Ground water condition ■ Rock-fall ii. Seismic condition of the area
3. Surface and underground constraints	i. Influence of tunnel portal construction on the surface ■ Definition of displacements that are acceptable for the structures ii. Influence of the portal on the hydrogeology and hydrology during construction (design of temporary solutions) and in operation (design of the final solution)
4. Prediction of ground mechanical behavior	i. Large-scale landslides ■ Evaluation of the critical parameters (water, etc.) and their influence ■ Definition of the safety factors with and without the portal ii. Small landslides ■ Evaluation of the critical parameters (water, etc.) and their influence ■ Definition of risk condition ■ Design of remedial techniques and evaluation of the obtained safety factor iii. Avalanche risks ■ Definition of risk condition ■ Design of protection techniques iv. Rock-falls ■ Evaluation of possible detachment points, block size, trajectories and speed of the blocks ■ Definition of risk conditions ■ Choice and design of rock-fall protection works during construction and when the portal is in use ■ Trajectories and impact energy evaluation ■ Safety factor of unstable blocks ■ Design of active or passive protection measures (if the artificial tunnel is chosen, the data) v. Preliminary portal design and evaluation of various possible construction methods

(table continues)

Table 1. Flowchart for tunnel portal design (after Pelizza et al., 1998) (continued)

Problem	Consideration
5. Architectural and environmental design	i. Final architectural and environmental design ■ Architectural design of the final portal solution with design and location of technological buildings ■ Design of final rehabilitation ■ Choice and availability of construction materials ■ Grassing techniques and choice of trees ■ Evaluation of environmental and visual impact of rock-fall protection structures ii. Evaluation of ancillary works (access roads, working areas, etc.) and linked environmental problems ■ Design of restoration works
6. Design choice and calculations	i. Portal construction method design ■ Design of reinforcement techniques ■ Short term safety factor evaluation ■ Long term safety factor ■ Choice and design of the excavation techniques (blasting, mechanical, etc.) ■ Safety factor evaluation taking tunnel portal into account ii. Structural design of the portal ■ Foundation design ■ Structural design taking the influence of the reinforcing techniques into account ■ Structural safety factor evaluation during each phase of back filling.

Slope Stability—Assessment and Design of Ameliorative Measures

The portal needs to be located in a stable area. Tasks include:

- Assess all the potential failure modes, especially after the portal excavation has been completed, and the effects of the portal excavation upon overall regional slope stability.
- Determine the strength of the relevant geologic materials and geologic structures.
- Analyze slope stability in the portal area for all possible failure modes. This includes assessing the potential of sliding rock blocks and wedges as well as toppling failures(using programs such as DIPS, SWEDGE from RocScience, 2013).
- Analyze the Stability of the overall slope, using the above programs as well as more general failure modes in programs such as SLIDE (RocScience, 2013).
- Provide engineering solutions to problems; these may include:
  - surface water and subsurface drainage and dewatering
  - support of slopes and individual rock blocks
  - cutting back the slope, depending upon the steepness of the topography
  - providing other appropriate engineering solutions.

Stability Assessment and Design of Tunnel Entrance (First 50 m)

Maintaining the brow and the stability of the first 50 m of tunnel is critical. Tasks include:

- Assess the geologic structures.
- Analyze the potential for falling and sliding rock blocks (from roof and walls of the tunnel), using some of the above-mentioned programs and UNWEDGE (RocScience, 2013).
- Design the tunnel support (which is often a combination of rock and cable bolts, wire mesh, shotcrete, concrete arch support, pre-excavation umbrella arch, etc.).
- Develop detailed excavation procedures for first 50 m of tunnel. Each step must be carefully laid out so everyone in the construction team understands the issues, the goals, and the procedures to be followed. This may require multiple headings in large excavation faces and/or in rock masses having poor quality.
- For tunnels to be driven by a Tunnel Boring Machine (TBM), the turn-under may be required to be fairly long, depending upon the length of the TBM with trailing gear, and how much space is available outside the tunnel portal for TBM assembly. In addition, it may be required to be larger in size than the completed tunnel in order to allow working space around the TBM inside the tunnel before the TBM starts up.

Construction

As everyone knows, good design can be defeated by poor construction implementation. Key factors are:

- Discuss procedures in advance with the construction team and maintain good communications.
- Apply any required pre-support before starting.
- Use ‘gentle’ excavation methods in the portal area.
- Once tunnel excavation of the tunnel starts, apply initial tunnel support immediately.
- Ensure there is daily geological/geotechnical mapping and assessment of conditions in the tunnel.
- Provide back-up systems of physical ground support in case they are needed.
- Provide good drainage.
- Monitor ground movements.
- Keep good permanent records (including shift reports, geological/geotechnical mapping, and movement monitoring).

SPECIAL CONSIDERATIONS FOR PORTALS IN HARD ROCK

A strategy is generally needed to support the portal brow and the first 50 m of the tunnel. Usually this involves a combination of rock bolts, wire mesh and shotcrete. In loose blocky ground, wire mesh is preferred to fiber reinforced shotcrete, since the mesh will tend to hold the loose rock blocks together should any rock loosening occur. Once the mesh is secured with bolts, a layer of shotcrete can be applied.

Special care should be taken to secure the brow of the tunnel by careful excavation of the first few blasts, followed by immediate support of the crown (roof) and sides of the tunnel. It is worth excavating this carefully, if only to try and avoid having to fix up

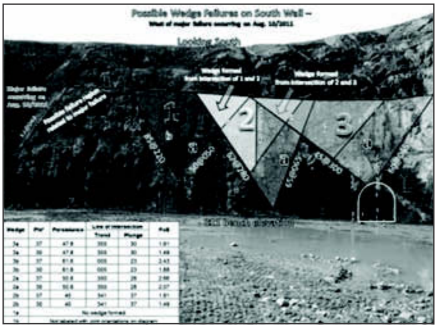

Figure 9. Stability assessment around portal

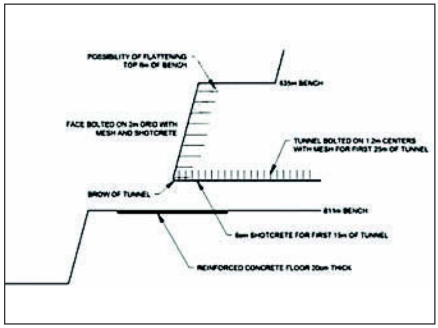

Figure 10. Support for rock face and tunnel

the portal with concrete or steel sets (as seen in Figure 8), before proceeding with the main tunneling operation.

A good example of this kind of design for two rail adits located in basaltic lavas was recently presented by Goel et al. (2012). One of the authors' examples to illustrate the methodology is provided below.

Example of Design Methodology Applied to a Mining Adit

This relatively simple example of the design of a tunnel portal in a rock face is included to illustrate the methodology often used by the authors.

Site Investigation and Basic Analysis of Outcrop Stability

The site investigation started with a review of the geology at three potential portal locations. The stability of the slopes above and around each portal location were carefully assessed. This required structural mapping to determine the orientations of faults and major joints, and stability analysis of potential sliding and wedge failures, as illustrated in Figure 9. Although there was a potential for instability, the risk was considered to be small enough to proceed on the basis that the face above and to the sides of the portal location would be secured with bolts, mesh and shotcrete, prior to starting any portal excavation; a sketch of the support plan is shown in Figure 10.

Investigation and Analysis of Tunnel Stability

The determination of rock quality included estimates of RMR and Q'. These parameters can be estimated from relevant outcrops and boreholes. In this case, the nearest borehole to the tunnel portal indicated the following parameters for the rock mass:

RQD = 89%
Jn = 4 (based on 2 joint sets)
Jr = 1.5 (slickensided and undulating)
Ja = 1.0 (staining only).

These numbers defined Q' = 33.

On the Q-support chart (as shown in many documents, such as Figure 1 in Barton, 2002), this indicates that the tunnel with a finished width of about 5 m should stand unsupported with only the occasional use of 'spot bolting.' This was good news for normal tunneling operations. At the tunnel portal, however, where the rock mass is stress relieved and weaker, Barton recommends that we multiply the normal Jn by 2 (to make 8) and that we use an ESR value of 1. The new Q' = 16, puts the portal on the edge of the graph area whereby the portal needs systematic rock bolting and 50 mm of shotcrete. Since portal stability was important and the client did not want to take any unnecessary risks, the recommendation was to use bolts, mesh and shotcrete for the first 15 m of the tunnel, and just bolts and mesh for the next 10 m of tunnel. After that, the tunnel would be clear of portal effects, and the support could be based on routine mapping, paying particular attention to the daily measurements of RMR and Q' at the face.

In this case, it was recommended that an initial thickness of 25 mm of shotcrete be shot on to the roof and at least half-way down the walls of the tunnel immediately after excavation; steel fiber reinforced shotcrete (SFRS) is preferred to plain shotcrete for this first coating at the entrance to the tunnel. (In normal tunneling operations only plain shotcrete would be used with wire mesh, as SFRS and mesh are considered to be alternatives.) Then after bolts and mesh have been applied, at least another 25 mm of *plain* shotcrete would be applied to tie the initial shotcrete into the bolts and mesh in order to provide a permanent lining of at least 50 mm thickness in the tunnel entrance for at least the first 15 m of tunnel. (After that, shotcrete can be applied according to the actual rock conditions encountered in each 5 m section of tunnel, based on the RMR and Q' measurements made by the geological engineers during their daily inspections of the tunnel.) As a check, the paper by Oraee-Mirzamani (2011) also indicated that a minimum shotcrete thickness of 50 mm should be sufficient for a rock mass with RMR greater than 40, and an overburden load less than 50 m.

Excavation Recommendations

Due to the size of the finished excavation (5 × 5 m) and the recommendation to cut the first 15 m of tunnel with a cross-section of 5.5 × 5.5 m (to allow room for extra support using concrete or steel sets should these be needed), it will be advisable to excavate the portal in two cuts. The top part will be 3 m high by 5.5 m wide, and the bottom part 2.5 m high by 5.5 m wide. The top part will be advanced and fully supported after each blast until 25 m is reached, then the bottom part will be excavated and supported.

With the first 25 m of excavation completed, drill weep holes at 1 m height and 1 m length every 1 m along all faces where shotcrete has been applied inside the tunnel; this is to help relieve any water pressure build up behind the wall of shotcrete.

Clean the floor of the tunnel and fill with 20 cm thickness of reinforced concrete (see Figure 10). The concrete should extend across the full width of the tunnel for at least the first 10 m of tunnel (with allowances for any drainage ditch) and outside the tunnel for at least 10 m as part of an apron. This will complete the stabilization of the portal area, as well as providing a safer surface for vehicles entering and exiting the tunnel.

Set up a simple monitoring system to monitor deformation of the tunnel, including vertical and horizontal closure, at about 3 locations in the first 25 m of the tunnel; ensure this information is passed on in a timely manner to the engineers responsible for design and construction.

In addition to constructing a stable portal area, the experience gained in constructing the first 25 m of tunnel, will enable a better support plan to be drawn up for the regular tunnel once the main tunneling operations begin.

SPECIAL CONSIDERATIONS FOR PORTALS IN SOIL OR WEAK MATERIALS

Design Problems and Possible Solutions

Portals located in soils and in weak rock can introduce many other issues that need to be considered in design. For example, avoid placing portals in an old landslide. Be aware that small micro-features, such as thin slickensided clay seams, varved clays, or old weak weathered foliation surfaces can be missed easily in core logging, and that this can have huge impact on ground movements during portal construction (Richards et al., 1985).

The Japanese Society of Civil Engineers (1996) in their Japanese Standard for Mountain Tunneling list and describe the characteristics of several types of tunnels, including Gravity and Semi-Gravity, Wing, Arch-Wing, Parapet, Protruding, Split Bamboo (or inverted), and Bell Mouth.

One of their tables is reproduced and amended herein as Table 2. It lists seven types of "typical" problems that can occur at portals in soil and weak rock (and some of them apply to portals in hard rocks as well). Again, this is another check list to be considered when considering design options.

Construction Options in Weak Ground

Each portal construction option has its own physical and technical limitations, which interact with constructability, available equipment and material limitations, skilled labor experience and availability, performance risk, design life and service longevity, construction cost, and construction schedule. The suitability of any one of these potential construction options depends upon a site specific stability analysis, and the trade-off between slope angles, cut height, and some combination of slope stabilization measures. Also, any solution will require surface water collection and diversion measures above the portal, as well as protection against long term overburden material surficial instability above and around the portal.

For example, one of the authors considered the following stabilization measures for a portal in colluvium and weathered soil and rock in South Korea:

- Soldier Piles and Tiebacks
- Gabion Baskets
- Soil-Nailed Wall
- Vertical Overlapping Jet Grout Columns
- Rock Reinforcement Supplemented by Shotcrete
- Open Cut—with no slope support

Variations of all these alternatives were considered before deciding on the final design of the portal.

In another situation, one of the authors (Richards et al. 1985) was involved in designing the portal for a coal mine in western Colorado. An earth fill buttress, sheet pile wall, and multi-strand tendon tiebacks, were all needed to stabilize the portal which was cut into an old landslide (see Figure 7). There were 5 parallel entries. It was first necessary, however, to tunnel through the buttress before getting into natural ground where the tunnel was supposed to start. The revised process caused substantial construction delay and cost increase.

Evaluation of Alternative Designs

The above examples, emphasize the need to consider alternative designs (of the tunnel and the portals) in order to optimize the design and construction of a tunneling project. In the case of a simple portal into a stable rock outcrop, the work required to evaluate the portals part of the project can often be done in a few days or weeks.

In the case of large civil engineering tunnels under deep cover, there are more alternatives to consider and the time (and cost) of evaluating alternatives is therefore a much larger effort. With portal construction costing up to $10M for some road tunnels, there is more at stake in optimizing the design. Standard 3-D Digital Surface Models (DSM) with geological packages can be used to assist in this process.

Table 2. Possible problems at portal section and design considerations

Problem	Consideration	Engineering Solution
Slope failure or landslide	Construction in the portal zone sometimes induces a slope failure or landslide. It is ascribable to the loosening due to the tunnel excavation or slope cut by the construction of the portal. When there is a possibility of a slope failure or landslide due to tunnel excavation, measures should be taken to protect the slope in advance of the excavation.	■ Move portal to a more favorable location ■ Stabilize slide – Install sub-drains and/or modify surface drainage paths – Unload crest – Buttress in front – MSE Wall in front – High Capacity tie-backs
Unsymmetrical pressure	Unsymmetrical pressure may act on the tunnel section and a large stress may occur in the tunnel depending on the positions of the slope and tunnel. Unless the tunnel stabilizes, measures are required to balance the earth pressure by using a counter-weight fill or cut floor slope stabilization.	■ Increased ground support on high pressure side ■ Unload crest on high side
Insufficient bearing capacity of ground	In the portal zone, where the depth of overburden is small, total load over the tunnel may act on the tunnel sometimes. The ground in the portal zone consists of unconsolidated deposits or is in a weathered zone so that the base often suffers from settlement or deformation owing to insufficient bearing capacity of the ground. Design not only of the portal but also of the construction method should be made so that the required bearing capacity of the ground can be obtained.	■ Ground improvement in unstable areas before excavation
Face collapse	In the portal zone, the ground is often weak and poorly consolidated. Even when the ground consists of hard rocks, faults or fracture zones may induce the development of fractures. Thus the stand-up time is short at the face in many cases. When the face cannot be expected to stand sufficiently long, adoption of an excavation or auxiliary method to prevent face collapse needs to be studied.	■ Pre-support over crown such as spiling or pipe umbrella arch canopy ■ Ground improvement before excavation

(table continues)

Table 2. Possible problems at portal section and design considerations (continued)

Problem	Consideration	Engineering Solution
Ground surface settlement	In the portal zone, small overburden, insufficient bearing capacity of the ground and insufficient stand-up time at the face are likely to cause the impact of settlement to be transported to the ground surface. For the structures at the ground level for which settlement should be restricted, adequate measures must be taken to prevent any problems and an auxiliary method must be adopted whenever required.	■ Smaller multi heading excavation faces and shorter advances per heading ■ Increased ground support and pre-support
Rock fall, debris flow or avalanche	The portal zone needs to be designed to be located in a place free from rock falls, debris flows or avalanches. When such positioning of the portal is impossible, adequate measures should be taken against possible disasters.	■ More portal to a more favorable location ■ Scale loose pieces in advance ■ Rock-fall nets or other control structures ■ Stabilization measures in avalanche source zones ■ Diversion structures to change avalanche path away from portal ■ Extended portal canopy beyond actual tunnel portal to catch and/or divert debris
Neighboring structures	Impacts of tunneling construction on existing structures in the neighborhood such as houses, steel towers, roads and railways, and the effect of noise and exhaust gas after the tunnel went into service should also be studied.	■ Move tunnel portal further away from sensitive structure ■ Increase ground support or pre-support to prevent movement ■ Underpin structure foundations ■ Compensation grouting

Recently CETU (Ministry of Transportation in France) has made available for general use the T-Tunnel software, which is dedicated to the design of tunnel portals. The software is able to accept topographic and geological information in various formats (including .dxf format).

In CETU's experience the costs of technically-viable portal designs can vary by a factor of four. Their software is designed to reduce the amount of detailed work that is needed to evaluate each of the alternatives. It does this by combining the geometrical and geological data with different construction layouts. The construction costs of excavating and supporting the different types of materials is also an input into the program, as shown in Figure 11.

When the geometry of the portal is added to the digital terrain model, the software automatically calculates the geometry of the surrounding slopes and the volumes of different geologic materials to be excavated or filled, along with the associated costs. The software can be used in the initial evaluation of alternatives (different portal locations) as well as optimizing the design at the preferred location. CETU (2013) and

Gaillard et al. (2011) describe the software. They say that the software allows designers to consider many more configurations than would normally be considered, including:

- Comparing an oblique angle of entering the face compared with a 'normal' angle of 90°
- Moving the portal more into the rock mass or moving it further away further away (trade-off between the size of the box cut and the amount of any heavy support needed at the beginning of the tunnel)
- Varying the slope angles in the different zones.

Also, they say that the 3-D visualization makes it a good tool for communicating with other members of the project team as well as other stakeholders.

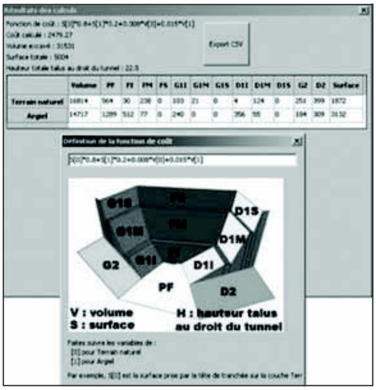

Figure 11. T-Tunnel input defining the costs of different construction functions

Some Instructive Case Histories

While the aim should be to locate the portals in good rock to minimize the costs of construction, this is not always possible. Consequently, there are hundreds of instructive case histories pertaining to portal design in the literature; most of these are focused on dealing with difficult ground conditions.

Unfortunately there is not sufficient space to discuss all of these designs in this paper, but some of them are mentioned below for possible further follow-up by the reader:

- Barisone et al. (1882, 1983) describe an umbrella arch method for tunneling in difficult conditions
- Peila & Pelizza (2002) describe a similar system which includes the use of reinforced shotcrete and steel sets below the jet-grouted canopy
- Wittke et al. (2007) describe the use of a pipe umbrella to support the roof in a railway tunnel in Cologne; this method could be applied also in the portal area where the roof is collapsing
- Kirsten & Alexander (2002) describe the design and construction of two portals in a talus slope in which they used reinforced-earth retaining concepts in conjunction with soft ground tunneling techniques. The east portal included a reinforced concrete apron to hold the talus, with 10 m spiles in advance of tunneling to hold the roof of the tunnel, which was supplemented with soil nails to hold the brow. For the western portal the spiles were driven from the surface through the talus into bedrock, and then fully grouted. Steel sets and shotcrete were also incorporated into the designs.
- As previously mentioned, Rothfusset al. (1995) discuss the causes of 7 portal failures in their paper, and they also discuss the designs and the remedial works undertaken; three cases are given in reasonable detail.

These and other case histories are listed in the References.

OTHER FACTORS

Depending on the climate, location, topography and geology, other factors may need to be considered in design. Some of the more common issues are discussed below.

Trends in Ground Support for Portals

In road tunnels, canopies extending 1–5 m beyond the slope face of the portal are becoming standard. An example is shown in Figure 2, which also shows the face and sidewalls supported by bolts and shotcrete; canopies are shown too in Figures 3, 4 and 8. These canopies are often constructed from pre-cast concrete segments. They are a relatively inexpensive method of controlling rock fall immediately above the portal by diverting any small rock or ice fall to the side of the tunnel entrance. Space should be left each side of the entrance, however, to provide occasional access for a small loader to scoop up the fallen debris.

Shotcrete is particularly effective in supporting mine portals and is frequently used as the major component of tunnel support in temperate climates. It is often applied to the face above the portal as well as within the first 50 m of tunnel. Also, it is the standard support for the sides of the box cut on the approach to a mine entry, as seen in Figure 5, where the main support was shotcrete which was occasionally supplemented with rock bolts. Since shotcrete machines are standard equipment in most mines and on most civil tunnel projects, any repairs to damaged shotcrete readily can be made. The important features of applying shotcrete are speed of application, durability, adaptation to uneven surface contours, and cost effectiveness. When combined with bolts and mesh, shotcrete can improve the effective support in the portal area by holding the loose rocks together, and providing corrosion protection to the steel mesh. Any severe movement of the portal or slope above it will cause cracks to appear in the shotcrete; these cracks can be readily seen and ameliorative measures taken to strengthen the support in that area.

The design of the shotcrete mix is a skill (art) in itself and depends on many factors including the rock conditions, the equipment available and the experience and skill of the operators. An excellent primer on Shotcrete is provided by Hoek (2011).

In hard rock, the authors often start with a design which can be described as ‘rock reinforcement using a combination of rock bolts, steel mesh, shotcrete, and weeps.’ The weep holes are needed, of course, to provide drainage of any subsurface and/or infiltrating water accumulating behind the shotcrete.

In most cases, the execution of portal construction is critical. Paying close attention to the principles of careful excavation (i.e., no heavy blasting) and the application of immediate support costs very little extra, but they can make the difference between a stable portal area and an unstable portal area (possibly requiring additional and ongoing maintenance).

If unexpected problems develop during portal construction, additional support systems may be applied. These include:

1. Adding an extra thickness of shotcrete to small unstable areas, if this is all that is needed.
2. Adding reinforced concrete arch support or steel supports (for the roof, sides and floor) to the first 10 to 20 m length of the tunnel (this is a good reason for cutting the first 20 m of the tunnel a little larger to ensure that any additional support still allows minimum design dimensions to be met).
3. Grouting the roof and sides of the tunnel using a suitable pressure grouting technique.
4. Within the tunnel itself, other techniques are available such as forepoling and pre-injection grouting.

Analysis of Rock Fall Hazard

As opposed to a rock slide, rock fall refers to rock(s) falling freely from a rock face. The rock fragment or rock block may become detached initially from the rock face by falling, sliding or toppling. Then it progresses vertically or sub-vertically down the cliff by falling, or by bouncing and rolling down the slope often in a ballistic trajectory.

The causes are a combination of near-vertical slopes with unfavorable geology. The rock mass contains discontinuities which can open up due to stress relief, external forces, root wedging, and weathering susceptibility including rainfall and freeze-thaw effects. Despite any site investigation that may have indicated that the face was safe at that time, it is important to realize that the stability will likely deteriorate with time. Falling rocks may dislodge other rocks and cause them to fall as well.

There are techniques, graphs and computer programs that can be used to forecast the trajectory of falling rocks from any location on the rock face. One of the best programs for doing this is the Colorado Rockfall Simulation Program, which has been recently upgraded from 2-D to 3-D (FHWA 2012; Anderson & DeMarco 2012). This and similar techniques can be used also to help design mitigation measures, both active and passive measures.

Active mitigation is carried out in the initiation zone with the aim of preventing the occurrence of the rock fall. These measures include, firstly, careful scaling, followed by rock bolts, mesh, shotcrete, and slope retention systems. Other active measures include modifying the geometry of the slope, dewatering, and the use of vegetation.

Passive mitigation measures are generally employed in the deposition or run-out zones. These include the use of drape nets, catchment fences and diversion structures. The rock fall does take place, but an attempt is made to control the outcome.

Rotec International (Rotec 2013) is one of the companies that designs, builds and installs engineered systems of woven nets to dissipate and stop the energy from random rockfalls. Also, their retaining fences can be supplied with a monitoring alarm system that issues a warning whenever the fence is impacted by unusually severe rockfall. In addition, their flexible net blankets can be draped over the slope; these blankets conform to the contours of the slope and are economical and easy to install. They ensure that any unraveling rocks stay within the net.

Geobrugg (Geobrugg 2013) is another international company that produces rockfall protection mesh systems (catch fences and rockfall drapery/rockfall netting) made from high-tensile steel wire to match different rockfall problems.

For tunnel portals, it is recommended that a combination of active and passive measures be taken. That is why the authors favor the use of relevant active measures designed for the particular face in combination with passive measures, such as a tunnel canopy to divert any small rock blocks that manage to become dislodged from an otherwise stable face. From recent observations of transportation portals throughout the world, it appears that more and more designers are employing the same design strategy.

Avalanche Assessment

In high latitude and/or high altitude tunnels, avalanches and/or tabular snow slides present a potential risk to people and structures in mountainous terrain when there is build-up of snow pack that becomes unstable and roars down the mountain causing death and destruction. In portal areas where avalanches are possible, a risk assessment should be carried out. This involves identifying geographic features such as drainages, vegetation patterns and seasonal snow distribution, and previous avalanche paths that are indicative of avalanche potential. Then the hazard can be assessed with regard to the chances that the avalanche will reach the portal and its nearby access (road, railway, or pipeline).

An avalanche control program is then designed to prevent and/or mitigate any avalanche hazard, The program includes monitoring the snow pack during the winter season. Control techniques either intervene directly in the evolution of the snow pack or lessen the effect should an avalanche occur. Active interventions include the use of mechanical and blasting methods to trigger an avalanche under controlled conditions (with humans safely out of the way).

Since portals and transportation corridors are permanent fixtures, it is sensible to create permanent ways of preventing avalanches from reaching the portal area. These measures include:

- Modifying the terrain to reduce the likelihood of avalanches forming, or diverting them away from the portal if they do form and run
- Reforestation to reduce the likelihood of an avalanche breaking away
- Installing snow retention structures such as snow nets, snow racks and snow bridges in the upper path of a probable avalanche
- Creating avalanche barriers using high strength steel wire mesh extending across the slope and reaching up to the surface of the snow. This helps prevent a potential avalanche from breaking away. The forces required to retain the snow are absorbed by the steel net, and transferred to ropes passing over swivel posts to fixed anchor points
- Building snow redistribution structures such as snow fences and snow baffles
- Building deflection structures to confine and/or deflect the moving snow away from a portal structure
- Constructing retardation structures such as snow breakers to enhance retardation at strategic points in the avalanche track
- Building snow catchment structures
- Protecting important structures directly, by constructing avalanche sheds over the structure (so the avalanche passes harmlessly over the top of the structure); these are often used at mountain tunnel portals as the most cost effective solution
- Architectural streamlining of the civil engineering structures in the avalanche path e.g., creating wedge shape buildings that do not take a direct impact from any avalanche.

Rotec International (Rotec 2013) and Geobrugg (2013) are two of the companies which sell yielding wire-rope net structures that form strong flexible barriers. These compact and harden the newly-contained snow within the net system.

Clearly, the study of avalanche risk assessment, hazard evaluation and design of control measures requires the input of people with this kind of specialization. Even with good permanent measures in place, it is still necessary to monitor the snow pack in winter as well as to make periodic inspections of the permanent control measures to ensure they will function as designed.

Seismic Effects

Earthquakes should be considered in earthquake prone areas, especially in soils and weak rocks. When P and S waves from an earthquake reach the ground surface most of the motion is reflected back into the underlying geologic materials. Hence the surface is subject simultaneously to upward and downward moving waves. This amplification may cause failure of the portal and nearby surface structures, as happened in Taiwan when the ChiChi earthquake struck in 1999; amongst other catastrophic damage, the earthquake caused 132 landslides, damage around numerous portals, and the closure of much of the islands transportation systems.

On the other hand, earthquakes almost never cause damage to deep tunnels or mine workings away from the portal (Ross-Brown et al. 1981; Bolt 1999).

Construction in Extreme Winter Conditions

Cold weather interferes with the design and the sequence of construction. For example, in cold weather climates it is recommended that shotcrete *not* be applied if the substrate (rock surface) temperature is below 5°C. In addition, the air temperature in contact with the shotcrete should be above 10°C; otherwise the setting and hardening of the shotcrete can be very slow and the bond to the substrate could be damaged.

The American Concrete Institute (ACI 2009) has this to say about Cold Weather Shotcreting:

> *Shooting may proceed when the ambient temperature is 5°C and rising (10°C for latex-modified shotcrete). Shooting shall discontinue when ambient temperature is 5°C and falling, unless protective measures are taken to protect the shotcrete. The shotcrete material temperature, when shot, shall not be less than 10°C or more than 32°C. Shotcrete shall not be placed against frozen surfaces…. Applicable procedures used for cold weather concreting may be used for cold weather shotcreting.*

Hence, if construction starts in the winter, shotcrete cannot be used without devising a heating system to keep the shotcrete at the recommended temperatures during placement and curing. This applies also to the placing of any concrete, such as the floor of the tunnel and the apron for vehicles entering and leaving the tunnel. For example, in the tunnels for the British Columbia Railway Tumbler Ridge Branch Line in the winter of 1982–1983 (Morgan & McAskill, 2005), the tunnels were boarded in with insulated timber (with a doorway for equipment access), and two large diesel forced-air furnaces were installed outside one end of the tunnel. A ventilation fan was installed at the other end and a flow of warm air forced through the tunnel. Generally, it took between 7 and 10 days to raise the temperature of the tunnel to about 10°C, with the rock face at a temperature between 5 and 8°C. Shotcrete was applied to the tunnel when the air, rock and shotcrete temperatures were all above 5°C. (Special procedures were needed to prepare the shotcrete for use in the tunnel.) The purpose of mentioning this example is only to point out that shotcreting can be done in winter conditions. The question then is: does shotcreting have to be done in winter?

If shotcrete is not used during initial construction, more reliance may need to be placed on rock bolts and mesh. Unfortunately, there is the additional problem of installing fully-grouted bolts under winter conditions. For example, cement-grouted bolts suffer from the same restrictions as shotcrete. Also, the temperature at installation time greatly affects the gel (set) time of resin cartridges. According to one supplier (DSI Underground System Inc., 2012), 'a 11°C rise in temperature reduces the set time by approximately 50%. Conversely, a 11°C drop in temperature will approximately double the set time of the resin. Resin should be stored and used at room temperature whenever possible.' If room temperature is assumed to be about 20°C, it is clear that there will restrictions in using resin cartridges under winter conditions; it will be difficult to get proper mixing, and a drop in temperature to near-freezing will quadruple the set time which may make it impracticable.

If shotcrete cannot be applied in winter, and it is not practical to use fully-grouted bolts with cement or resin, the next option is to use mechanically-anchored bolts. This is considered by the authors to be a relatively poor solution, since any movement of the anchor or the plate will likely loosen the bolt and render it ineffective other than for holding up the wire mesh.

It is worth noting that almost all of the alternative methods of portal construction involve the use of grout and/or mass concrete, including 'gravity or semi-gravity' designs which involve building a thick concrete retaining wall in front of the face (for example, Figure 6).

Generally portal construction will not be as robust in extreme winter when compared to doing the construction in warmer weather using fully-grouted bolts, shotcrete and concrete. If started in extreme winter, the portal area probably will need additional maintenance and support in warmer weather.

Independent of the effect of winter on construction materials, there may be a problem with ice buildup in the portal area during the life of the tunnel. So a strategy is needed to deal with this eventuality. Tattersall et al. (2002) discuss the approach and solution to the problem of ice build-up in the 4 km Anton Anderson Memorial Tunnel in Alaska. In 2011, the Trans-Pennine Express train carrying passengers crashed and derailed in a tunnel at 70mph after an 8 m chunk of ice fell from a ventilation shaft onto the tracks; this was in the UK, not generally considered to be a 'cold region'. As the bibliography indicates, Norwegian experience in dealing with these kinds of problems is extensive; the Norwegians have developed good solutions for these problems.

SUMMARY AND CONCLUSIONS

To summarize, an optimal portal design "solution" must consider and account for:

- Subsurface ground and groundwater conditions
- Topographic and climatic condition
- Long term surface water hydrology run-off patterns and infiltration and erosion considerations
- Portal cut dimensions and geometry
- Minimization of environmental impact both during construction and in-service
- Schedule compatibility with overall contractual program
- Construction cost compatibility with the "low bidder" agenda, without excessive risk
- Designer preferences based upon local experience, and locally available construction materials
- Contractor preferences based upon available skilled labor resources, specialized construction equipment, and specialty construction materials
- Requirement for ground monitoring for slope stability behind the portal cuts during construction (and possibly long term)
- Potential requirement for subsurface drainage within the cut slopes to maintain stability; but be aware that the drainage may freeze up in cold climates
- The side slope of the hillside relative to the tunnel alignment and portal entrance locations, which may require a staggered portal entrance arrangement if there are a pair of tunnels in order to have an acceptable overburden cover over each of the tunnels at the headwall
- In addition, grade separation in the vertical alignment between two tunnels, would also increase the overburden cover of the "downhill "tunnel, minimizing the requirement for a staggered entrance separation to achieve the same goal.

This paper does not claim to identify all of the potential problems and solutions related to tunnel portal analysis and stability. Rather, the intent has been to try to identify the most commonly occurring key issues, some of the most common problems associated with those issues, and some of the more commonly used engineering solutions to those problems. This paper can therefore serve as an initial checklist to a portal designer, to make sure that nothing critical has not been evaluated. With this perspective, and considering that every site is different, with its own set of physical, construction and in-service boundary conditions, a portal designer should at least be able to get started on the right track; but should not forget that some sites may require innovative or hybrid solutions based upon actual site conditions.

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These references and bibliographic entries are included to cover the general subject of tunnel portals sufficiently well to support the text, as well as the sub-topics like avalanche and rock-fall control, although not all were referenced in the text. The non-referenced entries, although consulted but not referenced, allowed us to determine if anything important was missing, and to decide what to include as primary input and what not to include as secondary material. In addition, a slightly more comprehensive reference list serves as an initial guide for the reader to investigate any related portal design and construction topics in more detail.

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IMPACT ASSESSMENTS ON UNBOLTED WEDGEBLOCK SEGMENTAL LININGS FROM OVER-TUNNELING

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ABSTRACT

With urban tunneling for a variety of infrastructure projects, there is increasing demand for underground space. This inevitably leads to the requirement to demonstrate that construction of proposed tunnels above existing ones does not result in adverse impacts, both structural and operational, on these structures. Recently there has been a push to undertake detailed 3D numerical analysis for such investigation.

This paper outlines a simplified, 2D approach that can be adopted for undertaking a technical check and assessment of potential damage focusing on unbolted wedge-block lined tunnels in clay. The approach couples 2D numerical analysis with well-established closed form solutions resulting in a pseudo-3D analysis. This method can be used to simply predict the integrity of unbolted, wedge-block linings subject to internal water pressure, due to over-tunneling.

BACKGROUND

Construction of urban tunnels can lead to concerns for owners of other underground assets where the new tunnels cross-existing ones either above or below. In particular, one of the main concerns is the potential for the construction of new tunnels to relieve sufficient pressure from around pressurized unbolted segments used for water supplies to cause the confinement pressure ratio (CPR) to drop below acceptable levels. This paper looks at methods of analysis in terms of displacements and distortions, change in hoop axial forces and bending moments.

The confinement pressure ratio (CPR) is the ratio of the hoop load in the segmental lining from the internal pressure to the load from the overburden. This is used to determine the effect of the change in stress from the over-tunneling of the pressurized tunnels, before and after tunnel construction. The CPR is critical for unbolted segments for internally pressurized utility tunnels, which rely on the externally applied pressure for stability. A CPR of 1 indicates that the tunnel lining is at a state of limiting equilibrium and any further reduction in external ground pressure, or an increase in internal water pressure may result in the integrity of the tunnel lining being compromised. Ideally, a CPR much greater than 1 is required to ensure an adequate factor of safety. The CPR is of little consequence when no internal water pressures exist.

The CPR is defined as:

$$CPR = \left| \frac{\text{Hoop load due to full overburden (compression)}}{\text{Hoop load from internal water pressue (tension)}} \right|$$

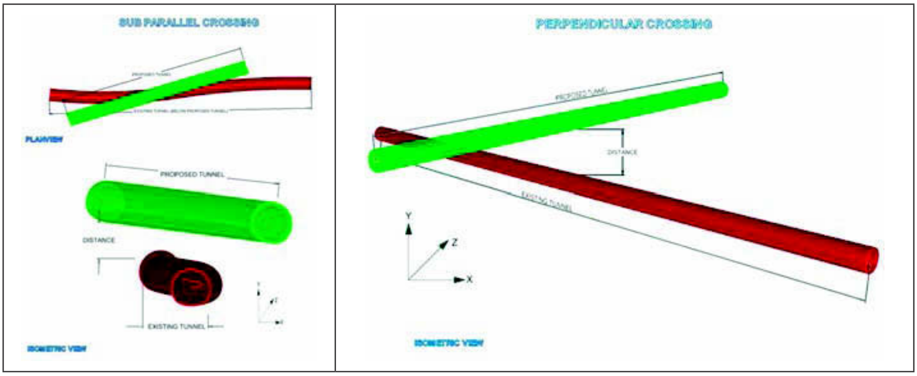

Figure 1. Plan location of crossings

As part of the impact assessment process as a first action, it is recommended that a visual inspection of the existing tunnel linings is undertaken though it is recognized that as the pressurized tunnels would need to be taken out of service and drained, this may not always be a practical solution.

The paper presents a methodology for analyzing cases where tunnels cross approximately perpendicular to one another and where the two tunnels are sub-parallel (see Figure 1).

Potential damage assessments can be undertaken using two–dimensional finite element analyses (2D FE) or three-dimensional analyses (3D FE). For more detailed representation of tunnel geometries and time dependent effects, particularly with complex layouts, 3D FE analysis offers considerable advantages in comparison to 2D calculations. Despite this 2D FE models are mostly used usually due to the significantly reduced computing time using simplified methods in comparison to 3D calculations.

For this reason, an approach which couples 2D numerical analysis with well established closed form solutions, resulting in a pseudo-3D analysis, was developed. This method can be used to simply predict the integrity of unbolted, wedge-block linings subject to internal water pressure, due to over-tunneling. Consequently, the potential damage assessment of the pressurized tunnels can be undertaken more efficiently and with sufficient precision in lieu of a 3D FE analysis approach.

MODELING METHODOLOGY

General Description

Sub-Parallel Crossings

Where the tunnels are approximately sub-parallel, a standard staged analysis models both the pressurized tunnel and new tunnel in the one 2D section. In this analysis, staged construction of the pressurized tunnel using undrained conditions is simulated, followed by a long-term analysis to simulate the consolidation which may have occurred after its construction. As both tunnels are explicitly modeled in the FE analysis, the change in stresses in the existing tunnel lining and the consequent effect on CPR caused by over-tunneling, can be assessed directly from the output of the analysis.

Perpendicular Crossings

Where the tunnels cross approximately perpendicular to one another (see Figure 1) both tunnels cannot be modeled in a single 2D section. The problem could be solved using 3D analysis but as this would involve extra time and cost, a 2D approach would be preferable. Therefore, a staged construction analysis in the cross-sectional plane of the proposed tunnel can be undertaken followed by the outputs from this analysis as inputs in to a closed form solution, in the plane of the existing tunnel.

In general this analysis approach and the closed form solution is based on the medium being clay and using the undrained (short-term) ground response of the ground, through a relaxation approach, followed by installation of the lining. A consolidation analysis is undertaken using the drained parameters of the ground to simulate the ongoing consolidation of the clay following the construction of the new tunnel.

Analysis Methodology

To facilitate the assessment of the induced stresses and deformations within the existing tunnels, first a 2D-FE analysis is undertaken in the plane of the new tunnel. This analysis considers soil-structure interaction effects and ground softening around the existing tunnel by adopting a relaxation approach to simulate over tunneling and a depth dependant, small strain stiffness model for the clay.

Ground Model

For each location, 2D numerical models are undertaken to assess the impact of the proposed tunnel works on the pressurized tunnel assets. These are used as they are simpler and can provide a conservative representation of impacts due to over-tunneling.

The non-linear stress strain behavior of clay and other soft ground has been well-established (van der Berg 1999; Mair et al., 1993). So widely accepted is this fact that the non-linearity is included in current numerical models as a matter of course. The debate has reduced simply to the values of the stiffnesses at various strains. Despite the sophistication of modern numerical models the fact is that they are only an approximation of reality. Though it has narrowed, there still remains a gap between the parameters determined during the site investigation and the parameters needed in a numerical model to produce realistic results. This is evident when the case of a real tunnel is back-analyzed and the results from the numerical model are compared with the actual measurements of behavior (e.g., Jones et al. 2008). For this study, the non-linear small strain stiffness model proposed by Jardine et al. (1986) has been adopted.

A distinct advantage of using a strain-dependent stiffness model is that 'ground softening' around the tunnels is captured. In this way a more accurate prediction of loads in the tunnel linings and ground deflections is captured. As part of this study, the closed form solution (discussed further in this paper), which was used to predict the change in loads in the existing pressurized tunnels, cannot explicitly model the non-linearity of the clay. Thus, the stiffness input in to the closed form solution is reduced based on the non-linear stress-strain relationship, taking into account the predicted current levels of mean effective stress and strain amplitude around the existing pressurized tunnel. The stiffness input into the closed form calculations considers both the strain induced from building the initial tunnels plus that from internal flow pressures pushing the lining against the ground coupled with that from over-tunneling. This simplification produces acceptable results.

Simulating Over-Tunneling

A maximum and minimum volume loss considered appropriate for construction of the new tunnel is first established. Following this the model is undertaken using a 'relaxation approach' to determine the bounds of stress change induced at the level of the pressurized tunnels due to construction of the new tunnels. In this way, the new tunnel is modeled with an appropriate relaxation factor to simulate volume loss. The change in ground loading on the pressurized tunnels due to construction of the new tunnel is then reported as a result.

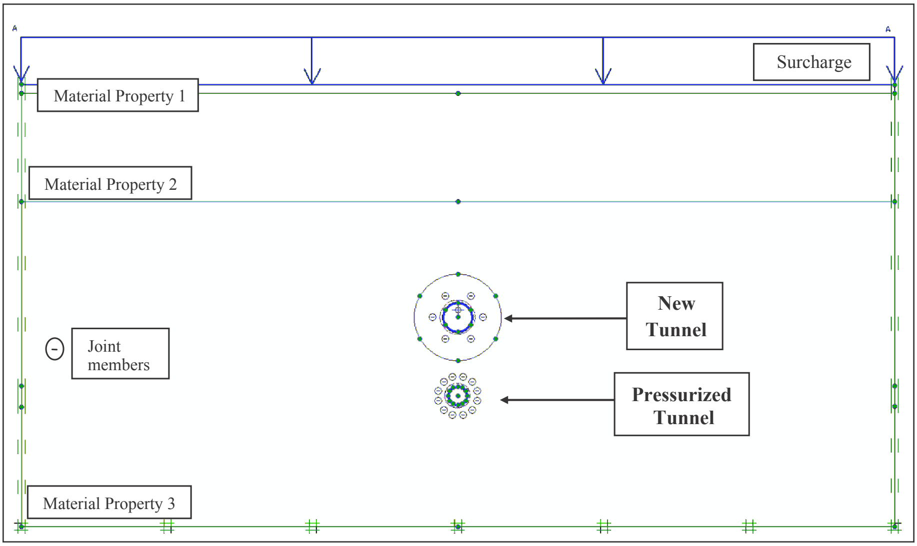

Figure 2. 2D Finite element model for sub-parallel crossing

For the sections where the new tunnel is perpendicular to the pressurized tunnels and both tunnels cannot be explicitly included in the same model, ground displacements and change of stress (see Figure 4) are extracted from the pressurized tunnel and inserted in to the closed form solutions.

The values obtained may predict a conservative volume loss. The results of the numerical analysis can be obtained in terms of stress, which represents the stress redistribution of the ground around the tunnel due to proposed tunnel excavation.

2D Analysis

Sub-Parallel Crossing

To study the impact of the potential damage assessment a 2D FE plane analysis is implemented for each pressurized asset. Where the two tunnels are sub-parallel, the 2D section may be simplified to include both tunnels in the one model as shown in Figure 2.

Perpendicular Crossing

To study the impact where the tunnels cross approximately perpendicular to one another (see Figure 3) it is not possible to obtain change in lining loads in the existing tunnel as it is not modeled explicitly—therefore the resultant change in CPR cannot be calculated directly from this 2D FE analysis. However, changes in ground stresses, strains and deformations at the level of the existing tunnel can be determined; these form inputs to the subsequent closed form analysis. A typical modeling sequence would therefore be as follows:

a. set up model with the appropriate geotechnical parameters and initial stresses and the groundwater profile;
b. simulate the construction of the pressurized tunnels by assuming an appropriate relaxation factor to obtain the target volume loss;
c. extract the stresses, strains and movements at the crown and invert of the new tunnel;

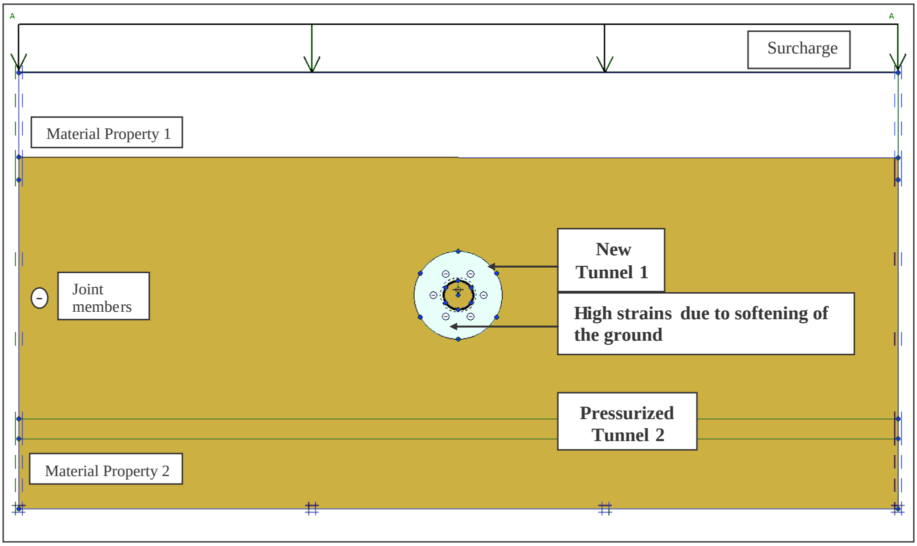

Figure 3. 2D Finite element model for perpendicular crossing

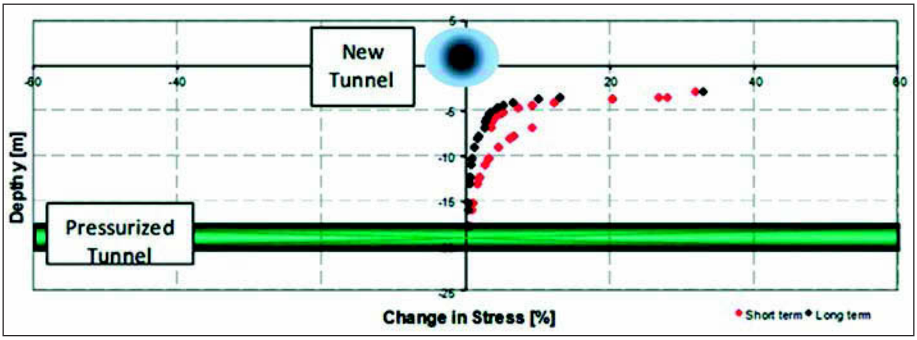

Figure 4. Change of stresses during the construction stage of the new tunnel

d. apply these values to a closed form analysis of the existing tunnel.

As only the new tunnel can be modeled, the changes in the existing pressurized tunnel lining loads cannot be explicitly detailed. For that reason, the results from the numerical analysis (see Figure 4) with the change of stresses are obtained from the pressurized tunnel and combined with the closed form solutions.

Closed Form Analysis

As the proposed tunnel passes above the pressurized tunnels, the resulting ground movements most likely result in a 'squeezing' mode of deformation of the pressurized tunnels (as shown in Figure 5). This squeeze produces induced bending moments in the tunnel lining, which may be calculated using the assumption of elliptical diametrical distortion using the equation by Morgan 1961.

Loganathan and Poulos.1998 used actual measurement data to support their method of induced ground movements caused by tunneling in clay; therefore, using this method, deformation in the pressurized tunnels due to over-tunneling is calculated. From these results, the longitudinal strains, curvatures, and diametrical distortion can

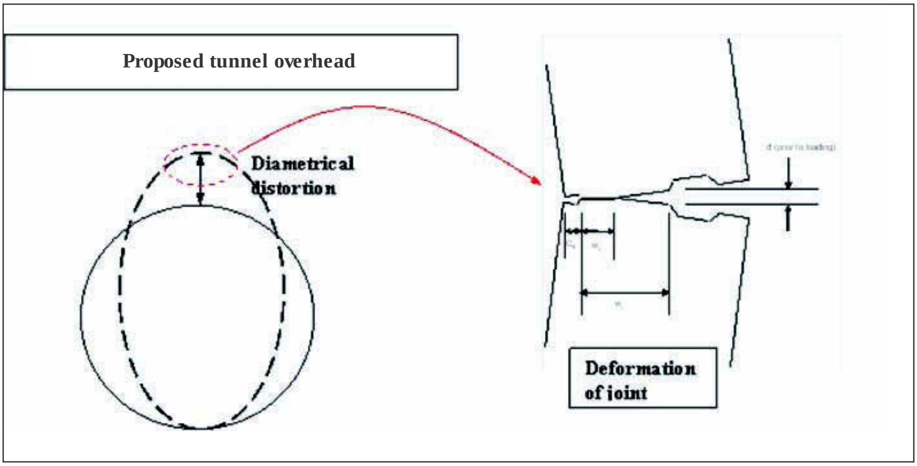

Figure 5. Likely mode of deformation

be determined. Furthermore, the local stresses and 'birdsmouthing' induced at the segment joints can be inferred.

Additionally, over-tunneling by the new tunnel will result in stress changes in the ground. These stress changes may act to reduce initial hoop loads in the pressurized tunnels. Therefore, to determine the effect of this change in stress from the over-tunneling, the CPR, of the pressurized tunnels, before and after tunnel construction is calculated.

The hoop loads due to ground pressure are calculated using the Curtis-Muir Wood.1975 solution. The change in stresses in the pressurized tunnels due to over-tunneling is calculated using the closed form solution by Kirsch.1898 as indicated in Figure 6 that shows, diagrammatically, the change in stress with distance from a circular opening in elastic ground, calculated by this solution.

Following this, the reduced ground pressures are used as inputs to the Curtis-Muir Wood. 1975 solution to determine the change in hoop load and bending moment due to over-tunneling. Additionally, a sensitivity check can be undertaken by varying the Elastic Modulus of the ground (E). It is expected that the lower bound results in the greatest predicted deformation of the pressurized tunnels, while the higher bound is used to predict the most conservative CPR. These CPR values show slight drops in magnitude as stiffness increases in both surge condition as well as normal operation condition. In addition, a check can assess the potential for opening of fissures and creation of a path of pressurized water between the pressurized and proposed tunnels. Fissure checks can be undertaken to show that the horizontal ground pressure is greater than internal water pressure at tunnel crown locations and sufficient to keep the fissures closed particularly during surge conditions.

CONCLUSION

Modeling for the analysis of over-tunneling of pressurized assets can be undertaken by sophisticated 3D analysis but this can be costly and time consuming.

Simplified 2D models and closed form analyses can be used and validated against empirical data. The method of analysis proposed allows calculation of movement around the tunnel and is therefore applicable to over-tunneling. The standard methods of analysis (e.g., O' Reilly and New 1982) are for prediction of ground movement above a new tunnel only.

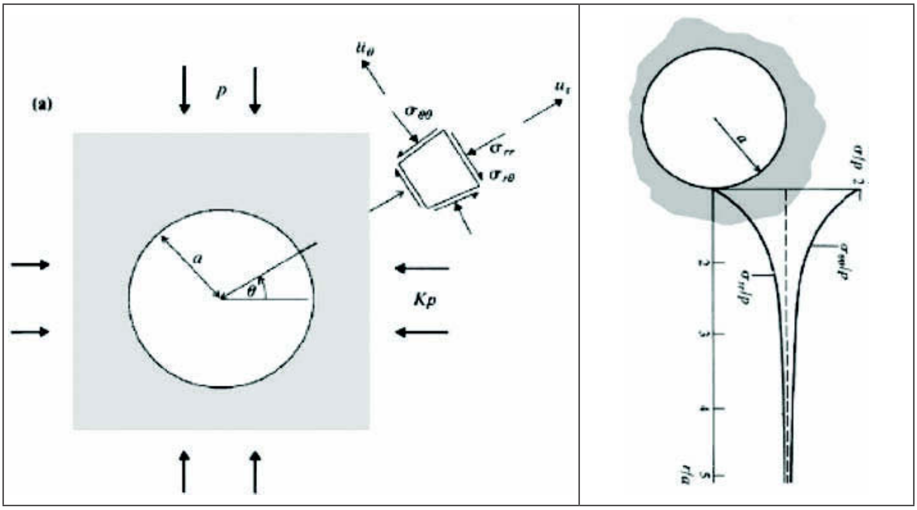

Figure 6. Change in stresses around a circular opening in elastic ground (Source: Kirsch 1898)

Changes in lining loads and ground pressures are undertaken with simple 2D models and coupling the numerical model and the closed form analysis allows simplicity and efficient replication the results that would be achieved from a 3D model.

It is recognized that there are limitations with numerical modeling and thus these are validated using closed form methods. This is the benefit of using this pseudo-3D approach to over-tunneling.

From analysis undertaken it can be seen that the closed form analysis and numerical modelling are in good agreement, both predicting a similar reduction in confinement around the pressurized tunnels. As a result any impact or lack thereof, of the integrity of the existing tunnels can be validated. Changes in the CPR (Confinement Pressure Ratio) of the pressurized tunnels following construction of the new tunnels can be compared.

ACKNOWLEDGMENTS

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NORTH STRATHFIELD RAIL UNDERPASS SHALLOW COVER DRIVEN TUNNEL

Ted Nye ▪ Mott MacDonald Australia Pty Ltd.

INTRODUCTION

The North Strathfield Rail Under Pass project is designed to grade separate 1.5km long diesel hauled freight trains from the electrified suburban rail network north of Sydney. The freight line will pass under three heavily trafficked suburban railway lines and one line currently not in use. The client and their consultants original concept was for a cut and cover tunnel that would take between three and five years to constructed because of the limited number of track possessions available during any one year. This paper describes other options considered before adopting an alternative 170m long highly skewed and very shallow driven tunnel under the operating railway tracks which will reduce the construction time to less than 12 months by minimising the need for track possessions.

DESCRIPTION OF PROJECT

The North Strathfield Rail Underpass is one of the first projects to be undertaken under the Northern Sydney Freight Corridor Program. This program includes a number of infrastructure projects to improve freight and passenger rail services along the 155 kilometre rail corridor between Sydney and Newcastle, and it is a federally funded project under the Nation Building Program.

A plan of the proposed skewed driven tunnel traversing the tracks is shown in Figure 1. The project involves the construction of a new rail underpass (driven tunnel) between North Strathfield Station and the Strathfield Junction which will allow the UP Freight movements to the Flemington Goods Loop to be provided via a route beneath the UP Relief, UP Main, DN Main and the DN Relief, thereby eliminating conflicting at-grade movements.

OPTIONS STUDY

As part of our Transport for NSW (TfNSW) Technical Advisor role as sub-consultants for the concept design phase various options were assessed with the primary aim of eliminating the cut and cover approach put forward by others in a previous study. Track possessions during the year are very limited with only four week-ends available including one long week-end of three days. These possessions have to be shared with equipment being used for maintenance rail works within the same rail corridor with no guarantee that these will not disrupt any surface works associated with the construction of the crossing. The works would also be complicated by the necessity for power outages of the OHW which supplies electrical power to the trains.

The tunnel alignment (UP Relief) is on a reverse curve of 400m radius with a down gradient from the north of 2.8% and up gradient heading south at 2.2%.

Option 1—Cut and Cover Tunnel

The cut and cover option involves placing standard pre-cast beams on bored piles placed along the tunnel boundary. The total length of tunnel is approximately 200m and

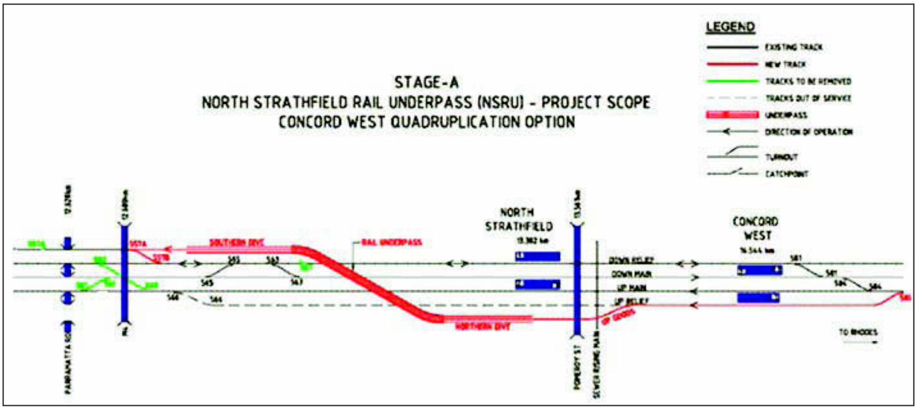

Figure 1. Plan of railway alignment showing indicative tunnel skewed across the rail corridor

will require the placement of approximately 190 piles to 278 standard beams (many of these piles were between the tracks). The top down method of construction will rely on the deck beams acting as props for the row of piles as the tunnel is excavated between. This method of construction will allow a shallower construction depth however it will need a considerable series of track possessions to complete which will extend the construction over many years. There are also significant risks to be addressed, namely piling adjacent and between tracks during possessions as well as the repetitive removal and reinstatement of equipment for piling and placement of concrete headstocks and beams. The other risk is the availability over an extended period of suitably experienced construction expertise.

Option 2—Hybrid Cut and Cover and Driven Tunnel

On the west side and east side of the rail corridor the tunnel would initially be constructed by cut and cover methods with the connection under the railway tracks between the two completed by a driven tunnel. All cut and cover works will require track possession, the driven tunnel would be excavated by a road header under the tracks. The short driven tunnel support would consist of canopy tubes installed ahead of the tunnel face and shotcrete forming the lining of the horseshoe section profile tunnel. The canopy tube method requires at least 1.6m of ground cover under the track ballast.

Option 2a—Interlocked Horizontal Steel Tubes and Driven Tunnel

To avoid tracks possessions, including piling between the Down Main and Down Relief tracks, on the west side of the corridor install perpendicular to the track alignment ~ interlocked steel tubes over the tunnel alignment previously designated for the cut and cover tunnel in Option 2. Once the steel tubes are installed the driven tunnel can proceed underneath. Excavate the driven tunnel in 1m long increments with a 300mm thick shotcrete lining following close behind the tunnel face. At the point where sufficient cover is reached the standard driven tunnelling method using forward canopy tubes can be used.

Option 3—Shallow Driven Tunnel with Alignment Moved North

The dive and tunnel alignment is moved north by approximately 60m with the northern dive structure extending beyond the Pomeroy Street road bridge. The east abutment of this bridge would have to be moved east as part of the required modifications to this

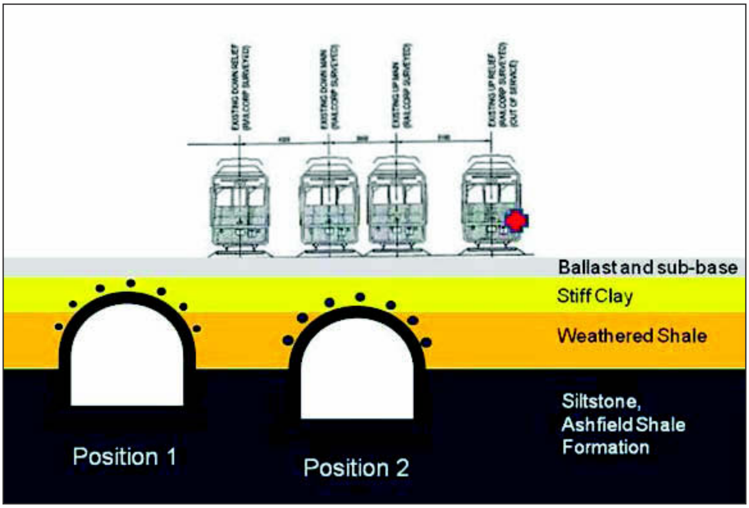

Figure 2. Schematic cross section showing tunnel and railway tracks, approx 2.5m maximum ground cover. Tunnel traverses the tracks on a skew.

bridge. This option will result in a deeper tunnel as there would be more room along the corridor to accommodate longer dive structures. The driven tunnel would be constructed using the standard tunnel construction method of canopy tubes and shotcrete (Figure 2).

Other Options

Other tunnelling methods have been considered but discounted, including a large jacked box, a Tunnelling Boring Machine(TBM) and an Open Shield but were rejected on the basis of practicality and cost.

The client, TfNSW directed that Option 3—Shallow Driven Tunnel be progressed into the concept design stage (which transitioned later into a reference design).

SITE INVESTIGATIONS

Site investigations borings and test pit excavations were carried out within the rail corridor during a long week-end track possession June 2011. The remainder of the boreholes beyond the danger zone to the operating tracks were completed over the weeks following this week-end.

Four boreholes were drilled near to or in between the railway tracks following the original cut and cover tunnel alignment. The boreholes drilled during the weekend possessions were drilled to a maximum depth of 11m using a small tracked drilling rig.

GEOLOGICAL MODEL

Our interpretation of the geological profile is that the tunnel will be excavated in very weak to strong rock, with stronger rock a minimum of 2.5m height above the tunnel invert within the excavation profile of the driven tunnel on the new alignment north of the original as mentioned above. Ballast, fill and residual clay are, for all practical purposes, considered being above the tunnel crown profile.

The rock units in the Table 1 consist of shale. The operational tracks are supported on concrete sleepers and are spaced at 600mm centres. Test pits were excavated

Table 1. Geological profile and parameters

#	Strata	Thickness	Description	UCS	Range E (MPa)
Material above the tunnel crown of the driven tunnel					
1	Ballast	500mm	Gravel and cobbles	—	100–200
2	Capping L-1	250mm	Clayey gravel/gravelly clay	—	50–100
3	Capping L-2	250mm	Silty gravel/ sandy gravel	—	50–100
4	Filling	0 to 1000mm	Mostly sandy gravel	—	60–100
5	Residual Clay	0 to 1000mm	Mostly stiff to very stiff residual clays	—	12–20
Material intersected at the tunnel face of the driven tunnel				**Min. UCS (MPa)**	**Range E (MPa)**
6	Rock Unit 1	1m to 3m in the tunnel face	Extremely low to very low strength, fractured to highly fractured	0.5	100–200
7	Rock Unit 2	1m to 4m in the tunnel face	Low to medium strength, fractured	2	300–500
8	Rock Unit 3	0.5m to 2.5m in the tunnel face	Medium to high strength, fractured to highly fractured	7	700–1000
9	Rock Unit 4	3m to 6.5m in the tunnel face	Medium to high strength, unfractured, RQD > 70%	16	2000–3000

between sleepers in critical locations along the track to determine the depths of the ballast and the ballast sub-base.

SETTLEMENT PREDICTION

Background

Settlement predictions that have been developed for this project and are based on the following methods:

- Case studies
- Two Dimensional Analysis–Finite Element Analysis (FEA)—Phases 2D
- Three Dimensional Analysis–FEA—Strand 7
- Engineering experience from previous shallow cover tunnels using the same construction method.

The case studies have been used to verify the FEA results and where necessary an “experience factor” applied increasing the FEA predictions of settlement.

Shotcrete is considered the most appropriate tunnel support to reduce surface settlement compared with steel sets alone (it can be used in conjunction with steel lattice girders if required). Further reading on this topic can be found in Nye 2012, Figure 3 and Table 2 have been taken from this reference.

Case Studies

Three cases studies that the author has had previous involvement through either access to all of the construction and monitoring data (Buranda, 1999) as a reviewer in both design and construction (M5 East Exit Ramp, 2002) or as the lead designer (Boggo Road Busway Tunnel, 2009) were used as examples to both help demonstrate

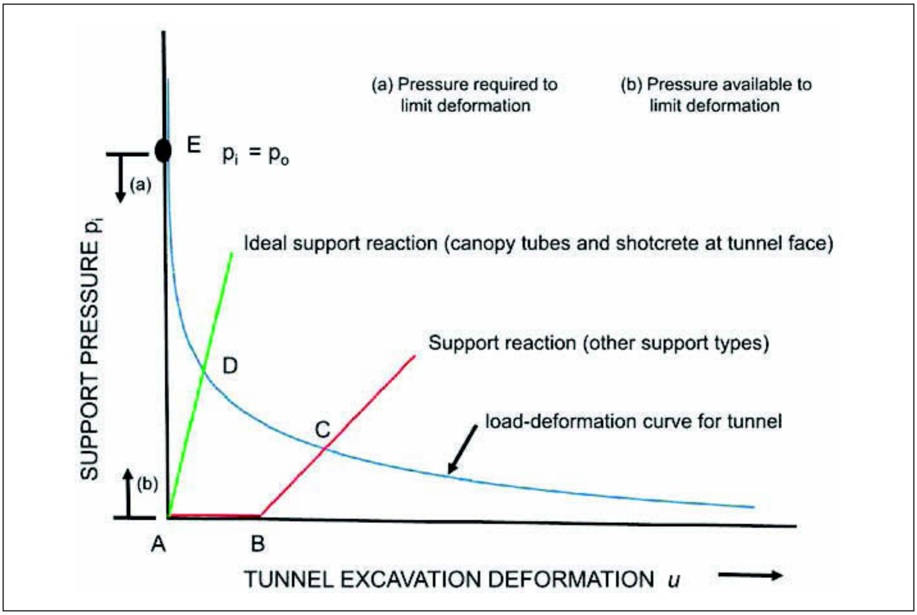

Figure 3. Support interaction diagram highlighting the relative stiffness of linings

Table 2. Comparing the axial stiffness of a shotcrete lining with steel sets/lattice girder

Material/Section /Age	Thickness or Section Size (mm)	X-area mm^2/m Length of Tunnel	Modular Ratio*	Equivalent X-Area (mm^2) to 28 Day Old Shotcrete	Axial Stiffness Ratio to Shotcrete at 24 Hours
Shotcrete 1 (12 hrs)	350	350,000	0.33	115,000	0.5
Shotcrete 2 (24 hrs)	350	350,000	0.66	230,000	1
Shotcrete 3 (3 days)	350	350,000	0.8	280,000	1.22
Shotcrete 4 (28 days)	350	350,000	1	350,000	1.52
Steel Section–1	150UC37	4,740	7	33,180	0.14
Steel Section–2	250UC89	11,400	7	82,600	0.35
Lattice Girder—3†	2×32,1×25 (15kg/m)	2,100	7	14,700	0.063

* the modular ratio of steel to 28 day old shotcrete is 7.
†Lattice girder–3 is the girder used on Boggo Road.

the feasibility of the driven tunnel option and also for the purpose of checking the settlement predictions. Further details of these projects can be found in the references (Gibbs et al. 2002 and Nye 2009, 2010).

Face Stability

Face stability is an important issue during the construction of the tunnel to avoid the excavated face slumping out in advance of the tunnel permanent shotcrete lining placement. To avoid this, the static stability of the tunnel face will be maintained by various methods including placement of face nails on a routine basis. Confining the face of the tunnel in this geological setting with these dowels and perhaps shotcrete will reduce the magnitude of surface settlement but this will not be significant compared to the overriding influence of the canopy tubes in combination with the stiff shotcrete lining which will closely follow the tunnel face excavation.

Theory–Finite Element Analysis

In addition to and to overcome the limitations of the 2D FE model, a 3D FE model was also developed in the concept design phase using the Strand7 FE structural software package. The 3D model includes the various ground strata and their modulus properties as well as the tunnel configuration. A sequenced analysis simulating the development of the tunnel has not been carried out. The model is representative of face of the tunnel when train live loads are applied. The model is also representative of the undisturbed ground (with train loading) but without the tunnel (a load set has been applied away from the influence of the tunnel).

It is noted that the model does not include the canopy tubes and therefore from a ground support perspective it is considered to be a conservative approach for analysis notwithstanding the possible limitations of a linear elastic FE analysis itself. It is also likely that the final tunnel lining will consist of a 350mm steel fibre reinforced shotcrete (spray-on waterproofing membrane plus a shotcrete waterproofing membrane protection layer).

The resulting surface displacement due to train loading alone using the 3D modelling gave surface settlements around 2.8mm, this was regardless of whether the loading was applied above the tunnel face or away from the tunnel face. Though the values are half of the 2D model the results are consistent in that tunnel has little influence on the settlement under the track due to a passing train.

Estimated Maximum Surface Settlement

It is predicted from a combination of the FE analysis, the case studies and previous direct project experience using the construction methodology proposed here that the surface settlement above the tunnel is likely to be small with a maximum settlement of up to 5mm. This settlement is within allowable limits defined in RailCorp's specification (Railcorp is the state government suburban train network operator).

The tunnel face from the tunnel invert has a minimum height of 3.5m of strong rock and above this layer there is still rock but of lower strength. The tunnel arch (Figure 4) therefore will be very stiff as it cannot spread out horizontally at the base of the arch and at the tunnel springline due to the confinement of the surrounding rock. So that in addition to the theoretical analysis, together with past experience of the construction method proposed and lining behaviour, there are other reasons to be reasonably confident that vertical settlements will not exceed about 5mm.

MONITORING PLAN

Deformation monitoring is a key mechanism to verify that the tunnel construction is being adequately controlled in accordance with allowable settlement limits. Prior to commencing any construction work on site a baseline survey must be undertaken. Surface settlement of the track will be carried out as per the RailCorp Specification SPC 207 (with any modifications as required and approved).

Tunnel construction monitoring data will be available before the full width of the tunnel passes under the first live railway track as up to 60m of tunnel from the south portal or 40m of tunnel from the north portal will have been completed prior to traversing under the railway tracks.

Continuous robotic survey methods are expected to be used when recording track surface movements. Automatic notification will occur at predetermined trigger levels (refer to Figure 5). The alarm levels 1, 2 and 3 are as set out in Railcorp specification

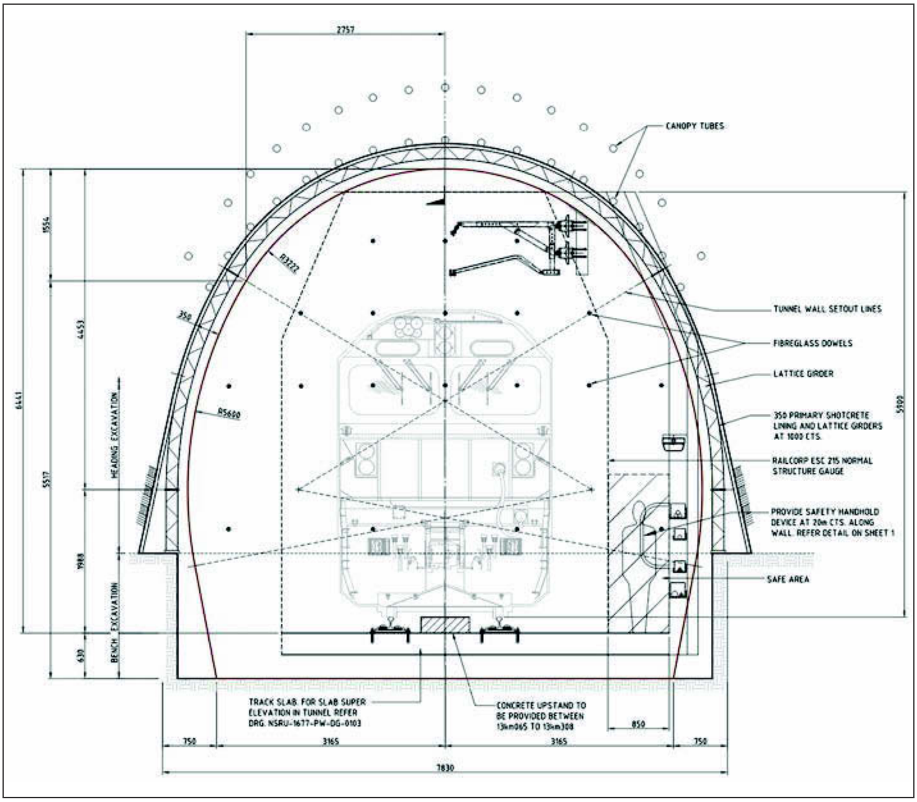

Figure 4. Tunnel cross section with lattice girders and shotcrete support over the arch

Figure 5. Typical robotic monitoring set up for track deformation

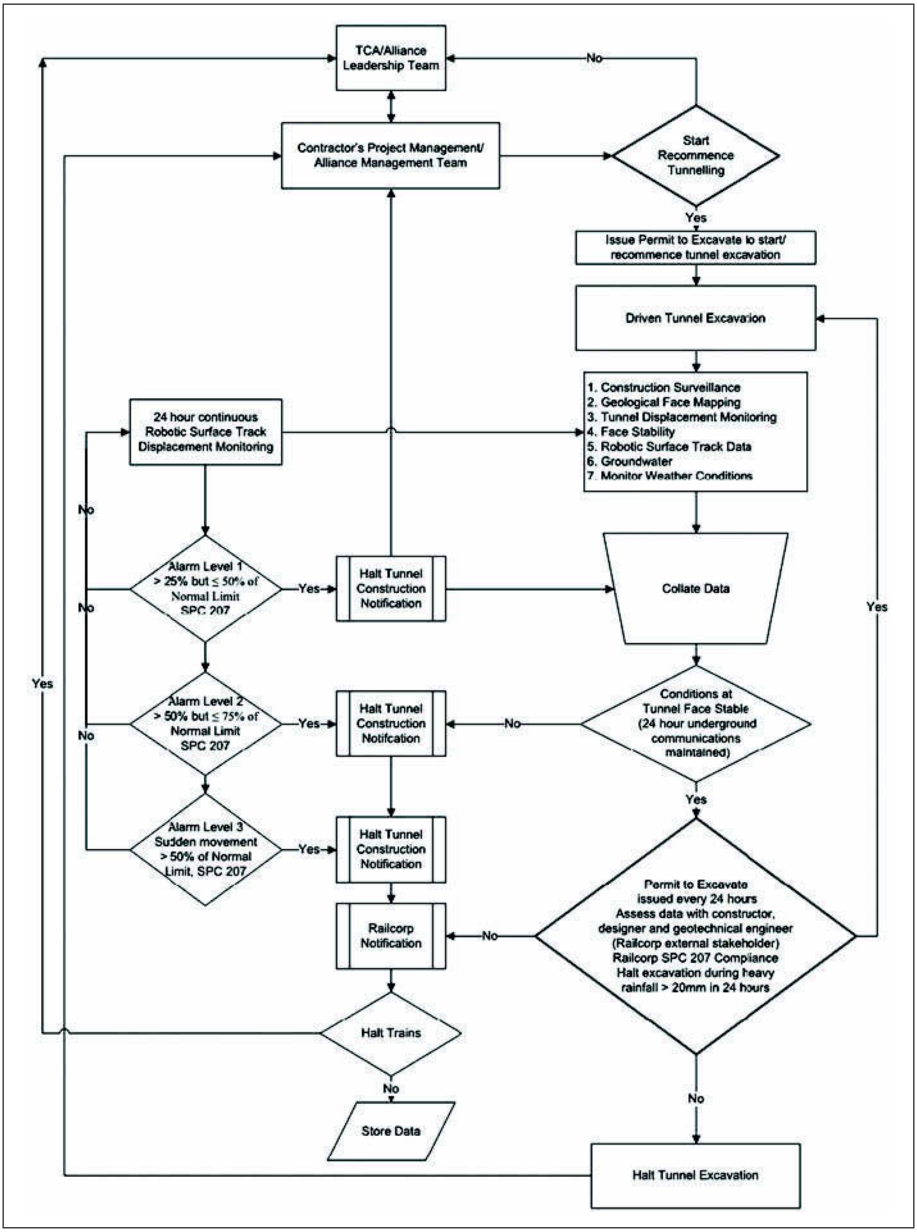

Figure 6. Schematic of recording and reporting procedure

SPC 207. The specification requires survey prisms every 2m in pairs along each rail. 100 prisms can be read in about 12 minutes. At North Strathfield there may be a requirement for perhaps 300 prisms requiring three instruments. Reading accuracy could be tolerable up to 100m sight distance; however, this will need to be assessed for the particular instruments proposed.

Figure 6 is a schematic of the proposed monitoring recording and reporting procedure. This is an important flow chart to be used to manage the process should any incident occur.

ACKNOWLEDGMENT

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CONCLUSIONS AND RECOMMENDATIONS

1. A driven tunnel option has been accepted as the preferred method of construction to shorten the construction period to months in place of an initial cut and cover option which required numerous track possessions and with a potential construction period extending over a number of years.
2. The driven tunnel construction method proposed has a reliable history of achieving both the predicted and low settlement values in similar weak rock tunnels. That is forward canopy tubes of the excavation and a stiff shotcrete lining kept near to the tunnel face.
3. The cross-sectional profile of the tunnel (with a pronounced curved arch) in combination with a relatively strong rock strata (at least up to the tunnel springline level, with weathered rock and stiff clay above) are favourable to achieving low settlement values.
4. The 2D finite element model predicts surface settlement in the order of less than one millimetre but in practice this in unlikely. The 3D FE model gave predicted settlements under train loading only and concludes that there is no significant difference between train loading surface settlement, whether there is a tunnel under the track structure or not. This is because of the very high stiffness of the tunnel arch profile.
5. Provided the geological model is accurate with shale rock at or above tunnel springline, the magnitude of surface settlement is greatly dependent on the early strength gains in the shotcrete, the construction sequencing being followed and the quality of construction workmanship. These four factors are considered the controlling drivers of the magnitude of settlement as the theoretical predictions are so low.
6. The real world settlements, ignoring the lower theoretical predictions would be expected to be around 5mm.

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PORT MANN MAIN WATER SUPPLY TUNNEL: IMPROVING SYSTEM RELIABILITY

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ABSTRACT

The Port Mann Main Water Supply Tunnel will increase Metro Vancouver's capacity to accommodate future regional growth while also improving overall system reliability. The project calls for construction of a 1,000 m long tunnel under the Fraser River between Coquitlam and Surrey in British Columbia, Canada. The tunnel, mined between two approximately 60 m deep vertical shafts, will be constructed using EPBM methods, and will be approximately 3.5 m in diameter. The tunnel alignment is located below the depth of riverbed scour and is designed to remain functional following the Maximum Credible Earthquake. This paper presents the project from planning through design completion, and describes some of the unique technical challenges encountered.

INTRODUCTION

Metro Vancouver provides a reliable source of safe, high-quality drinking water to over 2 million people in its 18 municipalities. This service includes acquiring and maintaining the water supply, operation of treatment facilities to ensure quality, and delivery of potable water to these municipalities. Water is collected from three mountainous watersheds: Capilano, Seymour, and Coquitlam, and delivered by an extensive system of 22 reservoirs, 15 pumping stations, and over 500 km (300 mi) of transmission mains.

The existing Port Mann Main–Fraser River crossing was constructed in 1974. It consists of a 1,200 mm (48 in.) diameter welded steel pipe approximately 1,000 m (3,300 ft) in length, crossing the Fraser River just downstream of the Port Mann Bridge. The crossing is a primary water supply link to municipalities south of the Fraser River. A location map is shown in Figure 1. The crossing was damaged by riverbed scour in May 1997, causing temporary but significant water supply problems to several municipalities south of the Fraser River during the summer of 1997. As it was recognized that a new crossing would be required in the future to address seismic and capacity deficiencies, the repair, completed in 1998, consisted of replacing only the damaged section of the water main and providing a protective apron to secure it against future scour and undermining. This temporary repair was intended to provide an acceptable level of service for about one to two decades, during which time the long-term options for a new crossing would be fully explored and a suitable course of action developed.

After extensive study, the Port Mann Main Water Supply Tunnel was selected as the long-term option that will replace the existing water main crossing. It is sized to accommodate future growth in the region, will be located below the depth of riverbed scour, and is designed to remain functional following a major earthquake with a return period of 10,000 years.

PROJECT OVERVIEW

During the mid-1990s, Metro Vancouver initiated a seismic review and retrofit program for its key facilities, and developed its own seismic design standard, which required critical infrastructure such as marine crossings remain operational after a 1:10,000 year event. As part of this process, it was identified that many of the critical service delivery marine crossings would not survive even a moderate earthquake, such as an event with a 475-year return period.

Prior to assessing the long-term options for a new crossing, Metro Vancouver conducted a seismic vulnerability assessment on the existing Port Mann crossing in 2001. The proposed crossing was assessed for damage potential when subjected to a seismic event with a 100-year return period, a 475-year return period, and the Maximum Credible Earthquake (MCE) event, with a return period of 10,000 years. Although no damage potential was identified for the 100-year earthquake, the study concluded that the crossing would fail at several locations during the 475-year or the MCE earthquake. It was determined that the subsoil would liquefy, causing permanent ground deformation along both riverbanks towards the river. These conclusions are generally consistent with observed damage to pipelines during past earthquakes in places such as California and Japan.

To address this concern, Metro Vancouver performed a preliminary design of a new trenchless crossing in 2002–2004. This work, which was based on several boreholes and other limited local geotechnical information, indicated that a new crossing would likely need to be trenchless and at a depth of at least 30 m (100 ft) beneath the river bottom to avoid future river scour (1:200 year event) and to withstand an MCE seismic event (0.5g at that time). As part of the study, several different trenchless methods and different pipe diameters were evaluated for construction of the new pipeline crossing. Trenchless methods considered were horizontal directional drilling (HDD), microtunneling, and tunneling using a pressurized face tunnel boring machine (TBM). A 2,134 mm (84 in.) diameter welded steel pipe installed in a tunnel, constructed using a pressurized face TBM, was eventually selected as the optimal water main crossing replacement solution.

Prior to the start of detailed design, Metro Vancouver undertook a conceptual level comparison of trenchless, trenched, and aerial crossings for Port Mann to ensure that a TBM-driven tunnel was the appropriate solution. The study concluded that if Metro Vancouver wanted to meet its established criteria of pipe size,

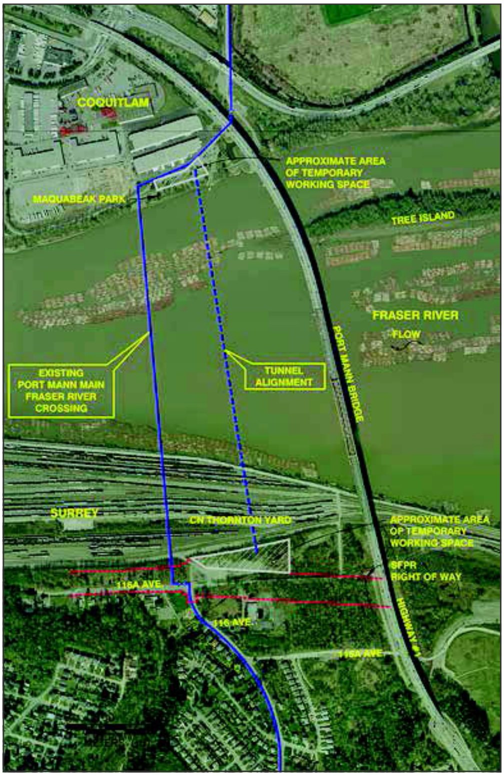

Figure 1. Port Mann water supply tunnel location plan: Fraser River crossing

scour resistance, and earthquake resilience, then a trenchless crossing should be pursued.

The Fraser River Tunnel Group, which included Jacobs Associates, then commenced detailed design of the project in late 2006; design was completed by December 2009. An extensive geotechnical exploration program—combined with significant seismic analyses plus geotechnical and structural modeling—was undertaken to complete the design. Environmental, archaeological, social, and public impact assessments were also carried out during the detailed design phase. Additional geotechnical explorations were carried out in 2010 at both the south and north shafts in order to provide data below the full depth of the proposed shaft wall panels.

The final project configuration consists of a 3.5 m (11.6 ft) outside diameter bored and segmentally lined tunnel mined between two deep vertical shafts, which will connect to the existing Port Mann Main on the north and south banks of the Fraser River. Two new valve chambers will be constructed integral to the concrete shaft lining to control water flows through the tunnel. The tunnel will be approximately 1,000 m (3,300 ft) long and sized to install a 2,134 mm (84 in.) diameter full joint penetration butt-welded steel water main. The tie-in piping required to connect the new tunnel to the existing water main will consist of a 1,220 mm (48 in.) diameter butt-welded steel pipe. Provisions are included in the design to allow for a future twin 1,524 mm (60 in.) diameter water main to connect into the tunnel when required because of increased demand.

Shaft construction will be facilitated by the installation of unreinforced concrete slurry walls prior to excavating the interior soils and installation of a cast-in-place reinforced concrete lining. Muck handling, materials stockpiling, heavy equipment, office facilities, and labor parking will be located at the TBM launch shaft on the south side of the river. The steel water main will be installed in the tunnel via the south shaft.

Procurement of the tunnel contractor commenced in 2010, and the contract was awarded to a joint venture of McNally and Aecon in mid-2011. At the time of the writing of this paper, the contractor had completed the south shaft excavation and the north shaft slurry wall construction, and the TBM was being manufactured in preparation for delivery to the site in early summer of 2013.

SUBSURFACE CONDITIONS

Geomorphology

The project site is within the lower reaches of the Fraser River Valley, about 35 km (22 mi) east (upstream) of the present mouth of the Fraser River in Georgia Strait. The valley is bounded by the Coast Mountains to the north and the Cascade Mountains to the southeast. The topography at the project site ranges from approximately El. +5 m (+16 ft) on the north river bank at the north shaft to El. –19 m (–62 ft) near the center of the main river channel. The south river bank in the general vicinity of the south shaft is approximately El. +4 m (+13 ft).

Tectonic Setting and Seismicity

The project is located in a seismically active area. The site seismicity results from the thrusting (subducting) of the Juan de Fuca plate beneath the Continental North American Plate. The offshore plate tectonic setup has resulted in shallow crustal earthquakes occurring within the Continental plate, deep intraplate earthquakes occurring in the subducting plate, and interplate earthquakes occurring at the contact between the plates. Over the past several decades, intraplate earthquakes have occurred at regular intervals—Campbell River (M7.3, 1946), Olympia (M7.1, 1949), Seattle/Tacoma (M6.5, 1965), and Nisqually (M6.8, 2001).

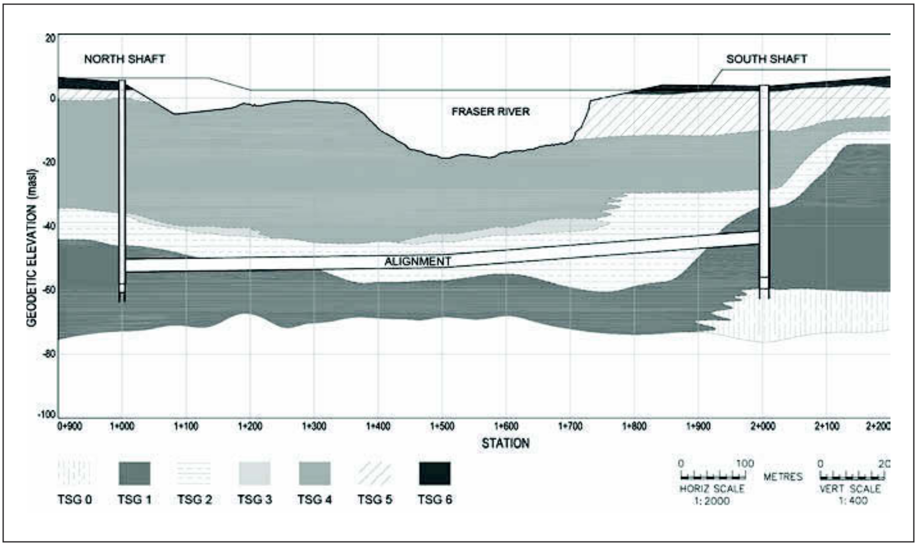

Figure 2. Tunnel baseline conditions

A site-specific seismic hazard study completed for the project area indicated intraplate and interplate earthquakes dominating the seismic risk at the site, with the intraplate earthquakes controlling the seismic risk for periods less than about 1 second and both intraplate and interplate earthquake contributing similar seismic risk for periods of longer than 1 second. The controlling earthquake scenarios included a M7.25 intraplate earthquake occurring at a distance of 55 km (35 mi) and an M8.8 interplate subduction earthquake occurring at a distance of 160 km (100 mi) from the area.

Regional Geology

The sediments that fill the Fraser River Valley were deposited during glacial, interglacial, and postglacial periods over the last 120,000 years. The tunnel and shafts for the project will be excavated through these sediments, as well as through fill deposits that mantle both river banks. The thickness of the interglacial, glacial, and postglacial sediment layers varies along the tunnel alignment because of uneven deposition and localized erosion.

The Geological Survey of Canada identified six geologic units in the project area. They are described (in order of increasing depth and age) as:

- Fill (layer of widely varying natural and man-made materials)
- Salish Sediments (postglacial bog, swamp, and shallow lake deposits)
- Fraser River Sand (postglacial channel and floodplain deposits)
- Capilano Sediments (glaciomarine and marine deposits)
- Semiahmoo Drift (lodgement till and glaciofluvial deposits)
- Highbury Sediments (interglacial silt, sand, and gravel deposits)

The stratigraphic profile for the proposed tunnel alignment is shown in Figure 2. See "Subsurface Characterization" for a description of Tunnel Soil Groups (TSGs).

Groundwater

Hydrogeological conditions at the project area are controlled by the combined influences of local topography, complex geology, and Fraser River tidal cycles. The upland areas that rise more than 100 m (330 ft) above the Fraser River, both north and south of the project area, are regional-scale topographic features that are also regionally significant groundwater recharge areas. The water table below the upland areas is elevated approximately 30 to 50 m (100–165 ft) above the Fraser River level, and groundwater generally flows toward the Fraser River. Groundwater flow is physically constrained by the presence of relatively low permeability sediments below the Fraser River floodplain, resulting in zones of highly pressurized groundwater. This pressure induces a partial upward groundwater flow gradient.

Piezometric levels above the ground elevation were recorded at various depths in the till-like sediments and in the underlying silt and clay deposits; these levels are indicative of artesian conditions at the south shaft site. Monitoring in standpipe piezometers installed near the North Shaft also confirmed the presence of artesian pressures within the till-like sediments, although the degree and extent of artesian conditions are less than at the south shaft.

Continuous monitoring of groundwater pressures has confirmed the influence of the Fraser River tidal surge on piezometric levels at both shafts. Monitoring data indicate a strong correlation to tidal cycles in the upper soil deposits, with corresponding piezometric levels varying within about 1 m (3 ft).

Geotechnical Exploration

A total of fourteen boreholes (using sonic and mud rotary methods, on land and over water) and six cone penetration tests were drilled during the detailed design stage to characterize the expected soil conditions at the shaft sites and along the proposed tunnel alignment. These supplemented another five boreholes that were drilled in the project area during the earlier preliminary design stage. During construction, additional borings were completed at each shaft site to install inclinometers and piezometers for ground monitoring. The in situ soil tests conducted during the investigation consisted of:

- Field vane tests
- Standard Penetration Tests (SPTs)
- Cone Penetration Tests (CPTs)
- Seismic Cone Penetration Tests (SCPT) with shear wave velocity measurements
- Measurement of piezometric pressures
- Hydraulic conductivity measurements

Laboratory testing of select soil samples consisted of:

- Soil classification tests (particle size, Atterberg limits, water content, organic content, unit weight)
- Consolidation tests
- Monotonic and Cyclic Simple Shear tests
- Bender Element tests
- Abrasion and x-ray diffraction tests
- Laboratory permeability tests

Subsurface Characterization

Based on the results of the geotechnical exploration program, seven Tunnel Soil Groups (TSGs) were defined to represent the soils expected within the shaft and tunnel excavations. They are summarized as follows:

- TSG0: Silty Clay to Clayey Silt. TSG0 consists of clays and silts deposited in proglacial lakes during glacial advance. Because of the geologic connection between TSG0 and upland recharge zones, this unit is characterized by artesian conditions (piezometric level about El. +13 m [+43 ft]).
- TSG1: Silty Sand, Sand and Gravel, and Silty Clay. TSG1 is a till-like, very dense or hard poorly sorted, heterogeneous mixture of clay, silt, sand, gravel, cobbles, and boulders. Like TSG0, it also has an artesian piezometric level (about El. +5 m [+16 ft]).
- TSG2: Silty Clay. TSG2 overlies TSG1 and typically consists of soft to very stiff silty clay to clayey silt with varying plasticity. Cobbles were encountered infrequently throughout this TSG. This layer is about 9 m (30 ft) thick at the north shaft and thickens to the south to a maximum thickness of about 30 m (100 ft). Scattered layers and/or lenses of coarse-grained soils are also part of this unit.
- TSG3: Gravel. A relatively thin (about 1.0 to 2.5 m [3 to 8 ft] thick), and flat-lying gravel bed forms the upper contact of TSG2 along almost the entire tunnel alignment. The gravel layer is compact to dense, and contains variable amounts of sand. This TSG is entirely above the crown of the tunnel, and will only be encountered within the north shaft excavation.
- TSG4: Sand with Gravel, and Silt Interlayers. TSG4 consists of clean sand containing a trace of to some gravel. The sands are very loose to very dense, although they are typically compact. Scattered within TSG4 are layers and lenses of silts and clays. Cobbles were also encountered infrequently in this TSG in the project borings. TSG4 thickness is greatest along the north side of the tunnel alignment boreholes—ranging from 18 m (59 ft) at the south shaft, to a maximum of 35 m (115 ft) at the north shaft. TSG4 overlies TSG2 and TSG3, and is not expected within the tunnel excavation.
- TSG5: Peat, Sand, and Silt. TSG5 consists of loose to compact (or firm to stiff) interbedded sand, silt, and amorphous and fibrous peat with wood fragments. This unit overlies TSG4. The unit increases in thickness towards the south, ranging in thickness from approximately 2 m (6 ft) thick at the north shaft, to 13 m (43 ft) thick at the south shaft. TSG5 will be encountered within the north and south shaft excavations, but not in the tunnel excavation.
- TSG6: Fill. TSG6 consists of fill deposits that mantle alluvial deposits along both the north and south Fraser River shorelines. About 3 m (10 ft) and 2 m (6.5 ft) of fill are expected within the north shaft and south shaft excavations, respectively. The fill is a heterogeneous mixture of compact to dense sand, gravel, cobbles, rubble, and organics.

Rock consisting of sandstone, siltstone, and mudstone exists below the soil deposits at each shaft site.

DESIGN CONSIDERATIONS AND CHALLENGES

Seismic Design Criteria

Because of limited system redundancy, Metro Vancouver classified the new tunnel crossing as a “Level 1” facility, which is required to withstand and remain functional following the MCE earthquake. The following seismic design scenarios for the new crossing and associated facilities were adopted:

- Operating Basis Earthquake (OBE) that corresponds to ground motions with a peak firm-ground horizontal acceleration (PFGHA) of 0.5g. Under this level of shaking, the facilities are expected to exhibit near-elastic response with no damage;
- Maximum Credible Earthquake (MCE) that corresponds to a return period of 10,000 years with a PFGHA of 0.7g, as per a site-specific seismic hazard analysis completed by Abrahamson (2006). Under this level of shaking, the facilities may experience some distress, such as cracking and minor leakage, but are expected to remain operational at full capacity.

For all design earthquake scenarios, the peak firm ground vertical accelerations (PFGVA) were to be taken as two-thirds of the corresponding PFGHA.

Seismic Analysis and Geotechnical Modeling

Site-specific ground response analyses were undertaken to quantify the magnitude and pattern of ground deformations resulting from the design earthquake scenarios. Of particular interest were the profiles of permanent ground deformations caused by earthquake-induced soil liquefaction occurring towards the Fraser River and the resulting interaction with the shafts and tunnel.

The ground response analyses were undertaken using two-dimensional (2D) methods of analysis (FLAC) that considered a geotechnical model extending approximately 1.5 km (0.9 mi) across the river, including both banks, and some 200 m (650 ft) on either side of the shoreline. The lower boundary of the model was extended to firm ground (glacial till) encountered at depths varying from 35 m (115 ft) on the south bank to in excess of 50 m (165 ft) on the north bank.

The soil behavior prior to, during, and following strong shaking was simulated using the user-defined constitutive model UBCSAND developed at the University of British Columbia, Vancouver. The model was calibrated using element tests to reproduce the empirical liquefaction resistance charts established by Idriss and Boulanger (2008). Both uncoupled and coupled soil-structure interaction analyses were undertaken, considering varying permutations and combinations of ground displacement profiles, compliance springs that represent the nonlinear soil reaction at the soil-structure interface, and structural properties.

The seismic analysis involving decoupled analyses consisted of a geotechnical 2D FLAC analysis of the entire crossing to determine, amongst other things, postliquefaction free-field soil displacements for the MCE event; and a structural SAP2000 3D nonlinear analysis of the entire crossing to determine the effects of the soil displacements and inertial loading on the shafts and tunnel.

The free-field permanent ground displacements obtained from the FLAC analyses were applied to the shafts and the tunnel in the SAP2000 structural model by means of constant displacements applied to the end of nonlinear soil springs. The free-field displacements used in the analyses were taken at the end of the shaking for the MCE ground motions considered. Springs were provided for all translational degrees of freedom for both shafts and the tunnel, to capture all variations in soil properties along the height of each shaft and along the length of the tunnel.

The analysis was carried out for two main cases, each with six different MCE input ground motion time histories:

- The “Base Case” refers to FLAC analysis using properties of the clay layer with a reduced effective shear modulus of $G_{max}/5$.

- The "Sensitivity Case" refers to the FLAC analysis with no reduction in the shear modulus (G) of the clay layer.

Structural Analysis

The SAP2000 analyses considered the nonlinear behavior of the soil and its interaction with both shafts and the tunnel. To determine the forces and deflections in each structure, reduced cracked section properties in accordance with ACI 318 guidelines were used. The structural analysis was performed for both the Base Case and Sensitivity Case clay properties for each of the six MCE input ground motions, resulting in a total of 12 load cases.

The results of the structural analyses indicated that the south shaft displacements could be accommodated by a strength-based design in the linear elastic range of material response. However, the profile and magnitude of ground displacement at the north shaft resulted in the requirement for a strain/performance-based design in the nonlinear range of material response.

South Shaft

The south shaft is not sensitive to the free-field displacement, having a capacity able to match the demand for all 12 load cases. The design requires that 3.5% of longitudinal steel reinforcement in the final concrete lining could provide the necessary bending capacity for the south shaft for all 12 earthquake load cases. This level of reinforcement is only required over a length of some 18 m (60 ft) at the location of maximum bending, and can be reduced and optimized elsewhere. The required capacity for shear also can be achieved within code requirements.

North Shaft

The required bending moment capacity of the north shaft (nonlinear design) was accommodated with 2% longitudinal reinforcement in the final concrete lining for all load cases, except for one case, which requires over 4% longitudinal reinforcement in local areas along the height of the shaft. For several Base Case loading conditions, the shear demand on the north shaft exceeds the capacity of the section using maximum allowable conventional reinforcement. For all Sensitivity Cases, the shear demand on the north shaft is about two to three times greater than the shear capacity of the section using maximum allowable conventional reinforcing details.

For the north shaft to behave as designed, composite action between the final concrete shaft lining and the slurry wall must be avoided. Composite action would increase the bending moment, thus increasing the shear to levels that exceed the shear capacity. To minimize the composite action between shaft structural wall and slurry wall, a low friction plastic liner will be placed between the two; this liner is designed to permit longitudinal "slippage" yet transfer lateral soil loading through the slurry walls to the shaft structural wall. Construction tolerances for the liner are tight, to ensure the expected performance is achieved.

Shaft Dimensions

The dimensions of the shafts are largely controlled by constructability issues. The shaft initial support was designed to accommodate all appropriate load cases during shaft excavation, tunnel excavation and lining, and water main pipe installation.

The working shaft needs to provide clearance for delivery, launching, and operation of the TBM, including muck transport and lining delivery, and installation of the water main pipe. Considering the expected TBM diameter and welded steel pipe lengths of up to 8 m (26 ft), an excavated diameter of approximately 15.5 m (50 ft) was selected.

The receiving shaft diameter was driven by clearance for removal of the TBM once it reaches the Coquitlam side of the Fraser River, and by water main pipe vertical clearance within the shaft. Based on these considerations, the receiving shaft excavated diameter was selected as 10.5 m (35 ft). Allowance for the slurry walls and concrete shaft lining resulted in completed inside diameters of 11 m (36 ft) and 5 m (16 ft) for the working and receiving shafts, respectively.

Tunnel Dimensions

The excavated diameter of the tunnel is governed by several factors, such as:

- Tunnel excavation equipment clearances
- Initial lining thickness, including annular gap
- Water main pipe backfill material thickness
- Internal diameter and thickness of the water main pipe

To allow for the installation of the 2,134 mm diameter welded steel water main, the following nominal design dimensions were selected:

- Steel pipe wall thickness: 25 mm (1 in.)
- Backfill material thickness: 225 mm minimum (9 in.)
- Segmental lining thickness: 250 mm (10 in.)
- Annular gap (outside segmental lining): 100 mm (4 in.)

Based on the above factors, the minimum tunnel diameter selected for design was 3.25 m (10.5 ft), which represents a mid-sized TBM. The excavated diameter of the TBM proposed by the tunnel contractor will actually be around 3.5 m (11.6 ft). Typically, segmentally lined tunnels that incorporate a secondary pipe are larger than 3 m (10 ft) in diameter, in order to provide sufficient working space and to avoid construction logistics bottlenecks. A typical tunnel cross section is shown in Figure 3.

Temporary Ground Support: Slurry Walls

The shafts were designed to resist the external earth and groundwater pressures, and surcharge pressures applied from construction equipment. Several ground support methods were considered, and because of the depth, piezometric levels, and potential seepage, slurry diaphragm walls were selected. The slurry wall system is considered to provide the best combination of strength, stiffness, and groundwater control while maintaining tight verticality tolerances. Aside from keyway reinforcement where the tremie slabs tie in, the slurry walls are designed with no steel reinforcement; this is to minimize the influence on the final shaft lining during an MCE event, where ground deformation could result in large lateral displacement of the shafts at the ground surface.

Groundwater was a key consideration in the design of the shafts. The high piezometric levels affected the depth of the slurry wall and raised concerns about bottom stability and slurry wall construction. Dewatering was considered as a method to reduce the artesian water pressure in TSG1, but was not deemed feasible because of the large estimated volumes of water (4,000 liters per minute [1,055 gpm] per well), issues related to discharge (permits, treatment, and location), and concerns over ground settlement caused by dewatering.

Calculations showed that it was possible for the slurry pressure to exceed the artesian water pressure at the bottom of the slurry wall, provided a heavy slurry and a small starter berm at the ground surface were specified. A slurry specific gravity on the order of 1.1 to 1.15, combined with a slurry level of about 1.5 m (5 ft) above the existing ground surface, would provide a slurry pressure that was about 10% greater than the groundwater pressure at the south shaft, which necessitated the requirement

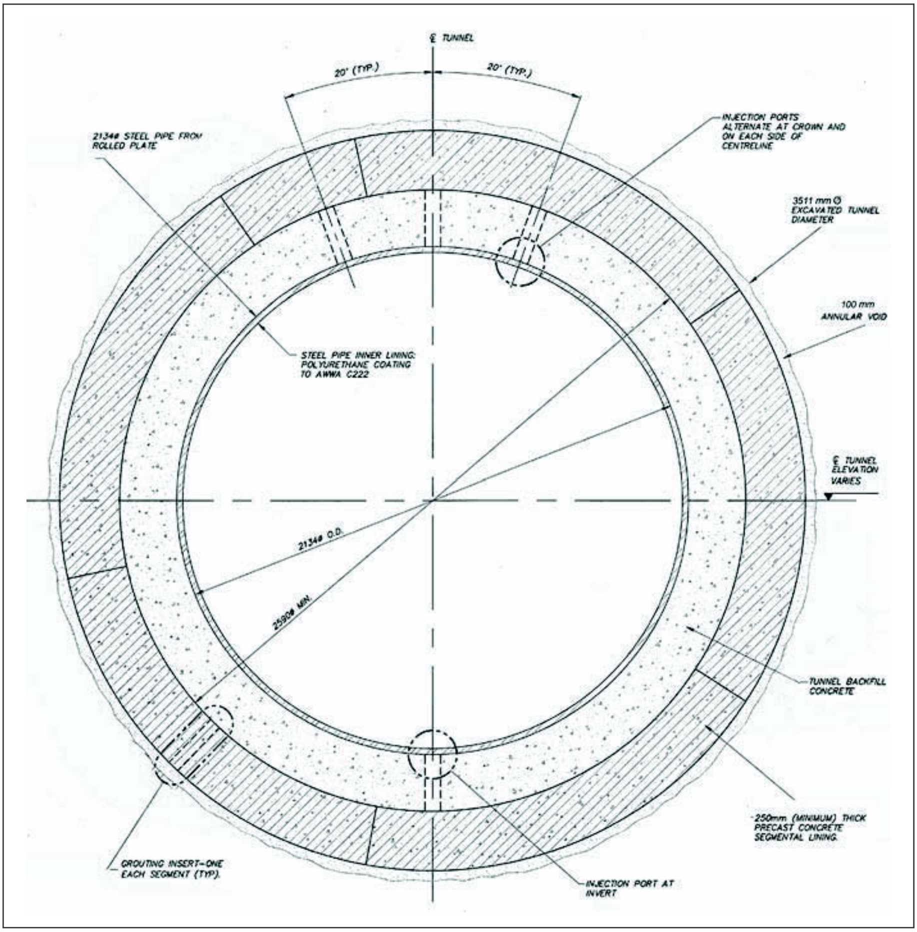

Figure 3. Tunnel cross section

to construct a starter berm. Because of the lower artesian pressures at the north shaft, a starter berm was not necessary, although the higher specific gravity slurry was still required.

Instability of the shaft bottom due to heave and piping was an additional concern at both shafts. Tremie concrete slabs are required to seal the bottom of each shaft, increase the resistance to bottom instability, and minimize groundwater inflows into the shafts. The original design included a provision for ground improvement around the bottom of each shaft to lower the permeability in order to improve bottom stability and reduce water inflow as the shaft was dewatered. Following discussion with the contractor, the decision was made to deepen the slurry walls to clay at the south shaft and bedrock at the north shaft, which eliminated the need for ground improvement.

The slurry wall forms the initial lining of each shaft. Once the shaft is excavated, a final cast-in-place lining will be constructed within each shaft. The final lining will provide the long-term resistance to earth and groundwater pressures as well as the resistance to seismic loading.

Seepage Criteria During Construction

It is desirable to prevent, or at least substantially limit, seepage (infiltration) into the tunnel during tunnel excavation and backfilling of the water main pipe. This is particularly true in the case of tunnels being excavated downgrade. Infiltration may occur at the excavated face, along the tunnel excavation prior to lining installation, and from the completed lining.

In order to control groundwater ingress, pressurized-face tunnel equipment will be required. In addition, the initial tunnel lining system is designed to minimize infiltration, which is typically achieved by specifying a bolted and gasketed segmental lining as the initial support system.

The maximum infiltration amount allowable was specified based on standard industry practice for the proposed excavation and lining means and methods.

The water main pipe will be pressurized and will therefore be fully watertight (i.e., zero infiltration and exfiltration), which is achievable with the specified welded steel pipe carrier pipe system.

Welded Steel Water Main Pipe

The welded steel water main pipe will be 2,134 mm in outside diameter with a 25 mm (1 in.) wall thickness. It will be fabricated from ASTM A516 Grade 70 steel, with a 260 MPa (37,710 psi) yield strength. Analyses show that the carrier pipe inside the south shaft remains elastic for all MCE load cases; however, because of the high displacement demand under an MCE event, the pipe inside the "hinging" north shaft enters its nonlinear range of material response. Therefore, the final analysis and design of the pipe were performed using ABAQUS, a commercial finite element program suitable for the most challenging nonlinear simulations. The nonlinear analysis of the pipe inside the north shaft includes the self-weight of the water-filled pipe, the internal pressure, temperature change of –13°C, and seismic displacement/rotations of the shaft.

The pipe inside the north shaft is only connected to the shaft/valve chamber at its two extreme ends. The bottom of the pipe is encased within a concrete plug near the base of the shaft at El. –49 m (–161 ft); it rises vertically into the valve chamber, then has a 90 degree bend and runs near horizontally within the valve chamber; the pipe bifurcates into two smaller diameter pipes within the valve chamber with centerline elevations of El. –0.2 m (–0.7 ft) for a 1,372 mm (54 in.) diameter pipe and El. –0.12 m (–0.4 ft) for a 1,220 mm (48 in.) diameter pipe; the two pipes are connected to the end wall of the valve chamber. Pipe layout in the south shaft is similar.

Design analyses indicate that certain sections of pipe within the tunnel will be subjected to significant tensile and compressive loading due to ground deformations during an MCE event. It is therefore important to require full joint penetration butt welds (joining pipe sections) to ensure that the welds are the same strength as the body of the pipe. Special welding requirements are included in the pipe specifications to deal with this issue.

SOCIAL AND ENVIRONMENTAL CONSIDERATIONS

The areas in which both the north and south shafts are located are quite congested, with limited property available to provide adequate working space. The south shaft is located in a narrow wedge of land between a large rail yard and a new freeway (South Fraser Perimeter Road). This site is 1.1 hectares (2.7 acres), making it very tight for construction and laydown. The north shaft is located in a narrow municipal park between an industrial park and the Fraser River on 0.43 hectares (1 acre) of land, also very tight. Because of the proximity of the shafts to the Fraser River and other fish-bearing water courses, numerous environmental impact mitigation measures were built into the project, including water treatment plants, collector drains, physical protection, and regular maintenance and reporting of water quality data. A requirement of the north shaft host municipality is to keep the park in which the shaft is located open to the public during construction. Access to the park needs to be safely shared among the tunnel contractor, other contractors working on a large nearby project, and the public. This was addressed through reorientation of roads, fencing, and signage.

SUMMARY

The new Port Mann Main Water Supply Tunnel will be an essential component of the Metro Vancouver system to assure a reliable supply of potable water to municipalities south of the Fraser River. It has been designed to resist a major earthquake, avoid river scour, and allow for growth in demand. This project has presented some unique technical challenges to the design team and Metro Vancouver. These challenges included detailed modeling and analysis of both the expected ground behavior and the performance of the shafts and tunnel structures during an MCE event. The design philosophy selected will ensure that the operational requirement that the facility remains fully functional following the MCE is successfully achieved.

ACKNOWLEDGMENTS

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TUNNEL DESIGN FOR THE WATERVIEW CONNECTION PROJECT

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ABSTRACT

The Waterview Connection project will complete Auckland's Western Ring Route, which will provide a high quality strategic alternative to State Highway One for travel through and within New Zealand's largest city. The design, construction and initial operation of a new 4.8 km section of motorway connecting State Highways 20 and 16 is the New Zealand Transport Agency's largest and most challenging transport project ever. The Waterview Connection project will construct twin three-lane motorway tunnels 2.4 km long and 13.1 m internal diameter. Tunnelling is scheduled to begin, using an earth pressure balance tunnel boring machine in late 2013. The tunnels will pass beneath an urban area and a major local arterial road. This paper describes the overall design and construction for the underground works. It summaries the design analysis, the basis for choosing the tunneling boring method rather than a sequential excavation method, and the proposed construction sequence. An invert culvert was requested by the client post-tender, to provide access to utilities during operations. As a result, an innovative construction process was developed for invert construction behind the TBM that is independent of tunnel excavation advance rates. The design and proposed construction of the sixteen cross-passages linking the two tunnels using sequential excavation method, each about 5 m in diameter, is described.

INTRODUCTION

The Waterview Connection is the New Zealand Transport Agency's (NZTA) largest and most challenging project to date. It will deliver an extra 5 km of six-lane motorway through and beneath the city's western suburbs to link State Highways 20 and 16. The project will complete a high-quality, 48 km, strategic alternative to SH1 to reduce congestion in central Auckland, New Zealand's largest city and economic powerhouse. The Western Ring Route was prioritised by the government as a Road of National Significance that will stimulate economic growth regionally and nationally and help to shape the future development of Auckland.

The Waterview Connection project scope includes the design, construction and commissioning of the missing motorway link. The tunnel portion of the new link will be completed using a custom-built, 14.5 m diameter earth pressure balance (EPB) tunnel boring machine (TBM). The EPB TBM will bore twin tunnels up to 45 m deep, passing beneath the layers of basalt that remain from the region's past volcanic activity.

The NZTA chose an alliance procurement model as the most appropriate method to deliver a project of this complexity and significance. Under this model, the participants—including the client—work together following principles of honesty, equality and agreed accountabilities to promote innovative thinking and outstanding results.

Figure 1. Project location

Following a competitive tender process, NZTA, as the client, appointed a combined local and international consortium, known as the Well-Connected Alliance (WCA), to manage the design, construction and initial operation of the Waterview Connection. The Alliance comprises Fletcher Construction, McConnell Dowell Constructors, Parsons Brinckerhoff NZ, Beca Infrastructure, Tonkin and Taylor, Japanese construction company Obayashi Corporation and the NZTA. This alliance brings together the knowledge and strong, home-grown reputation of leading New Zealand engineering companies with the tunnelling expertise of its international partners.

This paper summarises the tunnel design and construction methodology used on the Waterview Project. A specific part of the conceptual design effort was to develop the geological interpretation in more detail, which allowed alternative designs for the various methods of construction to be reviewed against cost and risk factors, including the sequential excavation method (SEM) and a number of TBM options. Multicriteria analysis concluded that tunnel construction by EPB TBM best addressed the project's geotechnical risks and uncertainties while minimising cost and program risk. Criteria considered during this evaluation were safety, cost, program, risk/opportunity, construction sequence and sustainability.

Excavation by TBM reduces both the magnitude and extent of anticipated settlement compared with SEM. The building damage category assessment estimated the number of properties potentially susceptible to damage was reduced from 39 to 17. Impacts on services and infrastructure are also similarly reduced.

The TBM will also allow WCA to make full use of our team's local and international strengths in EPB tunnelling to achieve major savings. Member organisations of our consortium are involved in the design and construction of several major large diameter EPB tunnel projects and Obayashi, in particular, has developed unique construction skills to successfully operate large diameter EPB TBM at shallow depth. Team members also have recent EPB TBM experience in Auckland excavating tunnels in East Coast Bays Formation (ECBF) (Figure 1).

TUNNEL LINING DESIGN

The mainline tunnel comprises a single pass precast concrete segmental lining with a nominal internal diameter of 13.1 m. The lining is 450 mm thick and uses a conventional configuration of nine interconnected segments and a smaller key segment. The

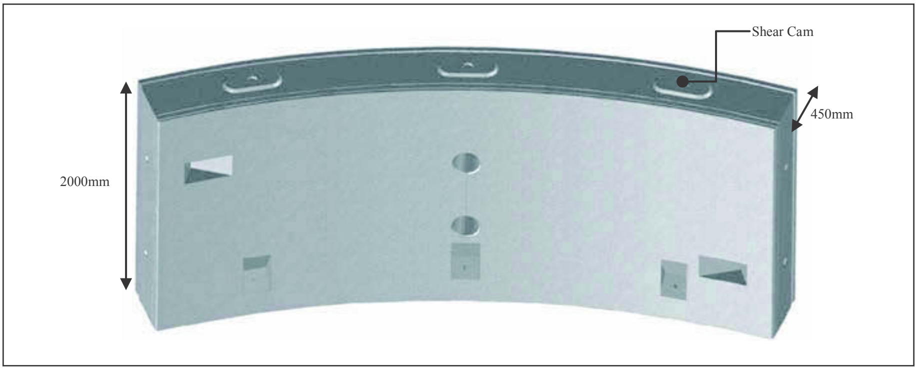

Figure 2. Typical segment

rings are either straight, left- or right-hand tapered, with the tapers specified based on the minimum horizontal radius of the curvature along the alignment. The lining will be installed inside the tail skin of the shield and grouted from the tail skin as the machine pushes forward. The rings are 2 m wide with a watertight compression gasket around each segment.

The assumed ground conditions along the tunnel alignment are expected to be variable. Around the portals the expected conditions consists of soft ground under shallow cover. Challenges encountered during the project thus far include:

- large diameter tunnel (13.1 m nominal internal diameter—using steel fibre reinforced concrete (SFRC) for the majority of tunnel lining)
- variable/mixed ground condition
- low cover with soft ground
- tunnelling under major arterial road, Great North Road (GNR)

The design of pre-cast segmental lining on the Waterview Project has adopted state-of-the-art techniques to develop a cost-effective solution. The small number of precedent linings designed of this size with steel fibre reinforcement instead of conventional reinforcement required extensive detailed modeling, review of other similar project designs and thorough peer review. Key features of the segmental lining are:

- SFRC used for most of the tunnel segmental lining
- shear cams to maintain ring shape at shallow depth
- ethylene polythene diene monomer gaskets
- shear pin connection

A typical example of segment is shown below in Figure 2.

Two reinforcement arrangements have been detailed to meet the anticipated load conditions:

- Type 1 is SFRC
- Type 2 is a conventionally reinforced concrete (CRC) segment

Type 1 segments have been designed to be used along the majority of the tunnel alignment; they contain conventional reinforcement ladders to prevent bursting from the TBM ram pressures and at longitudinal joints and steel fibre reinforced concrete.

Type 2 segments have been designed to be used in the low cover sections next to the portals and near faulted ground conditions where increased support is required. This type has a conventional reinforcement cage with plain concrete.

The segmental lining employs a conventional configuration of nine interconnected segments and a smaller key segment. Shear cams have been provided on the circumferential face of the segments to allow the rings to interlock and behave as a single structure, rather than individual rings. This design feature is of particular benefit in low cover, where confining pressures are restricted and there is greater risk of ring deformation. The spear bolt connection system will be used to secure adjacent segments and rings during the ring build. The cam helps to minimise lipping and stepping. High quality concrete and ethylene polythene diene monomer gaskets will contribute to maintaining the lining's watertightness for the entire design life, greatly reducing long-term maintenance costs.

DESIGN METHODOLOGY

The segmental lining of the mainline tunnels has been designed to achieve a minimum 100-year life with no maintenance. A full range of design and analytical methods have been used, including two-dimensional and three-dimensional finite element modelling. The combinations of effects judged to be critical of lining design were emphasised. Particular attention was given to sections of the tunnel with the reduced ground cover and sections where unfavorable ground conditions are expected. Sensitivity analysis were used to confirm the design is suitably robust and can cope with worst credible conditions.

The geometry of the segmental lining was developed to be compatible with the TBM. Specific geometric considerations included:

- providing enough space to install the key segment
- specifying optimum taper so the segmental lining can achieve the required horizontal curves without packers on the circumferential joints
- providing enough space to insert spear bolts when erecting the segments
- applying TBM thrust pressure across the maximum width of the circumferential joint bearing area
- detailing joints to consider the allowable 1% squatting of the rings
- ensuring that the contact area between segments will only occur across the designed bearing surface so that the caulking and gasket recesses will not be overloaded during maximum 'bird-mouthing'

A number of approaches were used to assess the structural adequacy of the lining to support long-term loading:

- a first pass 'closed form solution'
- two-dimensional 'bedded beam' plane frame model analyses
- three-dimensional 'plate element model' analyses

Closed form solutions were used for sensitivity analyses for differing load combinations and material parameters. The formulae are used to estimate maximum hoop load and bending moment, taking into account the resistance of the lining.

The two dimensional 'bedded beam' analyses were used to assess the structural response of the lining to various imposed load combinations to examine the influence of joint behaviour. A rigorous three-dimensional plate element model was developed to more accurately explore the coupling action of adjacent rings around the circumferential joints. At the interface between adjacent segments, rigid links and contact elements have been used to model the action of joints to pivot about the edge of the joint-bearing surface. The coupling effect of adjacent rings has also been considered by introducing a second ring that has been rotated by the roll pitch. The presence of the shear cams and circumferential dowels has also been included to assess the true interaction of adjacent rings.

The amount of bird-mouthing of the joint is related to the extent of the calculated lining deformation. The precast segmental tunnel lining design has been checked against an absolute deformation value. The combination of this rotation with high axial forces applied through the joint causes the development of an asymmetric strain profile across the joint face, with the greatest strain a the point of rotation. Segments have been designed such that they have capacity to resist these non-uniform induced bending moments when the highest axial loading is applied.

The rotation of joints and axial loading has been compared using the three-dimensional finite element models. Longitudinal joints have also been checked using empirical methods to confirm consequential tensile bursting stresses that have developed immediately behind the joint faces can be resisted.

WATERPROOFING

This requirement relates to the permanent condition, whereby the permanent lining will be designed to be effectively impermeable. For other tunnel elements such as cross passages and low point sump where initial support will be used, this initial support can be designed to be drained provided the permanent lining is undrained and a water proofing membrane is used. The allowable post-construction groundwater seepage must meet the following criteria:

- Total seepage inflow is limited to 1 litre per second.
- Tunnel must not be visibly wet.
- Permanent treatments must be used as required so that any water present on internal tunnel surfaces does not affect safety, durability and functionality.
- Groundwater seepage must not be visible or drip onto the road pavements, walkways, egress passages and plant and equipment rooms.

GROUND CONDITIONS

For the most part, the tunnels pass through sandstone of varying strength and degrees of weathering, except at the north portal where the tunnel will be partially within soil. The segmental lining has been analyzed and designed to resist the worst credible combinations of internally and externally applied loadings (see Figure 3 and Table 1).

CONSTRUCTION

An EPB TBM provides the best capability to handle the varying soil and rock conditions that will be encountered. It will also cope with groundwater inflows and limit the risk of lowering of the ground water table as required by the environmental condition of consents. The machine can inject foam and/or bentonite to facilitate spoil handling under all expected soil conditions. The EPB TBM provides fully-sealed support to the ground while advancing, which provides protection for personnel from rock falls, full control of the face support pressures and precise control of ground movements (Figure 4).

One TBM will be used, initially driving the southbound tunnel from the southern portal. The TBM will be turned at the northern portal to complete the drive south from the northbound tunnel. All construction materials will be conveyed to the southbound portal.

The lining will be installed within the tail shield of the machine using a vacuum erector that minimises installation risks to tunnel personnel. Two recesses are detailed within each segment to help locate the shear cones forming part of the vacuum pad arrangement. Primary grouting will be performed by the TBM tail skin injection, while

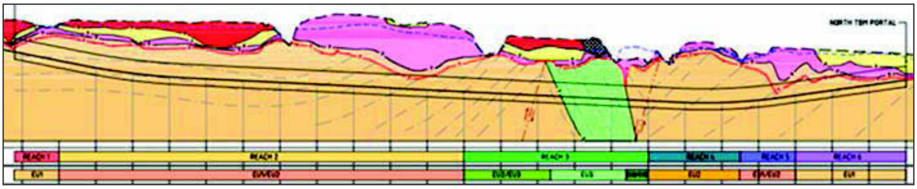

Figure 3. Predicted geology

Table 1. Predicted geology units along main line tunnels

Stratigraphic Name	Geological Unit	Symbol	Description
Man-made deposits	Fill	F	Made ground varying in composition from household refuse to clay fill
Auckland Volcanic Field	Basalt	Vb	Strong to very strong basalt, columnar jointed with a highly vesicular top and bottom, platy sub-horizontal flow parting joints and a rubble base
Tauranga Group	Undifferentiated Alluvium	A	Typically comprising clays and silts with occasional organic layers. Strength varies from firm to very stiff
Weathered East Coast Bays Formation (ECBF)	Residual Soil	ER	Firm to very stiff silts and clays
	Highly weathered to Moderately weathered ECBF	EW	Undifferentiated very stiff to hard silt and clay and dense to very dense sands
Un-weathered ECBF	ECBF—Rock Class 1	EUS_1	Extremely weak, uncemented, grain locked, fine to medium grained sandstone
	ECBF—Rock Class 2	EUS_2	Very weak, interbedded siltstone and fine to medium grained sandstone
	ECBF—Rock Class 3	EUS_3	Weak, volcaniclastic, coarse grained sand-stone (Parnell Grit)

each segment will be detailed with a grout hole, through which secondary grouting of the tunnel annulus can be done. A grouting socket will be cast into each segment but the hole will be partial depth. If secondary grouting is required, the full depth will need to be drilled out. The socket is located within one of the vacuum erector cone recesses. The grouting ferrule will comprise durable non-ferrous material.

Shallow Cover and Great North Road

Special attention has been given to formulating a minimum risk tunneling method under Great North Road (GNR) taking into account geotechnical conditions, EPB TBM capabilities and the experience that will have been acquired on the project by that time. A traffic management sequence has been developed that can maintain traffic flow on GNR without traffic having to travel directly above the TBM in operation. An intensive monitoring regime is planned to detect any unexpected increase in settlement.

The design has used NZTA's experience in operating the existing central motorway to develop a tunnel and highway alignment that features economical merge and diverge lengths within the GNR Interchange, with relatively shallow depth tunnels and safe minimum portal depths. In combination with the EPB TBM capability and lining design,a tunnel alignment design involving tunneling under GNR rather than using cut and cover has been developed, substantially mitigating disruption to local traffic and the

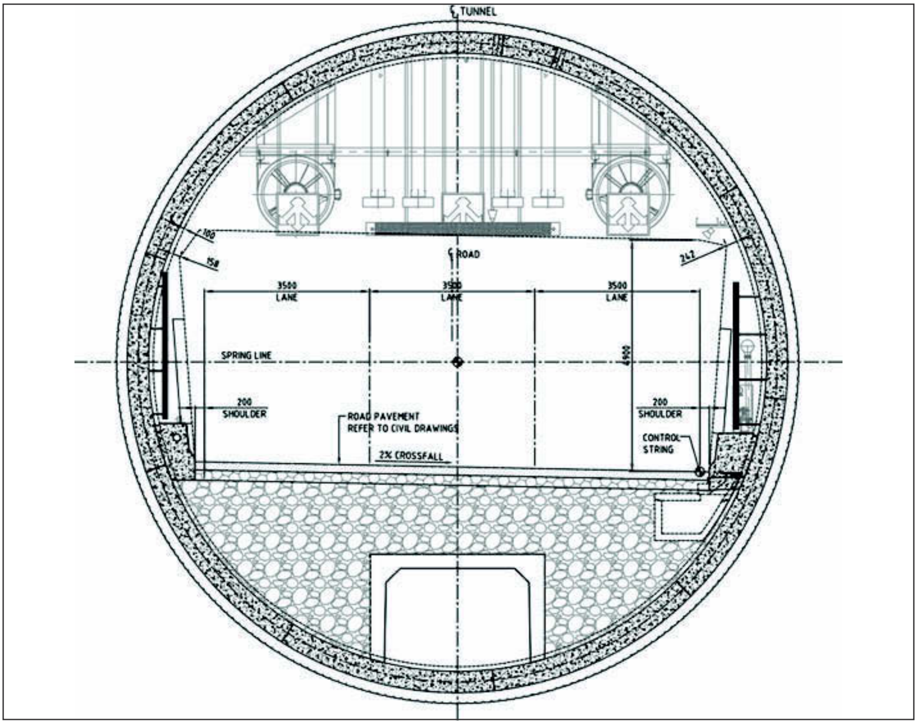

Figure 4. Typical tunnel cross section

community during construction. The northern tunnel portal is on the west side of GNR with a minimal length of cut and cover in which the northern ventilation building will be located, simplifying the exhaust inlet structure and reducing the land requirement.

During the tender it was decided the easiest, safest and most economical way to transfer the TBM from the completed first drive to the second drive was to turn the machine at the northern portal. Whilst this operation will present a number of challenges, the advantages were seen to outweigh the disadvantages, notably:

- lack of space and access for heavy lift cranes to disassemble the TBM at the northern portal
- difficulty of transporting large sections of the (disassembled) machine over secondary roads from the northern to southern portal
- time and risk involved in disassembling and re-assembling the TBM
- Obayashi's previous successful experience in turning large TBMs within tight site constraints
- avoidance of disassembly/re-assembly

Cross-Passage Construction

There are sixteen mined cross-passages that are spaced less than the permitted maximum 150 m apart to allow for slight position adjustments and economically cater for variable ring positions. The tunnel alignment was adjusted to minimise cross-passage lengths as far as reasonably practical whilst providing the necessary space for the permanent mechanical and electrical (M&E) installations. The low point sump, situated in a cross-passage in an enlarged cross-section, is close to the northern portal.

At each cross-passage, special steel frame segments will be installed in the mainline tunnels as part of the TBM lining construction to facilitate break-out and safely commence cross-passage excavation. Before cross passage excavation works begin, the ground at each cross-passage will be probed by core drilling through pre-installed ports within the steel frame segments and, if required, the ground will be grouted to strengthen the ground to improve stability and/or to reduce the water flow. Cross-passages will be constructed by the sequential excavation method (SEM). Excavation will use conventional excavation equipment with specialized attachments, depending on the ground conditions found.

The cross-passages will be constructed by separate specialist activity work parties, such as the excavation and temporary support, waterproofing, and concreting, to use different skill types efficiently. Each work party will move to the next cross-passage sequentially. For good quality control, workers will be trained by experienced superintendents or specialists in the particular aspect of work. A daily review meeting will be held to evaluate the excavation and support measures in use and to be used. This will be detailed in the required excavation and support permit to tunnel sheet, which includes observation of ground conditions of the excavation face, monitoring of supports installed in cross-passages and segments in mainline tunnels, shotcrete test results and ground evaluation and so forth. The daily review meeting will confirm the support mechanism required for the next sequence of excavations and temporary support measures.

Herrenknecht Interface

To obtain the best outcome from the design and construction process, WCA instigated close liaison procedures with Herrenknecht (HK), which included weekly video meetings as well as regular face-to-face meetings. The proposed solutions were extensively reviewed through the WCA home organizations. An outcome of these meetings was a review of the tunnel driving philosophy which has led to using a separate culvert installation gantry rather than installing the culvert as part of the TBM activity. This effectively de-links the TBM progress from the culvert and backfilling operation, thereby reducing risks to the TBM advance rates whilst providing the additional benefit of simplifying the TBM turn around at the north portal.

The TBM, while slightly larger than many, has many similar features of a normal EPB TBM, including:

- The probe drill can be mounted on the erector (systematic probing is not envisaged).
- Tool housings for interchangeable disc and ripper tools with access from the rear of the cutterhead.
- An escape refuge with a dedicated air supply for the TBM crew plus spare capacity.
- Multiple personnel and material locks to aid in hyperbaric interventions (if required).
- TBM has a shortened first gantry to help re-launch it, which will use gantry one only until the TBM has mined far enough to allow the full set of the remaining TBM and the culvert gantry to be installed as the northern portal prohibits the launch of the TBM in ‘full’ configuration.
- There are three TBM gantries and the separate culvert gantry giving an overall length of around 85 m.
- The culverts will be placed and backfilled about 150 m behind the rear of the TBM gantries, and will be operated by a separate work party. The TBM will

have priority so placing the culverts is a support activity rather than the driver of the whole sequence of the tunnelling process.

- The culvert gantry's design will allow the tunnel vehicles supplying the TBM to pass across the gantry/culvert placing operation at all times to minimize disruption to the TBM process.
- The tunnel rings and culvert will be transported underground using specialist multipurpose vehicles, which can carry a complete ring or two culvert sections.
- Samples from the portal excavations have been used for foam trials with different manufacturers' products to determine the most appropriate foam to condition the ground. This will give WCA a 'base mix' to start tunnelling with that can be modified as the drive progresses to maximise efficiency and will minimise wear.
- Spoil from the TBM will be transported using a continuous conveyer system to the south portal and then to a noise shed where it will be loaded into trucks for offsite disposal. Because of the restrictions placed on disposal and placing there will be great emphasis on the consistency (moisture content) of the spoil to minimize placing and compaction issues at the disposal site. WCA is investigating the drying (reducing the moisture content) of the spoil before removing it from site.
- The noise shed will allow storage of 48 hours of production to allow maximum drainage of the spoil and also allow 24-hour spoil removal operations.

CONCLUSION

The tunnel lining design has adopting contemporary state-of-the art methods to provide an economical segmental lining design to suit expected geological conditions and local manufacturing and construction capabilities.

Tunnelling is expected to start in mid-2014, with the Western Ring Route completed and opened by March 2017.

ACKNOWLEDGMENTS

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CRITICAL ASSESSMENT OF RMR-BASED TUNNEL DESIGN PRACTICES: A PRACTICAL ENGINEER'S APPROACH

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ABSTRACT

For professional tunnel engineers, for whom money and time are in short supply particularly in early stages of a project, this paper critically assesses what works best with the RMR system, today 40 years after its development and extensive accumulated experience. Five aspects of practical tunnel design are dealt with:

i. new design charts for rockbolt, shotcrete and steel ribs support as a function of excavation span *and* rock mass quality,

ii. deciding on tunnel shape and secondary liners,

iii. RMR ratings graphs updated for practical applications,

iv. refining prediction of *in situ* modulus of deformation based on rock mass quality alone, and

v. new approach to handling conditions of squeezing ground and rock bursting.

It is demonstrated how numerical modeling of tunnel design issues goes hand-in-hand with RMR-based estimation of rock mass properties and *in situ* quality.

INTRODUCTION

The tunnel support guidelines based on RMR were provided originally in the form of a table (Bieniawski 1989) giving support recommendations for a tunnel span/diameter of 10 meters. In view of the improving technology for rock bolting, shotcrete and steel ribs, it was left to tunnel designers to modify these guidelines for other tunnel sizes, which served its purpose well. Today, after 40 years of use, it has become apparent that it would be convenient for practical tunnel designers to have charts for the selection of rock support as a function of *both* tunnel size and rock mass quality. Accordingly, this is the main objective of this paper, together with updating the procedure for improved determination of the RMR ratings.

As shown in Table 1, the rock support measures for each rock mass quality include a combination of the various support types. Since, for example, two support methods are additive to some extent, determination of support requirements for individual types, such as rock bolts, shotcrete and steel ribs should be determined, and is dealt with in the next section.

An important question to be asked before proceeding with any recommendations is what is the current need by practical tunnel designers. It is the opinion of these authors that, with respect to modeling versus empirical assessments based on accumulated experience, using continuum models often gives unreliable results for support particularly at shallow depths, although it is useful for cases where squeezing effects are present and rock mass plasticity is extensive. For most purposes, a practical tunnel engineer needs design charts and simple aids to pragmatic design. It is not just an

Table 1. Original guidelines for support of rock tunnels based on the RMR system (Bieniawski 1989)

		Support		
Rock Mass Class	Excavation	Rock Bolts (20-mm Dia, Fully Grouted)	Shotcrete	Steel Sets
Very good rock I RMR:81–100	Full face 3-m advance	Generally, no support required except for occasional spot bolting		
Good rock II RMR:61–80	Full face 1.0–1.5-m advance Complete support 20 m from face	Locally, bolts in crown 3 m long, spaced 2.5 m, with occasional wire mesh	50 mm in crown where required	None
Fair rock III RMR: 41–60	Top heading and bench 1.5–3-m advance in top heading Commence support after each blast Complete support 10 m from face	Systematic bolts 4 m long, spaced 1.5–2 m in crown and walls with wire mesh in crown	50–100 mm in crown and 30 mm in sides	None
Poor rock IV RMR: 21–40	Top heading and bench 1.0–1.5-m advance in top heading. Install support concurrently with excavation 10 m from face	Systematic bolts 4–5 m long, spaced 1–1.5 m in crown and wall with wire mesh	100–150 mm in crown and 100 mm in sides	Light to medium ribs spaced 1.5 m where required
Very poor rock V RMR: <20	Multiple drifts 0.5–1.5-m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting	Systematic bolts 5–6 m long, spaced 1–1.5 m in crown and walls with wire mesh. Bolt invert	150–200 mm in crown, 150 mm in sides, and 50 mm on face	Medium to heavy ribs spaced 0.75 m with steel lagging and fore-poling if required. Close invert

Shape: horseshoe; width: 10 m; vertical stress: <25 MPa; construction: drilling and blasting.

issue of difficulty or complexity; numerical modeling does not currently have a good way of modeling the support effect from interlocking blocks, unless one used discontinuum modeling software such as UDEC. However in this case, one faces the problem that the range of joint parameters that comes from laboratory tests is so wide that at one end any excavation is stable and at the other nothing is stable. Experienced modelers can produce convincing results but the modeling process is arguably rather subjective.

Accordingly, the practical tunnel engineer is interested in design rules for both shotcrete and rock bolts as a function of tunnel size as well as rock mass quality, for assessing final support requirements. We believe the tunnel industry takes a pragmatic view being comfortable with an easy-to-determine RMR rating, including joint orientation as a component.

DESIGN OF SUPPORT SYSTEMS FOR UNDERGROUND EXCAVATIONS

Design Load

Rock load on a support system is a function of rock mass condition and initial stress state. Where the rock mass is not overstressed and squeezing is not occurring, the design load is taken as:

Design rock load

$$P_r = \frac{100 - RMR}{100} \cdot 10m \cdot \left(\frac{Span}{10m}\right)^{\frac{1}{2}} \cdot \rho_r \cdot \gamma_r \quad \textbf{(1)}$$

where γ_r is a partial factor and ρ_r is rock density. For γ_r =1.5 and ρ_r = 27kN/m^3 this gives the results in Figures 1 and 2.

This relationship gives a rock load increasing linearly with decreasing RMR with a maximum value equal to excavation span for a 10m tunnel. For different sizes a

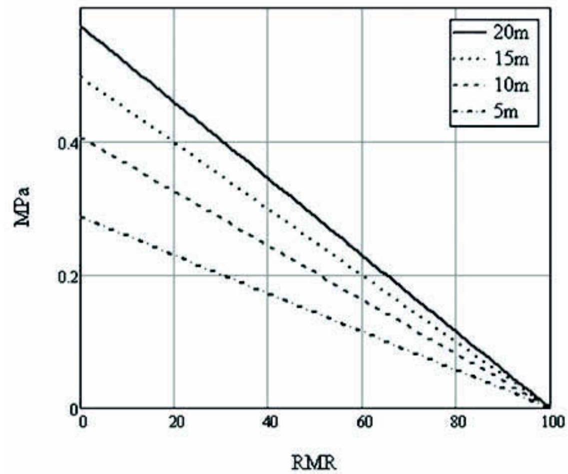

Figure 1. Rock load versus span

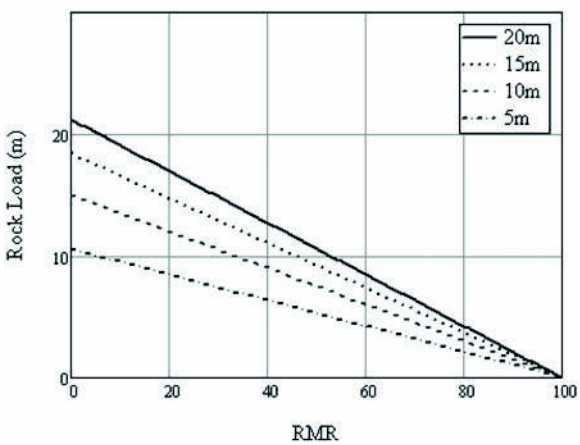

Figure 2. Height of supported rock versus span

modification factor has been applied so maximum load depends on the square root of the span. This was included to compensate for the thickness of any loosened zone round the tunnel not being proportional to span. It should be noted this load is for gravity-driven situations where there is no overstressing of the rock mass.

Stand Up Time

Depending on rock mass quality and the tunnel span, in some circumstances a tunnel may not require any support as is evident from Figure 3. This will be reflected in the charts that follow for selection of rock bolts, shotcrete and steel ribs.

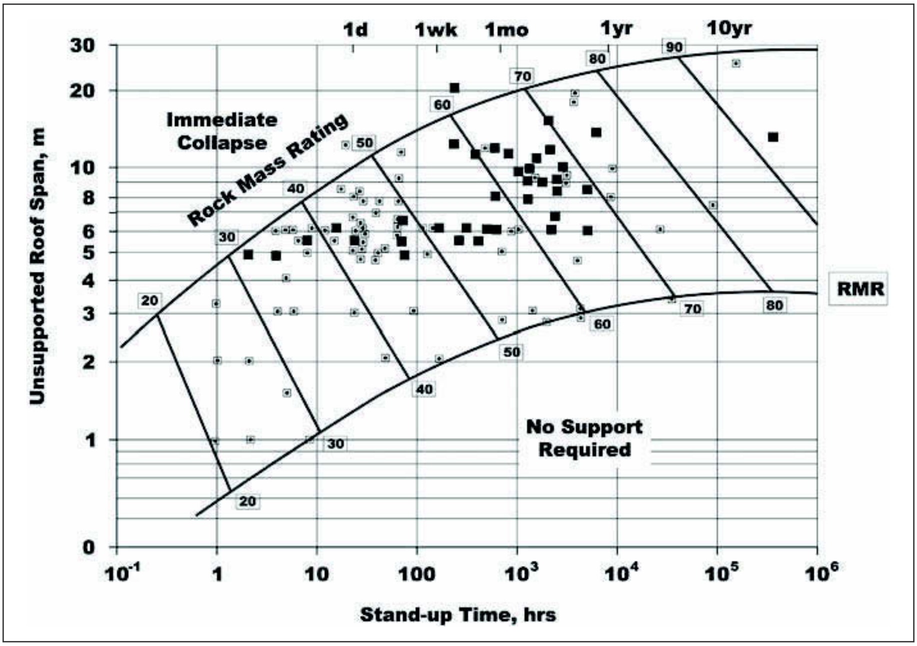

Figure 3. Stand up time of an unsupported tunnel span, as a function of rock mass quality RMR

Bolt Spacing

Bolt spacing is taken as a function of RMR only. Spacing has to reflect fracture frequency and the need for shotcrete to provide adequate support between the bolts at the better rock end of the scale where the shotcrete cannot realistically be considered as working on its own as an arch. Spot bolting only is assumed to be needed above RMR = 85 (see Figure 4).

Rockbolt spacing

$$S_b = 0.5m + 2.5m \cdot \frac{RMR - 20}{65} \quad \text{if } 20 < RMR \le 85 \tag{2a}$$

$$S_b = 0.25m + \frac{(RMR - 10)^{1.5}}{140} m \quad \text{if } 10 < RMR \le 20 \tag{2b}$$

$$S_b = 0.25m \quad \text{if } RMR \le 10 \tag{2c}$$

Bolt Length

Bolt length must vary with span and RMR. Based on empirical guidelines used in mining and the results of numerical modeling studies, the following relationship was obtained:

$$Span = \frac{(L_b + 2.5)^{\frac{RMR+25}{52}}}{3.6} \tag{3}$$

where Span is width of excavation in meters and L_b is embedded bolt length in meters (see Figure 5).

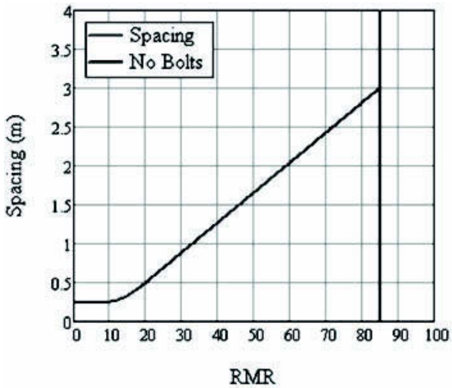

Figure 4. Rockbolt spacing versus RMR

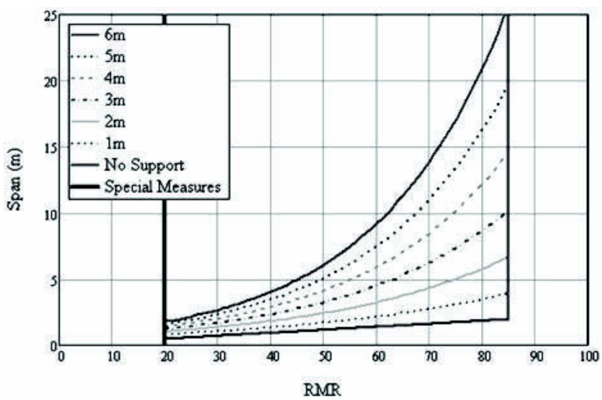

Figure 5. Rockbolt length chart

Rockbolt Capacity

The support capacity of pattern rock bolting is assumed to be given by the capacity of each bolt divided by the area it has to support. Capacity of bolts reduces as RMR reduces because of both the difficulty of obtaining an effective bond and the reducing effectiveness of the bolts in mobilizing arching action in the rock mass. The relationship developed for this is shown below. The chart shows capacity for a typical 25mm bolt with an ultimate strength of 25 tons.

Bolt capacity

$$F_{bd} = \frac{F_b}{\gamma_b} \cdot \left(\frac{RMR}{85}\right)^{\frac{40}{RMR}} \qquad \textbf{(4)}$$

where F_b is ultimate tensile capacity of bolt and γ_b is a partial factor (see Figure 6).

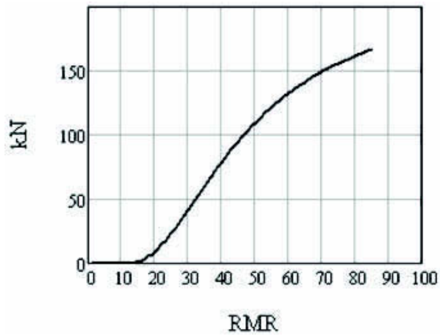

Figure 6. Design rockbolt capacity versus RMR for: F_b = 250kN, γ_b = 1.5

Shotcrete Capacity

The design capacity of shotcrete support is based on the concept of the shotcrete acting simply as an arch in compression. The basic formula for this type of support is:

Support pressure = thickness × design strength/radius

However, this has been modified to reflect the reality of both the real action of shotcrete and the construction process. Some different support situations are considered below.

Where rock is of good quality, RMR > 60, only a thin layer is required. Applying a thin layer of shotcrete to an irregular surface in blocky rock results in a layer of shotcrete that is itself irregular, and assuming it behaves as a stand-alone arch is not realistic. In reality its main function will be to lock adjacent blocks together to prevent dropouts and help the loosed rock in the blast damaged zone to support itself and any rock load outside this zone.

For medium quality rock, say RMR = 35 to 60, the shotcrete can more reasonably be thought of as acting as an arch. If large parts of the tunnel are in rock of this quality the tunnel shape will probably be a horseshoe with curved sidewalls, and certainly at the lower end of this scale the shotcrete will have filled a lot of the irregularities in the surface and the inside shape will be curved with few re-entrant areas.

Below RMR values around 35 the excavation is going to require multiple headings unless the tunnel is very small. This is done to keep the size of the face and therefore the size of any heading collapse to manageable proportions. Analysis of the support system by numerical methods or otherwise finds that the shotcrete is increasingly subjected to flexural and shear forces as well as simple axial compression.

Moment/Axial load charts (MN Curves) made to suit national design codes are widely used in the design of liner systems worldwide. If the axial load values on the Y-Axis are divided by member thickness the result is a Moment/Axial Stress curve. The curve shown below is for 32MPa cylinder strength shotcrete with a thickness of 300mm (see Figure 7).

A notable feature of this curve is the maximum design moment capacity occurring when mean axial stress is around ⅕ × cylinder strength. This is so no matter what shotcrete thickness (or design standard, provided it is follows limit-state principles) is used and this stress level is often used as a target value in the design of liner systems in soft ground. If thickness changes, the X-Axis scale will change but the Y-Axis scale and the shape of the curve remain the same.

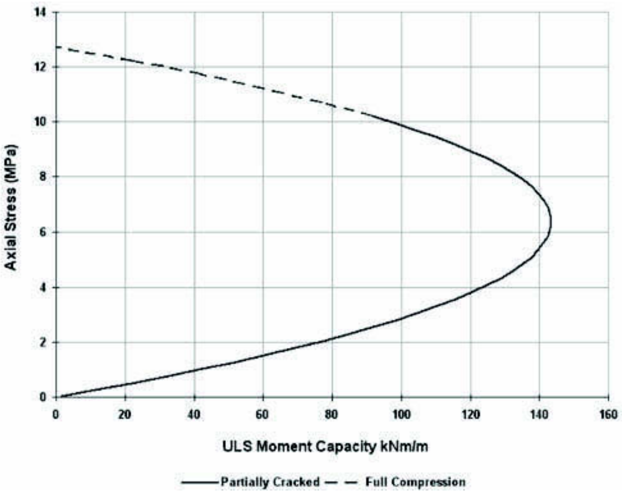

Figure 7. Interaction curve for 32MPa shotcrete to *Eurocode* 2

This property has been used to develop a relationship for shotcrete design capacity, which reflects the real behavior of shotcrete round an excavation and the demands that are placed on it. The chart below shows design strength for 30MPa shotcrete.

Design capacity

$$f_{cd} = \frac{f_{ck}}{\gamma_s} \cdot \left[0.2 + 0.8 \cdot \left(\frac{RMR}{100}\right)^{\frac{3}{2}}\right] \quad (5)$$

where f_{ck} is shotcrete cylinder strength and γ_s is a partial factor (see Figure 8).

Shotcrete Support Chart

Based on the above relationships for the capacities of shotcrete and rock bolts a chart relating Span and RMR for a given thickness of shotcrete can be derived. The capacities of the rock bolts and the shotcrete are considered to be additive.

The irregularities in the lines around RMR = 50 to 20 reflect rock bolts ceasing to have significant support capacity. The middle option in the expression for rockbolt spacing in Equation 2b, for RMR between 10 and 20, is used as a transition to give a reasonably smooth curve in Figure 9.

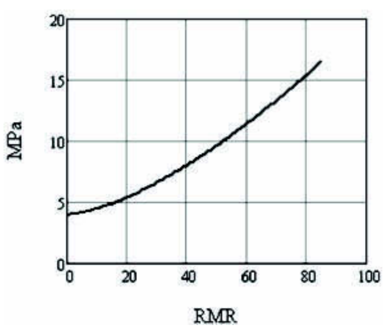

Figure 8. Design shotcrete strength for: f_{ck} = 30MPa; γ_s = 1.5

Non-Circular Profiles

The charts shown in Figures 9 and 10 are for cases where the shape of the tunnel above axis level is of a constant radius, so that Span = 2 × radius. In some situations, usually in good rock,

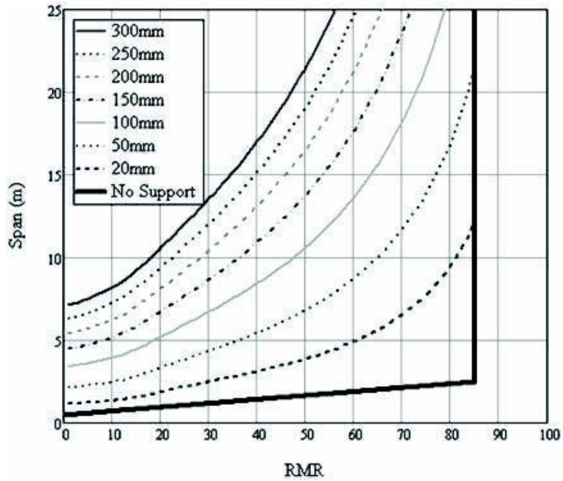

Figure 9. Shotcrete design chart assuming no ribs

flatter profiles are used with a relatively flat arch in the crown and smaller radius haunches. This is common in transportation tunnels and some large caverns. For these shapes the charts cannot be used directly. Support for these situations can be calculated basing rock load on Span and RMR, but then calculating the contribution of the shotcrete by dividing its design strength by the larger radius used in the crown. Alternatively the charts can be used but the shotcrete thickness must be factored up by crown-radius/half-span.

Steel Ribs

Steel ribs are used less than they once were because of advances in shotcreting technology including the development of wet-mix shotcrete, remotely operated shotcrete robots, and both steel and plastic fibers. These advances have made shotcreting both safer and less time consuming with the added advantage that primary support of good quality shotcrete can satisfy design-life requirements for permanent support that traditional ribs and lagging cannot.

These are still situations where steel ribs are the support system of choice. These situations include very wet excavations, where shotcreting can be ineffective, TBM situations where shotcreting can be disruptive and squeezing ground situations where ductility is particularly important and sliding joints may be necessary to accommodate high levels of convergence.

The design of steel ribs is carried out assuming the ribs are blocked at discrete intervals. Failure can then occur under the resulting combination of axial compression and flexure. Equations for the capacity of blocked ribs (Lowson 2012) are as follows:

Elastic limit

$$P_{el} = \frac{4 \cdot A_s \cdot I_s \cdot \sigma_y}{S_r \cdot r_i [4 \cdot I_s + A_s \cdot X \cdot r_i (1 - \cos(\theta))]} \quad \textbf{(6)}$$

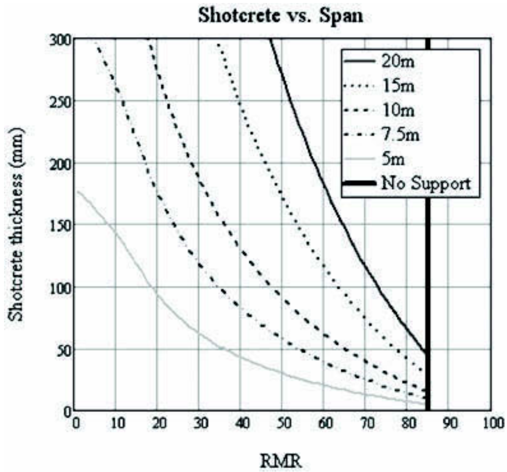

Figure 10. Shotcrete thickness for different tunnel spans

Plastic limit

$$P_{pl} = \frac{2 \cdot S_{pl} \cdot A_s \cdot \sigma_y}{S_r \cdot r_i [2 \cdot S_{pl} + A_s \cdot r_i(1 - \cos(\theta))]} \tag{7}$$

The terms A_s, I_s, S_{pl} and X are section area, second moment of area, plastic section modulus, and section depth respectively, and S_r is rib spacing. If the blocking angle q is small, typical of ribs with any gap behind filled with shotcrete, both formulas yield about the same result as an ultimate capacity, which is simply:

$P = A_s\sigma_y/r$ – the plastic capacity of a circular ring.

The effect of the blocking angle is to reduce capacity as shown below for a typical steel rib arrangement consisting of 152 × 152 × 37kg/m UC's with 250MPa yield strength at 1m rib centers in a 10m tunnel.

Increasing the blocking angle from zero to 15 degrees reduces capacity by about 60%. This reduction is independent of yield stress and is higher for lighter ribs and for TH sections and smaller for heavier sections (Figure 11).

Shotcrete support will still be needed to support the ground between the ribs. In most situations the shotcrete can be designed on the basis of carrying a nominal rock load equal to the rib spacing, with the remaining load being supported directly on the ribs, spanning between them as a series of jack-arches. Design of the shotcrete can then be done by considering the shotcrete carrying the load as a simply supported beam spanning between the ribs. The required thickness of shotcrete is then:

$$t = \sqrt{\frac{3 \cdot S_r^3 \cdot \rho_r \cdot \gamma_r \cdot \gamma_f}{4 \cdot f_{flex}}} \tag{8}$$

where S_r is rib spacing, ρ_r is rock density, γ_r and γ_f are partial factors on loading and shotcrete flexural strength, and f_{flex} is the flexural tensile strength of the shotcrete.

For typical flexural strengths of 5MPa peak and a residual strength factor of ⅓, giving a residual flexural strength of 1.65MPa the required thicknesses for typical rib spacings are shown below. Flexure is critical over the full range shown, shear only becoming an issue at shorter rib spacings (Figure 12).

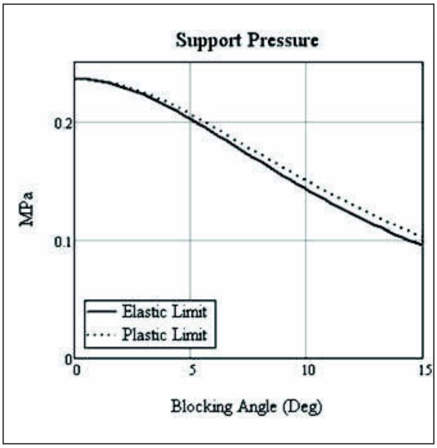

Figure 11. Rib support vs. blocking angle

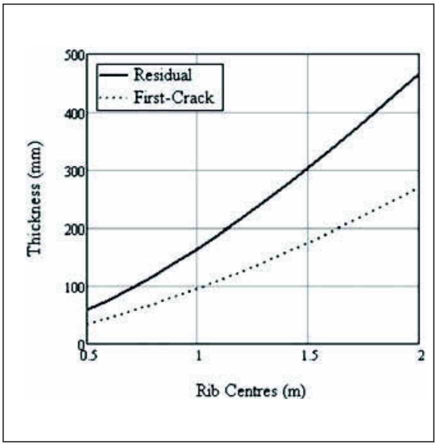

Figure 12. Shotcrete support between ribs

Where it is possible to monitor the support and carry out remedial work if necessary the shotcrete thickness can be determined based on the thinner, uncracked value (dotted line). If it cracks more shotcrete can be added. If repair is not possible an assumption of cracked behavior can be used to allow for drying, thermal, or other effects that can cause cracking with or without a load.

In very poor ground with little or no shear strength the ground will not span effectively between ribs and a shotcrete and rib support system will need to be considered as acting compositely as an arch.

DECIDING ON TUNNEL SHAPE AND SECONDARY LINERS

The shape and size of a tunnel cross section are arguably the most important things a designer has to decide on. The size of the finished internal cross section will generally be driven by functional requirements, for example transportation tunnels will usually have to fit round some roughly rectangular shape to accommodate vehicles, while hydropower and water supply tunnels will need to have a minimum cross sectional area for water flow. Excavation shape has implications for construction as with the exception of TBM drives the invert of the tunnel will ideally be flat to accommodate construction traffic. Excavation shape also has important implications for the design of both primary and secondary liners as it is the major determinant of how they will have to function as structures.

In principle the most structurally efficient shape for a tunnel is circular, because it means the liner can resist external loads from the ground or from groundwater by acting as a cylinder in compression. Whether the liner is made of steel or concrete a circular shape will minimize material costs and in the case of concrete the absence of flexural effects means reinforcement is often not necessary. However circular finished profiles are awkward to construct compared with D-shapes with flat bottoms both because of the special formwork needed for their construction and because a flat invert forms a better roadway for construction traffic.

The optimum shape for a tunnel is a complex function of the realities of construction, the loads that will be applied to the primary and secondary support, and the needs of the finished tunnel. Of these the difficulty and cost of the excavation process and the primary and secondary support systems are functions of the ground and its state of stress, and often also the groundwater regime. For construction purposes and the cost effectiveness of primary support systems ideal tunnel shapes are as shown in Table 2.

Table 2. Ideal tunnel shape versus ground condition

Ground	Shape	Comments
RMR > 50	D-shape with vertical or inclined sides and flat invert.	Easiest to construct
RMR 30 to 50	Horseshoe with curved sidewalls	Reduces sidewall support costs
RMR 20 to 30	Horseshoe with curved sidewalls and curved invert	A curved shotcreted invert can be more economic that bolting the invert and/or an RC structural invert.
RMR 10 to 20	Shape made up of 3 or more curves	Usually 3-curve comprising arch, haunch and invert radii, or 5-curve with arch, shoulder, sidewall, haunch, and invert radii
RMR < 10	Circular	

Secondary Liners

Secondary liners, if required, will often have to be designed to carry structural loads. These may include rock loads but often the loading that is the biggest determinant of secondary liner design is water pressure. Where a lined underground structure is built below the existing groundwater table (phreatic surface to some) the structure has to either be designed as a permanent drain or the liner system has to be designed to be watertight and carry external water pressure.

Permanent drainage of the ground above may in some circumstances, such as hydropower headrace tunnels in fair rock or better, be acceptable and can be done by providing weep-holes or a piped drainage system. However, where the rock is poor and erosion and loosening could occur over the long term this cannot be done. Otherwise environmental considerations, high permanent pumping costs, or functional requirements may mean liners have to be watertight.

The easiest shape to design for external water pressure is a circular shape, because it can be designed as a compression structure and only the minimum thickness has to be decided, which is easy to do using standard structural codes.

The next most efficient shape is one that is continuously curved. With an appropriate choice of thicknesses and radii it can be designed as a compression structure, with little or no tension on either face. Where tension does arise in analysis and reinforcement is necessary it is often possible to reduce the amount needed to satisfy structural design code requirements by using a non-linear model for reinforced concrete. This is because the reason for using reinforcement is that the concrete is going to crack in tension, and where it is cracking due to flexure it becomes much more flexible once it has cracked. The stress/strain behavior of reinforced concrete can be thought of as elastic—brittle, strain-hardening—plastic, and the secant modulus of cracked reinforced concrete can often be shown to be half or less of its uncracked value, even allowing for long term creep. Some numerical modeling packages now support a non-linear constitutive model for reinforced concrete and doubtless more will in future. Cost effective designs can be made using such models and it will often be found that an acceptable shape can be developed that uses little more than code-minimum reinforcement to distribute and control cracking.

Where the tunnel itself is a straight-sided horseshoe a horseshoe shaped secondary liner may be the most economic solution as it minimizes concrete requirements. However, these need to be proportioned to be designable and these proportions have to be got right in the planning stage. This is particularly an issue with hydropower headrace tunnels where feasibility stage designs often simply show a D-shaped liner throughout, with reinforced concrete used in zones with poorer rock. Where structural liners have sharp corners the liner proportions should always be checked for shear capacity, as this will often govern the design. Shear should be checked an effective depth away from any corner or the end of any flat fillet in a corner, and good target mean shear stresses using factored ULS loads are 0.5MPa for a design with no shear reinforcement and 1.2MPa to 1.5MPa for an easily installable pattern of links.

Designs should consider the possibility of rock load being applied to the liner and for planning and proportioning purposes the liner can be designed to carry a uniform load equal to the rock load from Equation 1 unless the rock mass is overstressed because of poor ground or high cover or both.

On many projects the ground will vary along the tunnel and it is usually not economic or practical to change the shape of the excavation to the ideal shape for a given rock class. This would mean changing to the shape one would choose if the whole tunnel was in a particular class of rock. The shape (or shapes) chosen, particularly for hydropower tunnels where simple D shapes are usually preferred, is often a compromise to provide an economic solution. The nature of that compromise, in terms of cost and also in terms of structural design, needs to be understood.

DETERMINING RMR PARAMETER RATINGS FROM CHARTS

Certain misconceptions are evident in the literature (Bieniawski 2011) concerning determination of the RMR parameter ratings. Traditionally, these were determined as shown in Table 1. However, some users were not aware that the ratings in this table were the *average* values for the ranges shown, and not the *minimum* values. For improved accuracy, it is better to use the recommended graphs, showing the continuous values of the ratings, as depicted in Figure 13. An important aspect to note here is that, the minimum values of all parameters are zero, so that, at worst, the RMR may be zero and not as RMR=8, as some users concluded from Table 3.

Use of Parameter RQD Is *Not* Recommended

This parameter was included originally among the six RMR parameters because the case histories collected in 1972 all involved RQD. Over the years it became apparent that RQD was difficult to determine at tunnel face, being directed to borehole characterization, and it was subsequently combined with parameter "discontinuity spacing" ("joint" spacing)—and named "spacing density" since the two are interrelated. For the best practical use, this led to the preferred use of "fracture frequency" as an invert of "fracture density"—as depicted in Figure 14. Neither of these approaches changed the basic allocation of rating values to these parameters.

ESTIMATION OF ROCK MASS DEFORMABILITY FOR ANALYTICAL MODELING

Following the updated RMR determination procedure discussed earlier and analytical modeling performed in this paper, based on RMR expressions and data, the authors would like to comment on a practical aspect of determining the modulus of rock mass deformability, necessary for numerical analyses.

First, one should note that there is a great difference between "determining" and "estimating" rock mass deformability: *determining* is highly desirable, *estimating* is done in the absence of reliable in situ data for preliminary designs.

The *in situ modulus of deformation* is needed for tunnel design to determine deformations and displacements in a tunnel under the load of the overburden and induced stresses. This type of input data is best obtained by such in situ tests as plate bearing tests or large flat jacks, but these are very expensive and time consuming, and accordingly seldom used nowadays. Thus, the RMR rock mass classification system was

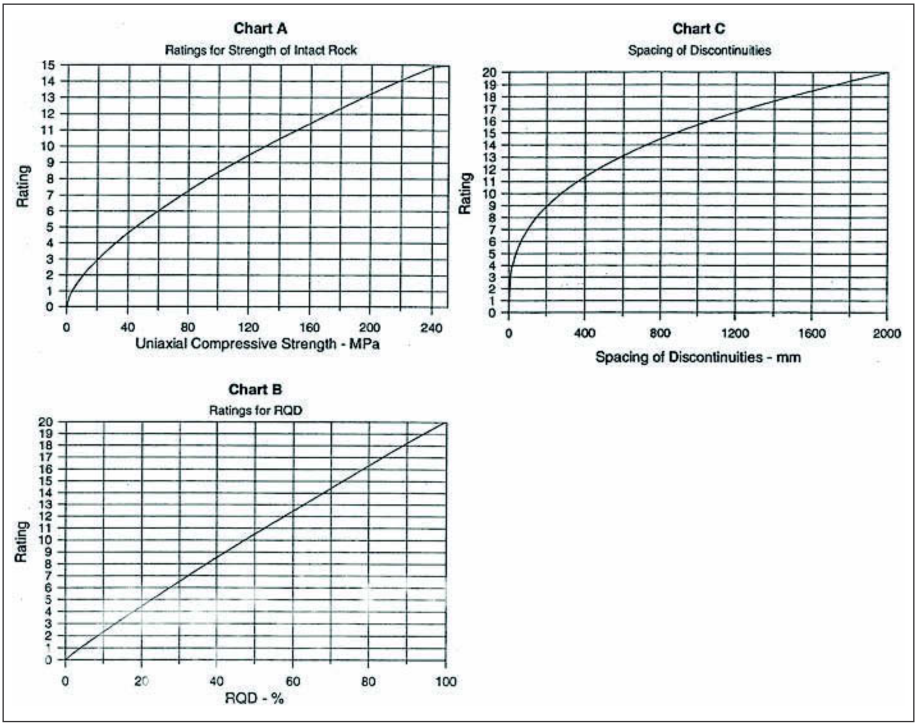

Figure 13. Rating charts for RMR parameters

the first to be used for this purpose (Bieniawski 1978), proposing a direct correlation between rock mass quality and the *field* modulus of deformation E_M, as depicted in Figure 15. This is preferable to using a ratio of the laboratory-obtained modulus of elasticity to the field modulus, because the latter only adds another variable and uncertainty inherent in laboratory testing procedures. In practice, reliable intact rock modulus data are seldom available, and in some countries laboratories adhering to ISRM recommendations are difficult to find. On the other hand, the RMR–modulus direct correlation was based on numerous in situ large-scale tests, carefully monitored and analyzed, and the data obtained formed the bases of further studies.

Subsequently, Serafim and Pereira 1983 extended the validity of the original relationship to lower quality rock masses.

Today, it is unfortunate that we hear an argument that *qualitative* estimates are preferable since they are easier and cheaper to use, as it is clearly a step backwards adding more empiricism to an already empirical approach. Nevertheless some designers and planners accept such reasoning.

The authors recommend an approach first proposed by Palmström and Singh 2001, *with different relationships for two ranges of RMR*, depicted in Figure 15a, as the best fit to experimental data and represent a realistic practical approach, instead of relying on endless correlations appearing in the literature.

In this investigation, the authors propose an improved relationship for the range of RMR greater than 56. This reflects the idea that, at high RMR, deformations will be dominated by intact modulus, whereas at lower RMR weathering and joint infilling will largely control deformation. This approach has the advantage that modulus values are *NOT* overestimated at the higher range nor underestimated or overestimated at the

Table 3. Rock mass rating system (after Bieniawski 1989)

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
	Parameter		Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90%-100%	75%-90%	50%-75%	25%-50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6-2 . m	200-600 mm	60-200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10-25	25-125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1,-0.2	0.2-0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

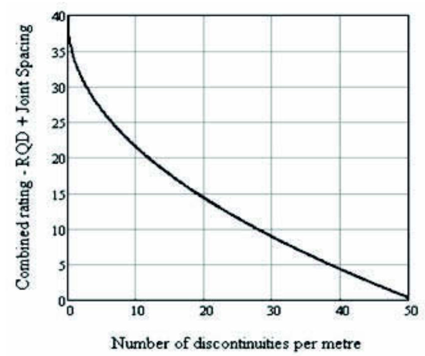

Figure 14. Chart D for combined rating of the discontinuity density parameters: RQD plus discontinuity spacing

lower range. This is more realistic than relying on one sigmoidal equation. The relationship for the range RMR below 56, remains as the one developed by Serafim and Pereira 1983:

$$E_m = 10^{\frac{RMR-10}{40}} \qquad \text{for RMR} \leq 56 \tag{9}$$

The new relationship developed by the authors for the higher range is:

$$E_m = 14 + (E_i - 14) \cdot \left[1 - \left(\frac{100 - RMR}{44}\right)^{\frac{RMR}{70}}\right] \qquad \text{for RMR} > 56 \tag{10}$$

Equation 10 is presented graphically in Figure 15b.

A note of caution: A number of sigmoidal equations have been proposed that give rock mass modulus as a function of intact modulus and a rock mass rating. These equations may give a good estimate of modulus given the correct input data, however

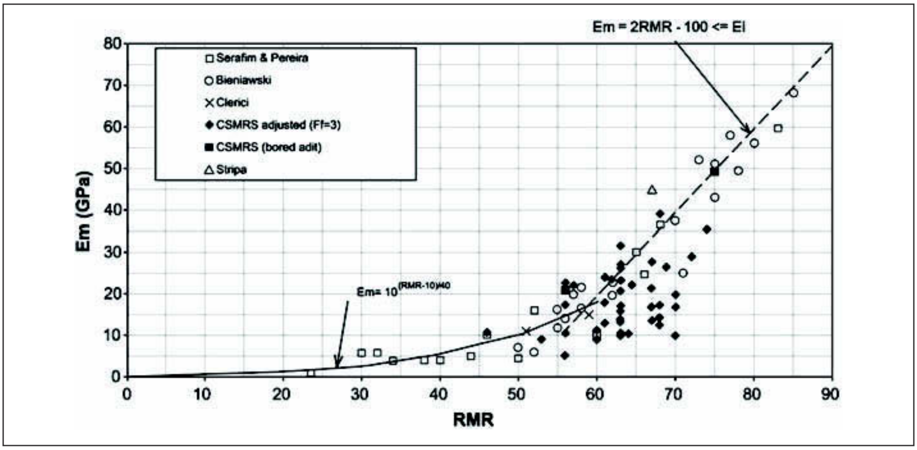

Figure 15a. Rock-mass modulus Em vs. RMR (Palmström and Singh 2001)

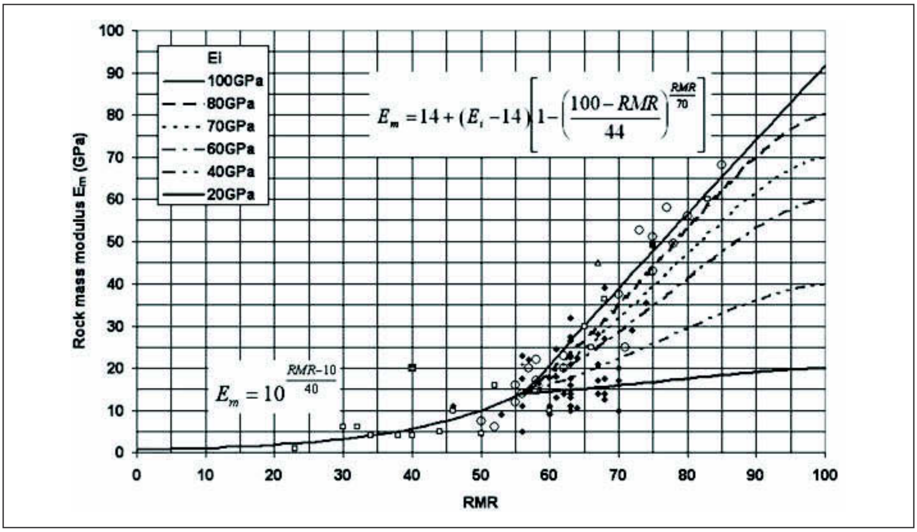

Figure 15b. Graphical representation of the Lowson–Bieniawski equation for estimating rock mass modulus of deformation, at RMR >56. Experimental data are defined in Figure 15a.

it is difficult to obtain reliable intact strength or intact modulus values from laboratory tests on samples from highly disturbed rock masses. Because of this limitation something that is commonly done in practice is to base intact modulus values on test results done on good samples of intact rock from locations with competent rock, using either laboratory measurements of intact modulus or on an assumed ratio between intact strength and modulus for a particular rock type. This ignores the possibility that the material in zones with poor rock will often be highly weathered, and it ignores the possibility that even without weathering a zone of poor rock may represent rock which simply has a lower intact strength, and that is why it became disturbed while zones of stronger rock on the same project did not.

Squeezing Ground

Squeezing is a slow convergence of the completed tunnel due to high in situ stresses relative to rock mass strength. It can be very difficult to control and can require very high support pressures possibly combined with high levels of convergence for the rock mass to achieve equilibrium.

The degree of plasticity that will occur around an excavation can be estimated using the *ICE Index of Elastic Behavior* (Bieniawski and Celada 2011) and this can be used as a guide to whether squeezing will be an issue. The ICE is defined as:

$$ICE = 100 \cdot \frac{\sigma_{cm}}{\sigma_{t.\max}} \cdot F_s \tag{11}$$

where:

Global strength

$$\sigma_{cm} = \sigma_{ci} . e^{\frac{RMR-100}{24}} \tag{12}$$

Vertical stress

$$\sigma_0 = H_r . \rho_r \tag{13}$$

Maximum tangential stress

$$\sigma_{t.\max} = \sigma_0 \cdot (3 \cdot k - 1) \qquad \text{if } k > 1 \tag{14}$$

or

$$\sigma_{t.\max} = \sigma_0 \cdot (3 - k) \qquad \text{if } k \leq 1 \tag{15}$$

Here σ_{ci} is intact rock strength, H_r is cover to surface, ρ_r is rock density, and k is ratio of horizontal to vertical total stress at tunnel elevation.

The term F_s is a shape factor used to account approximately for different excavation shapes. Values derived from numerical modeling studies are

6m circular	$F_s = 1.3$	14m horseshoe	$F_s = 0.75$
10m circular	$F_s = 1.0$	25m × 60m (W × H) cavern	$F_s = 0.55$

The term σ_{cm} used here is global rock mass strength. This is a higher value than the unconfined strength of the rock mass and originated as a simple strength parameter that can be used for design of support pillars in mining, which are laterally restrained, to an extent depending on their shape, by the rock mass above and below. Rock on the intrados of a tunnel also has some degree of confinement resulting from the shape of the tunnel so the global strength value is relevant to stability.

Based on case histories, numerical modeling, and studies of axisymmetric rock mass behavior using algebraic models, a guide to the conditions where squeezing starts to be significant is:

ICE = 25

This corresponds to a lower limit on rock mass-strength to overburden-pressure ratio of 0.5 if the stress field is isotropic.

The relationships used for deriving the ICE Index can be rearranged to give maximum cover to avoid squeezing resulting in the following:

$$H_{\lim} = \frac{\frac{100}{25} \cdot \sigma_{ci} \cdot e^{\frac{RMR - 100}{24}}}{\gamma_r \cdot (3 \cdot k - 1)} \cdot F_s \quad \text{if } k > 1 \tag{16a}$$

or

$$H_{\lim} = \frac{\frac{100}{25} \cdot \sigma_{ci} \cdot e^{\frac{RMR - 100}{24}}}{\gamma_r \cdot (3 - k)} \cdot F_s \quad \text{if } k \le 1 \tag{16b}$$

Another stress-related issue is rock bursting where otherwise competent rock suffers from spalling failure because of its stress state. Hoek and Brown (1980) and Palmström (1995) both present guidelines for intact-strength to tangential-stress ratio suggesting severe spalling effects might start around a ratio of 2. This occurs on the ICE = 25 boundaries at an RMR of 50. Heavy rock bursting will occur at an intact-strength/stress ratio of 1, which the ICE lines reach at RMR = 67.

Figures 16 and 17 show the cover limits for ICE = 25 for different intact strengths and k-ratios. The plots become horizontal at RMR = 67 where severe rock bursting would occur even in massive rock.

The support charts in Figures 9 and 10 are based on shotcrete having to resist flexural and shear loads during the construction of a tunnel using multiple headings and so the design strength has been reduced. In very poor rock conditions and, particularly if squeezing behavior is encountered, the invert will have to be closed close behind the heading and the shape will need to be if not circular then at least a reasonably structurally efficient one with a curved shape throughout. Once the invert is closed and the lining can work as a compression structure the reduction in design strength is no longer needed to the same extent. For a circular opening a support system design based on the charts above will have a support capacity equal to around 3 diameters of rock load, so the charts have a built-in allowance for a degree of squeeze. Below ICE = 25 (above the lines on the charts) the support pressure required for stability will start to exceed the design capacity of the support shown in Figures 9 and 10.

It can be seen from Figures 16 and 17 that squeezing can occur at quite modest cover levels if RMR or intact strength is low, or if there is a high horizontal stress ratio. In many cases the zones of very poor ground encountered will be in short stretches at faults or localized shear zones where the poor ground is supported by more competent ground each side of it. However where squeezing can be predicted it should be allowed for in the design. Charts 15 and 16, or similar charts easily made using the ICE Index as defined above can be used as a simple method for deciding if stress-related problems can occur. Where squeezing is predicted numerical modeling studies should be carried out to investigate its severity and determine appropriate support. Specialist guidelines are also available for dealing with rockburst problems. Because of the complex interaction of excavation shape, in situ stress ratio, and rock mass failure characteristics simple charts cannot be used for support design where stress-related problems are expected, except perhaps, with care, for initial cost estimating for a project.

CONCLUSIONS

The objectives of this paper were to provide easy to use design charts for preliminary support design based on RMR, and to cover some of the other issues, excavation shape and squeezing ground in particular, that have to be considered in underground excavation design. The intention was to provide this guidance in a readily accessible form including where possible the mathematics used to generate the charts as well as the underlying logic.

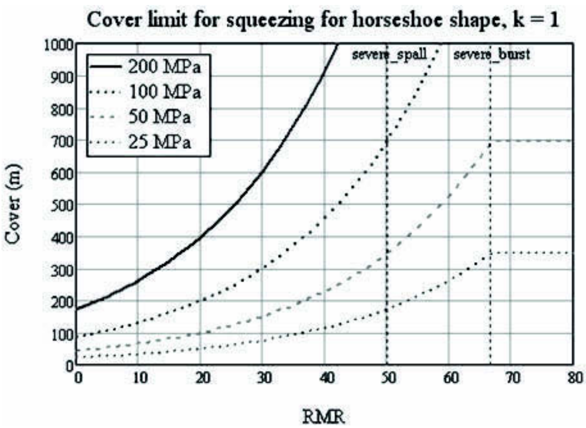

Figure 16. Cover limit for different intact strengths, k = 1

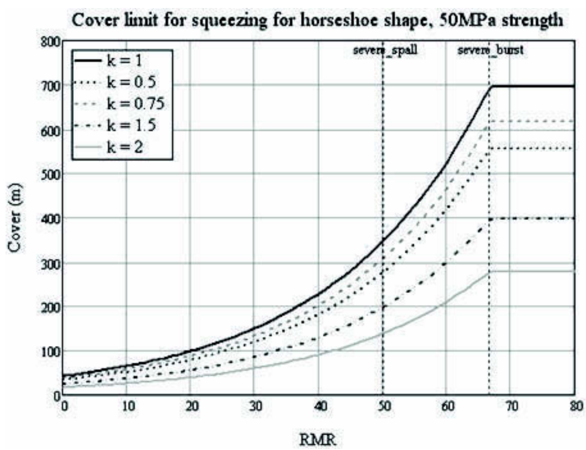

Figure 17. Cover limit for different k ratios, 50MPa intact rock strength

The statement that these charts are for preliminary design must be emphasized. The same is true of all design charts and simplified methods for underground design. They are essentially expert systems, developed to give similar decisions on design to those an expert would make by following a set of rules. It is also true however that while an expert might arrive at what he considered to be a reasonable set of support designs based on information available before construction started, he might well decide it was appropriate to modify those designs during construction. This might mean more support or it might mean less, or it might mean favoring one type of support over another to suit the available equipment or the skills of the workforce. Ground conditions are inherently variable and the stress state of a rock mass can vary dramatically from "average" values assumed in design. There should therefore always be provision in any contract to allow some revision of preliminary support arrangements to suit actual conditions. It is often impossible to foresee every condition that will arise underground as the unexpected can always occur, but it is hoped that the content of this paper will help.

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PART 4

Difficult Ground

Chairs

Evan Wheeler
Frontier-Kemper Constructors, Inc.

Brad Cavitt
Kiewit Infrastructure Co.

CONSTRUCTION OF THE SANTA ANA RIVER INTERCEPTOR (SARI) RELOCATION: MICROTUNNELING THROUGH ABRASIVE SOIL WITH HARD COBBLES AND BOULDERS

Dave Young ▪ Hatch Mott MacDonald

Rory P.A. Ball ▪ Hatch Mott MacDonald

Lance Natsuhara ▪ Orange County Flood Control District

David Philips ▪ Orange County Sanitation District

Les Wong ▪ Tetra Tech

ABSTRACT

The Santa Ana River Interceptor (SARI) Relocation Project in Yorba Linda, CA will relocate four miles of existing interceptor sewer pipeline out of the Santa Ana River scour zone. Approximately 1,433 m (4,700 ft) of the product pipe will be installed using 1,956- or 2,578-mm (77- or 101.5-in.) outside diameter microtunneling in 5 segments, including 2 siphon crossings and 2 curved tunnels, one of which is a 477 m (1,567 ft) S-shaped alignment. Tunneling was chosen to avoid construction impacts to the community and environmental impacts along the river. One of the tunnels may cross the Whittier-Elsinore Fault Zone. Soil conditions on the project include a complex mix of alluvium with abundant cobbles and boulders in a weak sandy matrix. During design, subsurface investigation reports indicated abrasive soils based on Miller testing. Groundwater levels are well above the pipeline elevation during flood season along several of the tunnels. Temporary shaft structures are excavated up to 21 m (70 ft) deep and include the use of cement deep-soil mixing, secant piles, and soldier pile and lagging walls with permeation grouting designed to limit groundwater inflow. This paper discusses shaft and tunnel construction techniques as well as the machine design that included the capability of compressed air face access. Lessons learned are presented and include how challenges were successfully overcome. Microtunnel production cycles are presented and compared. Backfill grouting of plastic carrier pipes within the microtunnel casing is described, including how the heat of hydration was minimized to prevent damage to the carrier pipes.

INTRODUCTION

Construction of the Santa Ana River Interceptor (SARI) Relocation Project began in mid 2011 with the start of the Yorba Linda Spur (YLS) contract and is expected to be complete by 2014. The project consists of the following two contract packages:

1. SARI YLS Contract—Picks up domestic wastewater from the City of Yorba Linda that currently flows into the existing SARI, but will be unable to once the new SARI is relocated to its new alignment on the other side of the river. It consists of 1,428 m (4,685 ft) of 381-mm (15-in.) gravity sewer, including 242 m (794 ft) of siphon pipes inside a 1,956-mm (77-in.) outside diameter (OD) casing pipe installed via microtunneling under the Santa Ana River

for the construction of a twin barrel 305-mm (12-in.) siphon with a 406-mm (16-in.) overflow, and odor control facilities. The project was awarded to LA Engineering for $7.2M.

2. SARI Mainline and Metering Station (Mainline) Contract—Consists of approximately 6,309 m (20,700 ft) of 1,372-mm (54-in.) diameter gravity sewer, with several reaches of 2,578-mm (101.5-in.) OD casing installed via microtunneling. Work includes installation of gravity sewer and casing behind an existing tie-back wall; open trench construction within Canyon RV Park; crossing a documented wildlife corridor adjacent to the Santa Ana River and State Route 91 Freeway; open trench construction within the Green River Golf Club; and several tunneling segments including two planned curved microtunnels, one that will be the longest multiple curve microtunnel in the United States. The design also includes a new metering station to be located within Canyon RV Park requiring construction of a 9.1-m (30-ft) deep by 7.6-m (25-ft) wide and 13.7-m (45-ft) long below grade cast-in-place concrete vault. The vault will house parallel 760-mm (30-in.) pipes with magnetic flow meters, isolation valves and water quality testing equipment. An above ground building houses the electrical equipment, restroom and storage. Upon completion of the new facilities, the contractor will complete the demolition/abandonment of the existing SARI pipeline and metering station. The project was awarded to W.A. Rasic Construction for $41.85M. (Agor, 2012)

This paper provides details of the design and construction of the microtunneling drives which involved the use of both steel and concrete casing in difficult ground conditions. At the time of publishing, two of the five following microtunneling drives have been completed:

1. YLS Siphon Crossing Drive (completed)
   a. 242 m (794 ft) long; 1,956-mm (77-in.) OD; 1,918-mm (75.5-in.) ID; welded steel casing
   b. Launch & receiving shafts: unreinforced secant piles
2. Mainline Gypsum Canyon Drive (completed)
   a. 190 m (622 ft) long; 2,578-mm (101.5-in.) OD; 2,134-mm (84-in.) ID; RCP casing
   b. Launch & receiving shafts: soldier piles and lagging with permeation grouting
3. Mainline Siphon Crossing Drive (planned)
   a. 332.8 m (1,092 ft) long; 2,578-mm (101.5-in.) OD; 2,527-mm (99.5-in.) ID; Permalok casing
   b. Launch shaft: overlapping cutter-soil mix panels and reinforced shotcrete
   c. Receiving shaft: unreinforced secant piles
4. Mainline Coal Canyon 2-Curve Drive (planned)
   a. 477.6 m (1,567 ft) long; 2,578-mm (101.5-in.) OD; 2,134-mm (84-in.) ID; RCP casing
   b. S-Curved; 4,838-m (15,873-ft) radius curve first, then a 4,730-m (15,518-ft) radius curve
   c. Launch shaft: soldier piles and lagging
   d. Intermediate shaft (cutterhead inspection): lattice girder reinforced shotcrete

   e. Receiving shaft (same receiving shaft will be used in other Coal Canyon Drive): lattice girder reinforced shotcrete
5. Mainline Coal Canyon 1-Curve Drive (planned)
   a. 190 m (622 ft) long; 2,578-mm (101.5-in.) OD; 2,134-mm (84-in.) ID; RCP casing
   b. Curved; 1,676-m (5,500-ft) radius
   c. Launch shaft: soldier piles and lagging
   d. Receiving shaft (same receiving shaft will be used in other Coal Canyon Drive): lattice girder reinforced shotcrete

PROJECT BACKGROUND

Constructed in the mid-1970s, the Santa Ana Regional Interceptor (SARI) Line was originally constructed with approximately 6 m (20 ft) of cover within the floodway of the Santa Ana River between Weir Canyon Road and the Orange/Riverside County boundary. In some locations, the low-flow of the Santa Ana River has meandered toward the existing SARI Line and the bed of the Santa Ana River has degraded leaving the SARI Line virtually exposed to the river at several locations requiring the placement of temporary rock riprap revetment and grade stabilizers to protect the SARI Line nearly every year.

The U.S. Army Corps of Engineers (Corps) is working with the flood control districts of San Bernardino, Riverside and Orange Counties to complete the $2.1 billion Santa Ana River Mainstem Project (SARMP). As part of the SARMP, the Corps reconstructed the outlet works for Prado increasing its capacity for making controlled releases from 280 m^3/s (10,000 cfs) to 850 m^3/s (30,000 cfs). Additionally, the Corps widened and strengthened the banks of the Santa Ana River downstream of Prado Dam to accept the planned higher releases. Scour studies completed by Tetra Tech as part of this project indicate that a single 850 m^3/s (30,000 cfs) release may cause significant damage to the existing SARI, and that the anticipated releases over the life of the SARMP (one hundred years) could result in additional riverbed degradation and scour nearing 6 m (20 ft) imperiling the stability of the existing SARI. As a local sponsor for the SARMP, it is the Orange County Flood Control District's (OCFCD) responsibility to relocate or protect utilities like the SARI Line within Orange County that are impacted by the SARMP. The project alignment is illustrated in Figure 1.

The design team led by Tetra Tech, with tunnel and shaft design by Hatch Mott MacDonald, was retained by the Orange County Flood Control District (OCFCD) to complete the preliminary and final design to relocate the existing SARI, as part of the U.S. Army Corps of Engineers Santa Ana River Mainstem project. Within Orange County, the SARI line is owned and operated by Orange County Sanitation District (OCSD) and maintenance costs are shared with a third agency, Santa Ana Watershed Protection Agency (SAWPA) based upon the capacity in the line owned by SAWPA within this reach of the SARI. This project replaces 6.0 km (3.7 mi) of 990- to 1140-mm (39- to 45-in.) VCP and 1070- to 1140-mm (42- to 45-in.) RCP with 6.20 km (3.85 mi) of 1,370-mm (54-in.) gravity sewer, roughly from Green River Golf Club to the SAVI Ranch control gate structure. (Agor, 2012)

ANTICIPATED GROUND CONDITIONS

The SARI Mainline and YLS alignments are located in the lower Santa Ana River Valley, below Prado Dam, which lies within a deeply incised gorge. This area is divided between Riverside County to the northeast and Orange County to the southwest

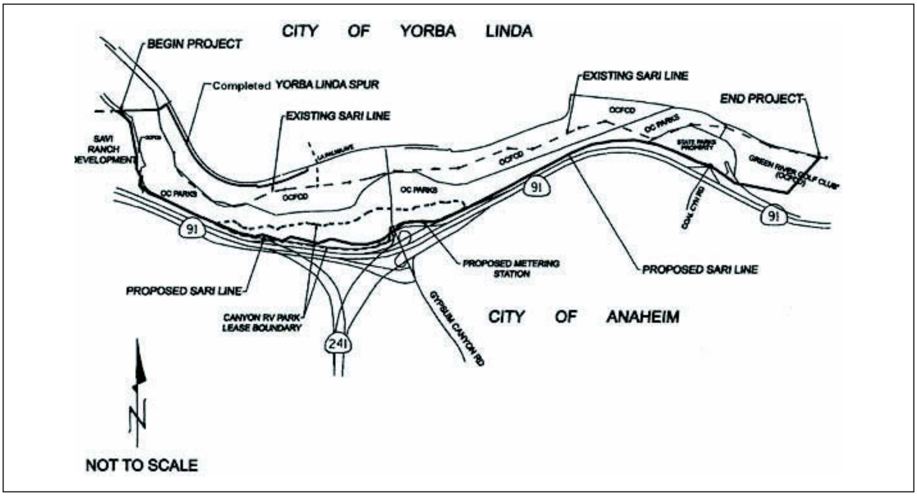

Figure 1. Existing SARI line and SARI Relocation line alignments

between the northern Santa Ana Mountains and southern margin of the Puente Hills, part of the Peninsular Ranges geomorphic province of Southern California.

Both the Santa Ana Mountains and Puente Hills consist generally of sandstones and shale bedrock of Cretaceous to Tertiary age (65 million to 1.8 million years old). The Puente Hills represent a complexly folded and faulted block of Tertiary age marine sediments uplifted between the Whittier fault on the southwest and the Whittier-Elsinore fault to the northeast (Leighton, 2010a).

Baseline Conditions of Subsurface Materials

Based upon the subsurface conditions encountered during the field exploration, their depositional origin and engineering characteristics, the materials that are expected to be encountered during construction are grouped into five units in the Geotechnical Baseline Report (GBR): Sand Mix, Gravel Mix, Silt Mix, Clay Mix, and Bedrock consisting of thinly-bedded Puente Formation and massive sandstone of the Topanga Formation (Leighton, 2010b). The units are summarized in Figure 2.

The microtunnel boring machines (MTBMs) are expected to encounter cobbles and boulders up to 36 inches in size primarily within the Gravel Mix unit and to a lesser degree within the Sand Mix unit. The cobbles and boulders present in the Sand Mix and Gravel Mix are composed of sedimentary, igneous and volcanic rock and could have an unconfined compressive strength up to 30,000 psi.

Representative samples of the Sand Mix and Gravel Mix were tested for slurry abrasivity by ASTM G75 Miller Number Test Procedure. The Sand Mix and Gravel Mix fall into the “Very High” slurry abrasivity category with Miller Numbers of 356 and 377, respectively.

Groundwater was encountered along the entire SARI alignment ranging from the river surface to depths 9.1 m (30 ft) below ground surface. The groundwater fluctuates seasonally, during heavy rainfall and increased releases from Prado Dam. The maximum measured increase in groundwater elevations within the alluvial deposits was over 1.5 m (5 ft) and this occurred between the time period of January 15 and January 28 of 2010 during which time seven inches of rainfall was recorded nearby. Various elements of the SARI Relocation Project will be constructed below groundwater, depending on location.

Figure 2. Soil and bedrock stratigraphy and exposed cut (Leighton, 2010b)

SHAFT AND TUNNEL CONSTRUCTION TECHNIQUES

Shaft Construction Techniques

The YLS Contractor constructed both the launching and receiving shafts utilizing circular, unreinforced 915-mm (36-in.) diameter secant piles as seen in Figure 3. This pile size is a minimum required pile diameter intended to ensure sufficient overlap while encountering cobbles and boulders during construction. The shaft properties are below:

1. YLS Launch Shaft:
    a. 9.8 m (32 ft) clear inside diameter
    b. Pile tips installed to around 13.3 m (43.5 ft) depth
    c. Working slab invert 11.1 m (36.4 ft) deep
2. YLS Receiving Shaft
    a. 5.5 m (18 ft) clear inside diameter
    b. Pile tips installed to around 14.2 m (46.5 ft) depth
    c. Working slab invert 10.7 m (35.0 ft) deep

The Mainline Contractor is in the process of constructing eight shafts using a variety of techniques including: cutter soil mix (CSM) panels, secant piles, shotcrete and lattice girders, and soldier piles and lagging. The Gypsum Canyon Drive included soldier pile and lagging shafts (Figure 3) with permeation grouting around the perimeters and at the entry and exit locations. The Gypsum Canyon shaft properties are below:

1. Gypsum Canyon Launch Shaft:
    a. 6.4 m (21 ft) wide by 10.5 m (34.4 ft) long
    b. Working slab invert 6.1 m (20 ft) deep
2. Gypsum Canyon Receiving Shaft
    a. 4.9 m (16 ft) wide by 8.0 m (26 ft) long
    b. Working slab invert 6.83 m (22.4 ft) deep

Figure 3. Views of the YLS Siphon Crossing launch shaft (left) and the Mainline Gypsum Canyon launch shaft (right)

Lessons Learned for Shaft Construction

Communication between contractors on microtunneling projects is critical to project success. Many microtunnel projects involve multiple subcontractors to prepare the shafts for tunneling. These separate subcontractors may include those who have responsibility over the following: grading/site development, shaft development, headwall/pushwall installation, permeation grouting, surveying, and microtunneling support equipment installation. Some of the communication issues to address include the following:

1. Provision of a relatively water tight shoring system and entry/exit seals to contain slurry and jacking pipe lubrication
2. Shaft sizes to accommodate the microtunneling equipment
3. Shaft invert and headwall location on the correct elevations for the microtunneling operation
4. Shaft area grading and invert to protect against flooding and promote drainage
5. Pushwall design with enough room to mount guidance systems independently
6. Headwall design with enough room to mount seals

On the SARI project, a project requirement is for the prime contractor to enlist a professional engineer to take overall design responsibility for the entry/exit process. The intention of this requirement is for all of the subcontractors, including the microtunnel subcontractor, to communicate their needs to one designer who can develop a robust strategy for this process.

Tunnel Construction

MTBM Design

The minimum specified microtunneling requirements including the requirements for face access and an air lock were based on the Engineer's interpretation of the expected ground conditions, which includes high groundwater levels, flowing soil behavior, abrasive soil and boulders. Vadnais Corporation (YLS microtunnel subcontractor) and James W. Fowler Company (Mainline microtunnel subcontractor) each selected Herrenknecht slurry microtunneling machines with these capabilities.

Compressed air tunnel work has been permitted by Cal/OSHA in Southern California before. Cal/OSHA has a process for granting variances away from their decompression tables, if necessary, but this process takes time and needs to be started early. Bidders were encouraged to contact Cal/OSHA to gain an understanding of what is required to obtain the necessary permits to perform compressed air interventions. Additionally, the contractors each benefited from hiring a specialty dive company as a sub for the compressed air work, if needed, to lean on their expertise. This approach helped clarify what equipment is necessary to perform the work once the variance is obtained.

MTBM Operation

The slurry microtunneling machine utilizes a closed loop slurry system to transport soil from the face during machine advance, balance naturally occurring groundwater pressure, and mitigate adverse ground conditions. In highly permeable ground conditions with groundwater that does not cover the tunnel, slurry will tend to flow away from the face of the machine. Contractors can reduce this fluid loss and improve ground control by using slurry consisting of water with bentonite, polymers, and additives that tend to create a plug in front of and around the microtunnel MTBM face. Some slurry additives may have the added benefit of reducing wear on the cutterhead as well as the slurry circuit. The consequences of not properly treating the slurry may include over-excavation at the face leading to settlement, additional wear, and loss of slurry circulation. If the slurry does not circulate, the slurry cannot remove the soil thereby preventing the MTBM from advancing. The loss of slurry circulation may also cause the soil to drop out of circulation, plugging the slurry chamber and slurry lines, which could lead to the contractor performing an intervention.

The use of external dewatering or increased shaft sump pumping may further reduce slurry circulation in certain groundwater conditions. On the SARI project, the contractor is required to design the slurry for the soils, manage the slurry system and soil separation plant, and operate the MTBM without significant over-excavation within the existing ground and groundwater conditions.

CONSTRUCTION OUTCOME

At the time of publication, two of the five SARI tunnels have been completed: the YLS Siphon Crossing Drive and the Mainline Gypsum Canyon Drive. This section highlights some of construction related information for each of the tunnels including some lessons learned.

YLS Construction

Vadnais began the YLS Siphon Crossing tunnel in mid-May and finished contact grouting in mid-June 2012. The baseline ground conditions were the following:

- ~21 lineal meters (70 ft) of mixed-face conditions with Silt Mix and Clay Mix overlying Puente Formation
- ~223 lineal meters (730 ft) of Sand Mix
- Cobbles and boulders expected to be encountered
- Number of boulders approximately 915 mm (36 in.) in size measured along the longest dimension: 1
- Tunneling completely below the groundwater table

It took 5 days (12 hour day shift only) to completely launch the 16-m (52-ft) long Herrenknecht AVND-1600AB MTBM into the ground that bored a path for the 1956-mm (77-in.) OD welded steel casing pipe. What remained of the 242-m (794-ft) tunnel, constructed primarily on a 24/7 basis per contract requirements, took 16 days for the MTBM to reach the receiving shaft seal. Extraction of the MTBM and removal of equipment from the tunnel occurred over a three day period. Contact grouting of the entire tunnel was performed over six days. Details of the entire tunnel schedule are illustrated in Figure 4 and include details of some of the downtime. Figure 5 illustrates the production cycle data from the YLS tunnel. The production cycle data came from

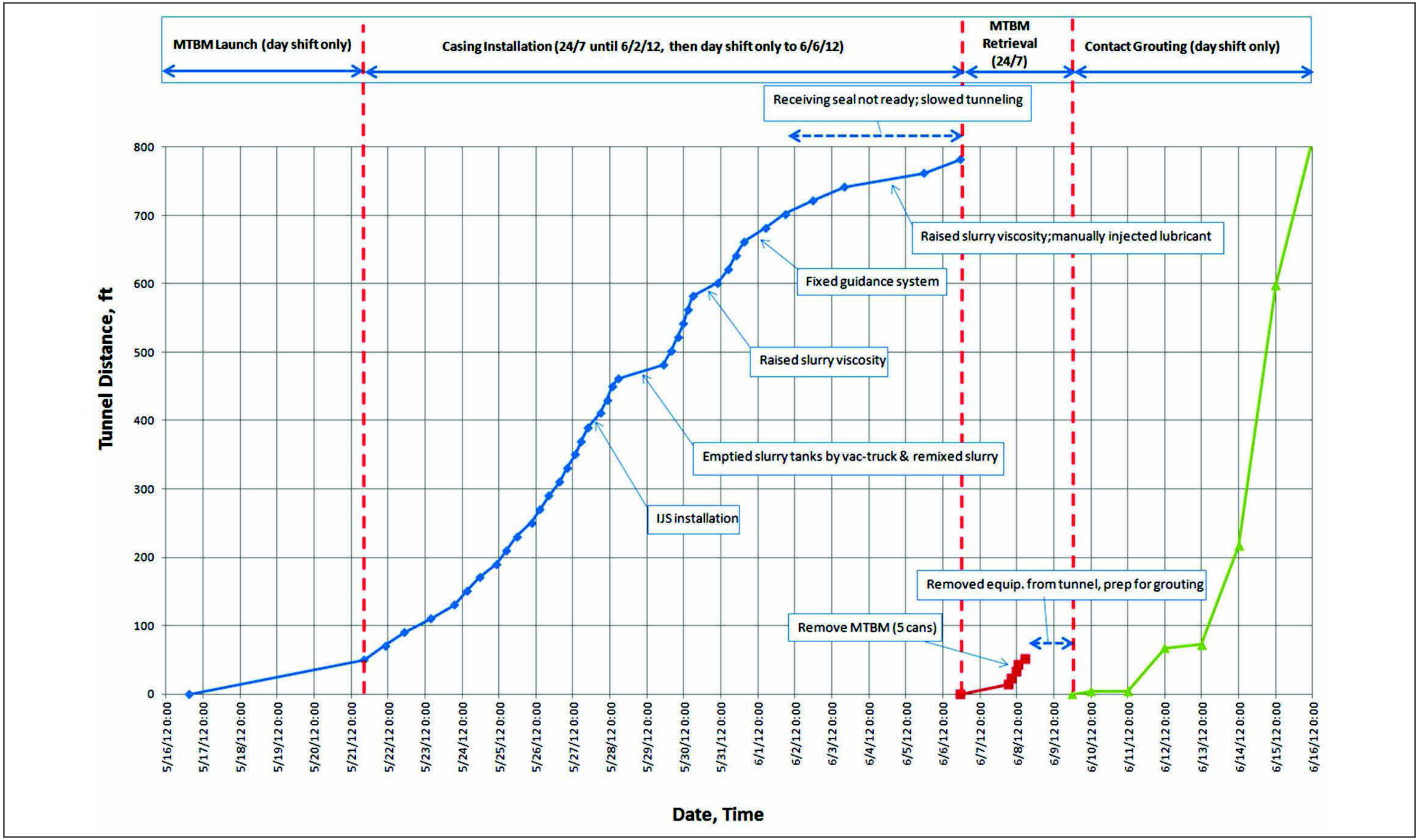

Figure 4. YLS Siphon Crossing tunneling schedule

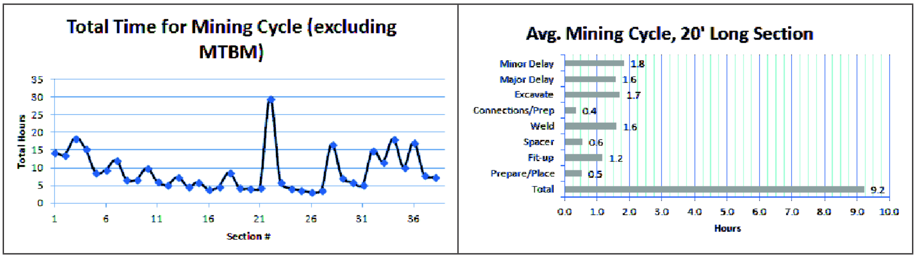

Figure 5. YLS production cycle data

Hatch Mott MacDonald inspection records that tracked the hours of activities affecting the utilization rate of crew time. Major delays were assumed to be single events that last more than four hours. Minor delays included activities that impacted the standard production cycle. These minor delays included surveying, slurry adjustments, installation of some tunnel equipment. Vadnais made the effort to coincide the (required) daily survey checking with the production cycle to minimize delays. For the majority of tunneling, they were successful in performing survey checks concurrently with tunneling.

The following paragraphs highlight some incidents that led to downtime. Where possible, suggestions are offered to improve utilization on future projects. Although boulders were likely encountered, no interventions were necessary along this alignment as the MTBM was able to digest all the excavated material.

- Before MTBM launch, the VMT laser-theodolite guidance system was held up in U.S. customs for about a month after it arrived from Germany. Sourcing this system early (and other critical international parts) is recommended and possibly hiring a customs expediting specialist might be worthwhile.
- During the MTBM launch, one of the machine’s four steering cylinders stopped producing reliable position data. Herrenknecht immediately responded by sending out a replacement cylinder from Germany as well as working with Vadnais over the phone. Within two days, one of Herrenknecht’s electricians came to the site and worked with their operator to diagnose a faulty electrical connection that was repaired and calibrated. This proved beneficial because the part from Germany took much longer to arrive as it had to pass through customs.
- The jacking frame used for this drive included two yoke positions that pushed the casing pipe 3 m (10 ft) before a “birdcage” spacer was added behind the casing to push the remaining distance of each 6.1-m (20-ft) pipe. Because the jacking frame could not push the entire length of casing without the spacer, time was lost for the crane to place and remove it for each section of casing pipe. The yolk on the Herrenknecht compact frame can typically change between positions within less than five minutes to resume tunneling, while using the spacer on this project required up to three times that amount of time. A jacking frame with more yolk positions would have required a longer shaft, but time is lessened without using the spacer. As an additional advantage, risk of injuries is reduced by moving less equipment (the spacer) overhead of the shaft with the crane. Based on drive records, the YLS Contractor might have saved 20 hours of crew time by not using the birdcage spacer.
- Welding of the 1956-mm (77-in.) OD steel casing was reportedly performed by two welders over a 1.5 hr (average) or longer period. Some inefficiency occurred during the fit-up of the steel pipe prior to welding. At best, this fit-up was performed in around 30 minutes using a Mathey Dearman double chain

Figure 6. YLS casing fit-up using a double chain clamp

clamp (see Figure 6). However, some fit-ups were recorded as taking over four hours when the steel pipes did not align well. Timber stulls were removed by the Contractor from within the casings once on-site. Typically, maintaining the stulls within the casings is recommended by manufacturers as a way to maintain the roundness of steel pipe and reduce fit-up times. Additionally, welders and fit-up experts familiar with the use of the double chain clamp have the most success with using it for fit-up. The welders who achieved the shortest fit-up times simultaneously tightened the clamp's dogs as they each used electric impact wrenches while working from top to bottom. If the YLS day crew had more familiarity with the double chain clamp and achieved the night shift's average fit-up time, the YLS Contractor might have saved up to 20 hours of crew time.

- Slurry plant and slurry mixture problems contributed to the principal loss of production time during construction of the YLS Siphon Crossing. When excavating with slurry MTBM, the contractor must adjust the slurry properties to meet the ground conditions encountered at the face of the excavation. Slurry helps the microtunneling process in the following ways: it forms a filter cake in permeable soils to prevent fluid loss, it enables transportation of spoil from the excavation chamber to the surface separation plant, it supports the excavation face by balancing ground and groundwater, it helps spoils to be removed from the slurry at the separation plant, and slurry may reduce the wear and tear on parts of the machine and ancillary equipment. The YLS Contractor lost approximately 30 hours of crew time due to slurry issues.
- One of the primary slurry ingredients, besides water, is fast hydrating powdered bentonite for use when excavating permeable soils. As the content of fully mixed (and hydrated) bentonite within slurry goes up, so does the viscosity of that slurry. As the MTBM encounters different ground conditions, the operator must have the ability to efficiently adjust the slurry to meet the demands of the soils. The separation plant on the YLS tunnel suffered from the inability to efficiently mix bentonite into the slurry because of an inadequate delivery and mixing system. Therefore, downtime was encountered each time the contractor attempted to raise the viscosity of the slurry. Approximately

15 hours of crew time might have been saved if a more efficient mixing system was utilized.

- To properly mix and hydrate the bentonite, clean fresh water with the proper pH (between 8.5 to 10) is necessary to start with. Soda ash, a relatively cheap ingredient, should be kept onsite to mix into the system to neutralize calcium ions and adjust the pH. When tunneling through cementitious materials, such as a concrete shaft or grout blocks, slurry containing bentonite may react with the cement and turn the slurry into thick, unmanageable 'ketchup.' Traces of the cement must be removed from the separation plant to reduce the risk of this happening to the slurry. Unfortunately, on the YLS Siphon project, some cementitious material remained after tunneling through the secant pile walls and downtime was incurred to drain the thick slurry out of the tanks and remake fresh slurry. Contractors might benefit in using separate holding tanks to efficiently remove cement contaminated slurry. Proper test kits to check for inadequate slurry properties are also recommended.
- If separation plants are not designed to settle material within the holding tanks, the plant benefits from ways of agitating the slurry within holding tanks to help prevent excess sediment building up in the bottom of the tank. This agitation helps to keep sediment within circulation, allowing the mechanical separation equipment more opportunities to remove it from suspension. Easy access within separation plants allows for efficiently cleaning out sediment buildup. Adjustments to spoil volume estimates also need to be made if sediment is building up within slurry tanks. On the YLS Tunnel, an excessive sediment buildup had to be extracted from the tanks using vac-trucks. This downtime is noted on Figure 4 and contributed the bulk of the major delay time.
- Problems with the VMT guidance system led to downtime on a few occasions during the tunnel drive. The most time lost was when the MTBM's active target unit (ELS) receiver was transmitting erratic line and grade positions to the operator's screen. VMT had been monitoring the guidance system remotely through an internet portal during the entire drive and supported Guida Surveying who was on-site every day, per contract requirements, to check the position of the MTBM. The problem was fixed when the MTBM's ELS receiver was changed out and the tunnel finished the drive within acceptable guidance limits.
- When the MTBM was nearing the end of the tunnel drive, Vadnais attempted to bolt on the rubber seal to the concrete headwall at the receiving shaft. Unfortunately, they were supplied with a poorly manufactured rubber seal that began to delaminate from just the miners handling it. Two new rubber seals had to be fabricated by a different manufacturer, International Belt and Rubber, and shipped down from the Pacific Northwest. To prevent the buildup of excessive jacking pressures if the pipeline were to sit still for too long, Vadnais switched to day shift and attempted to push only one casing segment a day. This strategy bought the contractor some time, but thrust forces increased dramatically to restart tunneling after overnight shutdowns. Additional downtime was incurred for the crew to manually inject lubricant at multiple injection ports all along the pipe string. The operation would have benefited from an automatic lubrication system that the operator could control and cycle injection points more quickly than doing so manually.

Gypsum Canyon Construction

Fowler began the Gypsum Canyon tunnel in mid-October and finished contact grouting in mid-November 2012. This tunnel drive is 190-m (622-ft) long and ranges between 4.3 m (14 ft) to a maximum of 8.5 m (28 ft) below ground surface to tunnel centerline. The baseline ground conditions were the following:

- ~146 lineal meters (480 ft) of mixed-face conditions with Gravel Mix over Sand Mix
- ~37 lineal meters (120 ft) of Sand Mix
- ~6 lineal meters (20 ft) of Gravel Mix
- Cobbles and boulders will likely be encountered
- Number of boulders approximately 915 mm (36 in.) in size measured along the longest dimension: 2
- Baseline groundwater elevation above invert but below crown of tunnel

It took 17 days (day shift only) to completely launch the 13.4-m (44.0-ft) long Herrenknecht AVND-2000AB MTBM into the ground that bored a path for the 2,578-mm (101.5-in.) OD RCP casing pipe. Before half of the first MTBM segment was excavated, approximately 15 of these days were lost due to a hydraulic pump and electrical power plant failure. What remained of the 190-m (620-ft) tunnel, constructed on a day-shift basis, took 16 days for the MTBM to reach the receiving shaft seal. Extraction of the MTBM occurred over a two shift period. Removal of equipment from the tunnel took place over a 10 day period. Details of the entire tunnel schedule are illustrated in Figure 7 and include details of some of the downtime. Figure 8 illustrates the production cycle data from the Gypsum Canyon tunnel. The production cycle data was analyzed in the same manner as the YLS data.

The following paragraphs highlight some incidents that led to downtime. Where possible, suggestions are offered to improve utilization on future projects. Although boulders were likely encountered, no interventions were necessary along this alignment as the MTBM was able to digest all the excavated material.

- The MTBM experienced some difficultly as it excavated through the permeation grout outside of the launching shaft. The tunnel subcontractor reported difficulty transporting PVC pipe pieces through the slurry system that were left in place from the permeation grouting. Prime contractors may benefit from holding subcontractor collaboration meetings to help each group understand the other's needs.
- The Herrenknecht MTBM experienced difficulty using the telescopic station incorporated into the last can of the machine. The telescopic can was able to expand its thrust cylinders to push the face forward, but the operator chose not to close the station. The high thrust force needed to close the station resulted in increased face pressures so it was left open for the remainder of the drive due to a risk of heave to the surface.
- To monitor muck volumes, the contractor removed muck in-between pipe segments from the slurry plant and kept track of volumes based on bucket logs. This is necessary to compare the estimated excavation volume to the theoretical volume. This cleanout must be done consistently after each pipe segment is completed for accurate volumes. Also, different bulking factors may need to be applied depending on the type of spoil seen coming off the separation plant.
- Guidance systems must always be independent from portions of the shaft that may move during pipe jacking. To avoid problems, install the guidance system separate from the thrust wall, jacking pad and jacking frame. On the Gypsum

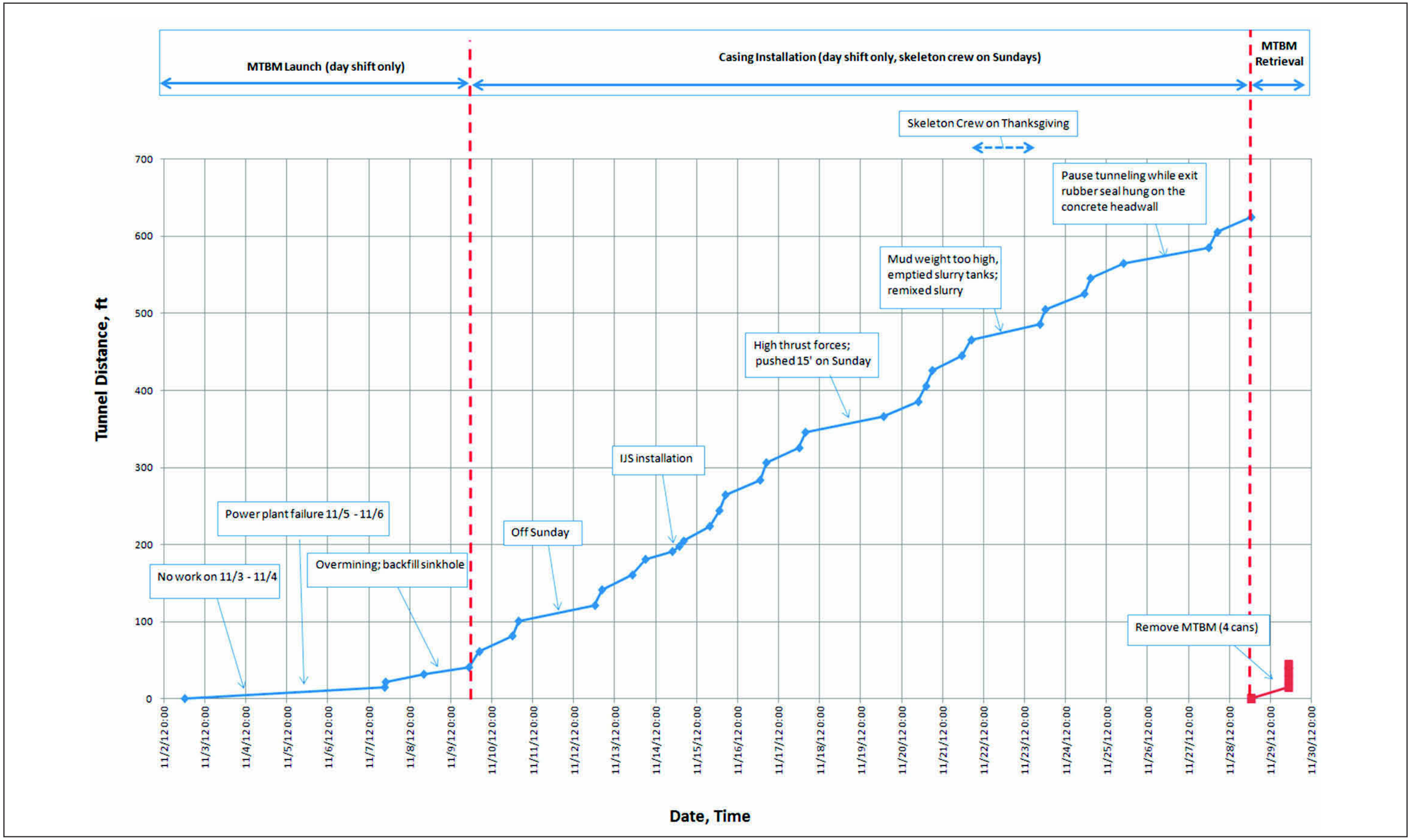

Figure 7. Mainline Gypsum Canyon tunneling schedule

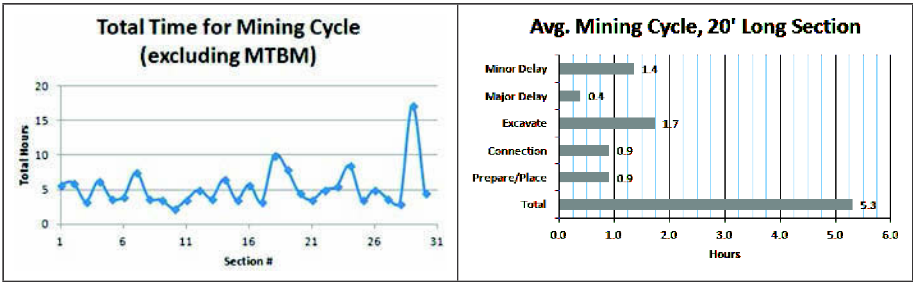

Figure 8. Mainline Gypsum Canyon production cycle

Canyon drive, the guidance system was mounted on a pillar connected to the base of the shaft. When high pipe jacking thrust forces were required, the laser was seen to be moving. This observation indicated the shaft floor was moving. Guidance systems benefit from a soft connection between the thrust wall and the shaft floor to prevent this movement.

BACKFILL GROUTING OF PLASTIC CARRIER PIPES

After the completion of each tunnel, carrier pipe(s) are installed within spacer supports, and grout is injected between the outside of the carrier pipe(s) and the inner surface of the tunnel casing. For the three tunnels where 1370-mm (54-in.) Hobas carrier pipe will be installed on the Mainline, lightweight cellular grout backfills the annulus. For the two siphon crossings that utilize plastic carrier pipes, a neat-cement grout is required to meet the buoyancy design requirements.

The two siphon crossings were both designed to prevent long-term uplift of the tunnel from buoyancy. To accomplish this in the design, the finished tunnel weight per foot was calculated as the resisting force against the uplift from the soil displaced by the tunnel. The carrier pipes were assumed to be full of water and the steel casings, which are not coated for corrosion protection were ignored in the buoyancy evaluation. The design did not allow for voids to be present within the completed tunnel where grout failed to fully penetrate the entire volume of the tunnel. Therefore, a neat-cement grout was required that would have maximum flowability to fill all the voids as well as decreasing the risk of aggregate clogging the feed lines during pumping.

The finished YLS tunnel layout is illustrated in Figure 9. The bottom portion of the casing was filled with normal weight concrete and sloped at the required angle for the carrier pipes. Afterwards, two 305-mm (12-in.) and one 406-mm (16-in.) HDPE pipes were fastened on redwood boards and strapped in-place on 4.6-m (15-ft) centers to prevent flotation and movement. PVC vent and grout feed pipes were placed in the crown, followed by bulkheads at each shaft. The next step, before backfill grouting around the plastic piping, was to mitigate the heat of hydration of the cement to prevent damage to the carrier pipes.

The heat of hydration of a grout mix depends on factors including: the type of cement, the amount of cement content, the initial placement temperature, and the types of interfaces the grout is in contact with. Vadnais, along with their subcontractor Cell-Crete, undertook the following steps to limit the curing heat from negatively affecting the HDPE carrier pipes:

- Replaced 65% of the cement content with Class F flyash to minimize the peak heat of hydration
- Used type II/V Portland cement

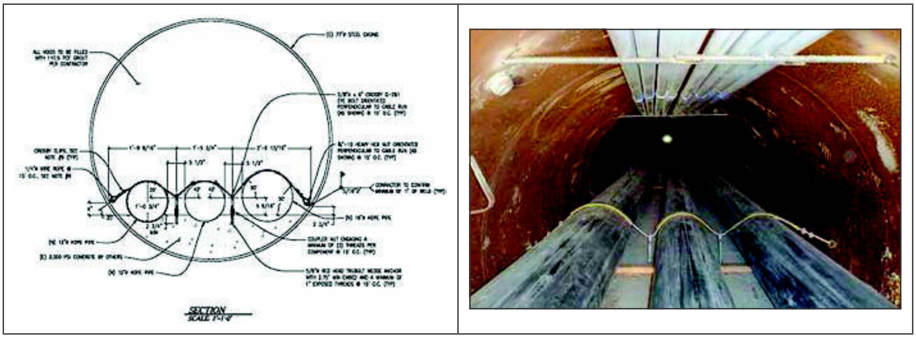

Figure 9. View of the backfilling operation for the YLS Siphon Crossing tunnel

- Removed heat by running a continuous flow of water through all three pipes at 1,140 Lpm (300 gpm) total during the first week of the grout curing process
- Monitored the temperature of the cooling water outflow; the cooling water pumps included the ability to increase the flow if necessary
- Monitored the temperature of the grout/pipe interface; the cooling water pumps included the ability to increase the flow if necessary
- Placed grout in multiple lifts over a three day period

The backfill operation was successful on the YLS tunnel using the above steps. Almost no change in cooling water temperature was noted even after the water flowed through more than 244 m (800 ft) of HDPE carrier pipes. Also, the temperature monitor at the grout/pipe interface only reached a maximum of 41°C (106°F), well within the acceptable limits.

CONCLUSIONS

The project's geotechnical conditions included challenging ground conditions with risks for the fracture of tunnel fluids, abrasive ground affecting MTBM operation, and encountering cobbles and boulders. Additionally, tunneling carried the risk for settlement of roads and the existing SARI line. The Herrenknecht slurry machines chosen by each contractor have succeeded in digesting all of the encountered ground on the two completed tunnels without interventions.

This paper included a detailed analysis into the production cycles for the two completed tunnels. Several downtime incidents were discussed along with implications and possible fixes on future projects.

Regarding backfill grouting of plastic pipes, the heat of hydration must carefully be estimated and controlled. Some of the successful steps implement on the SARI project were discussed in this paper.

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DESIGN-BUILD TUNNELING PROJECT IN URBAN SETTING WITH FAST-TRACK ENGINEERING AND GROUND MODIFICATIONS

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Michael Gilbert ▪ CDM Smith

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Paul Schmall ▪ Moretrench

ABSTRACT

The paper presents a design-build case study of the Tingey Street Diversion Sewer Tunnel in Washington, DC. The project consists of 1,180 feet of 72-inch ID microtunneling, 110 feet of pilot-bore installation of a 36-inch ID tunnel, multiple diversion structures and manholes, grouting for protection of existing infrastructure, and for ground improvements. Work is performed within a major Anacostia waterfront development near the Washington Nationals stadium. Project challenges include schedule, constraints for construction due to existing infrastructure constraints and interferences and ongoing development activities. Fast-track engineering is successfully applied to coordinate design time and construction time to meet project milestones.

PROJECT HISTORY

The Tingey Street Diversion Sewer (TSDS) is a one element of the Clean Rivers projects. It is part of a Consent Order by the U.S. EPA ordering improvements to city infrastructure as part of a long-term control plan for storm water runoff and to alleviate combined sewer overflows (CSOs) in much of the city. The TSDS consists of a 1,200 feet microtunnel consolidating two CSO's (identified as CSO-013 and CSO-014) through diversion structures and will eventually be connected to the 13.5 miles, 23 feet ID storage tunnel (Figure 1).

The overall objective of the TSDS is to divert and transfer the flows from two CSO lines, CSO-014 and CSO-013, via a minimum 66-inch tunnel along Tingey Street. The tunnel will convey the flow to a connection point at the interface between the Division B Contract and the Division I Contract, work not included in this contract.

The alignment extends from the intersection of 5½ Street and Tingey Street at Station 21+80.36 flowing west to the interface location with the Division I Contract located at Station 10+53 near the intersection of 2nd Street and Tingey Street.

There are several major challenges that were addressed in the design portion of this design build project. The advantage of a design build contract has allowed the designer to develop a design that was adapted to the means and method best suited by Northeast Remsco Construction (NRC) as the lead in this project. NRC in turn is a subcontractor to Forest City Realty Group a major developer in the DC area.

This paper presents the subsurface conditions that were used as the basis for the geotechnical design and construction decisions along with the discussions held between NRC and CDM Smith the lead designer; details of the specific constraints and obstacles that had to be addressed in the design; our design approach to mitigate

Figure 1. Tingey Street

the impact of these items and still allow for an economical construction; and, once construction is on-going a presentation will include a comparison of predicted and measured values.

SUBSURFACE CONDITIONS

The subsurface data used as the basis of the geotechnical design was prepared by the DCCR as part of the contract documents. Our interpretation of that data with regards to the proposed work in this project was summarized as follows.

Sta 10+50 to 14+75, approximately 425 lf, the tunnel will be in the finer grained Alluvium consisting of loose to medium dense silty sand, clayey sand and poorly graded sand. SPT N values range from 9 to 29. Hydrostatic pressure on the tunnel face at springline will be about 0.6 bars. In an unsupported face the tunneling behavior of this ground is expected to be a flowing condition.

Soil profile from Sta 14+75 to 17+25 the tunnel is still expected to be in Alluvium, but 30 feet of it will be coarser grained. The soil is medium dense to dense, N values in the 20 to 50 range, predominately consisting of poorly graded sand and poorly graded gravels. The borings encountered refusal either at tunnel horizon or within one diameter of the tunnel horizon. Refusal was identified on the geotechnical profile as 50 blows with less than 6 inches of penetration of the split spoon sampler. We interpreted these refusals as either cobbles or boulders. It is anticipated that some cobbles and boulders will be encountered during the tunneling. We estimated that 22 boulders and 10 cobbles will be encountered. The hydrostatic pressure on the tunnel face at springline is expected to be 0.5 bars of pressure. The stratum has high permeability and in an unsupported face will behave as a fast raveling material.

Between Sta 17+25 to 18+50 the tunnel continues in a finer-grained Alluvium. Groundwater pressure is expected to remain at the 0.5 bar pressure. Tunneling behavior of the ground in an unsupported condition will be slow raveling.

Starting at Sta 18+50 and extending to 21+94, the geology will transition to the Fill stratum that consists of very loose and very soft soil consisting of highly plastic organic clay and silty sand. The silty sand material that can also be expected will behave as a flowing material in an unsupported face as it is subjected to 0.5 bars of hydrostatic pressure.

PROJECT TECHNICAL ISSUES

Major design issues that were addressed to mitigate potential problems during construction were:

- A receiving shaft only 12 feet from the 100-plus year old brick lined and cracked Tiber Creek Sewer

- A launch shaft in the middle of the intersection of 4th Street and Tingey Street that is within the WMATA Zone of Influence
- Crossing under the existing East side Interceptor sewer with a vertical clearance of about 3 feet
- A second receiving shaft tight up against a historical brick building
- Compressible fill ground in which modification using jet grouting was specified

Launch Shaft and Tiber Creek Sewer

The Tiber Creek Sewer is an approximately 14-foot-diameter, horseshoe-shaped, brick and concrete sewer that was constructed in 1902 to convey the drainage that was formerly carried on the surface to the Tiber Creek, which was subsequently backfilled. This sewer is very old and fragile.

The Klein and Hoffman report on the condition of the Tiber Creek Sewer indicated that no cracking of the sewer was observed in the single barrel portion of the sewer (i.e., the portion of the sewer near the launch shaft). Additionally, according to the Rice Associates survey conducted within the sewer in 2010 indicated that cracks in the sewer walls were only observed in the double barrel and transition sections approximately 100 feet south of the launch shaft. The Rice Associates survey further indicated that the sewer is horseshoe shaped and changes configuration at a point approximately 35 feet south of the launch shaft. South of this point, the interior of the sewer is brick below the springline (cradle section) and concrete above the springline (arch section), while north of this point, the interior of the sewer is entirely brick. The original contract drawings for the sewer show a section that consists of a brick-lined concrete cradle section and a concrete arch section.

The contract drawing showing the location of this sewer states a 12 feet minimum offset from this trunk sewer is required. Two initial concerns to the Design-Build team were the effect of installing the sheeting for the lateral support of the launch shaft and the jacking loads and their impact on the sewer. Based on research conducted for a previous project in Seattle, Washington, our opinion is that angular distortion is the primary driver in damage to brick sewers. McKim and Barsoom* (2002) suggested that damage can occur to brick sewers due to slope changes of greater than 0.3 percent and significant damage can occur due to slope changes of 0.6 percent. Based on the findings of the inspection of the sewer, it is in poor condition. The existing cracks in the sewer, although a concern, are not in the immediate area that would be subjected to jacking loads.

East Side Interceptor Sewer

The most critical tunneling zone is at Sta 17+05 where the Tingey Street Diversion Tunnel crosses under the existing 6'-3" East Side Interceptor sewer. There is only 3 feet of clearance between this structure and the new tunnel. Prior to tunneling in this area, we evaluated various methods to improve the soil under the existing interceptor. That evaluation is presented in Table 1.

The areal extent of ground to be modified extends 15 feet to either side of the ESI for a distance of 30 feet (15 feet on either side of the intersection of the Tingey Street Tunnel and the ESI). The vertical depth of the modified ground zone is approximately 12.5 feet extending from springline of the ESI to 3 feet below invert of the Tingey Street Tunnel. The GDR data shows the material in this zone as medium dense silty sand with gravel consisting of 15 percent to 25 percent fines.

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* McKim, R.A., Barsoom, J.W. (2002). Impact of Constructing a Large Diameter Tunnel near a Large Diameter Brick Sewer in Denver—A Case Study–Part 1, North American Society for Trenchless Technology (NASTT) NO-DIG 2002.

Table 1. East side interceptor sewer evaluation

Type of Ground Modification	Advantage	Disadvantage
Ground freeze	■ Self-supporting ground	■ Ground heave ■ Difficult to control freeze growth ■ Limited access to install freeze elements
Compaction grouting	■ Self-supporting	■ High risk of movement/damage to the ESI
Permeation grouting	■ Injected under low pressure ■ An increase soil strength ■ Will reduce permeability	■ Ground contains 15 to 25% fines, difficult to penetrate
Jet grouting	■ Self-supporting ground	■ High risk of movement or damage to the sewer ■ Difficult to control the grout zone

WMATA Proximity

The Tunnel crosses under the ESI only about 40 feet from CSO-013 MH structure. This structure is required to collect the flow from the ESI at the diversion structure where it will be discharged via a small diameter tunnel into the new Tingey Street Sewer tunnel. Both the ESI crossing and the 013 MH are within the zone of influence of WMATA. Therefore the proposed construction method has to be approved by WMATA.

Weak Soils

The contract documents specify ground support of the tunnel between Sta 18+50 and 22+00.

Historical Building

Building #74 is designated as a historical building built in the 1930s. It is a brick multi-story structure that is shown to bear on piles. The inspection of the structure shows it to be in poor condition regarding existing cracks in its southwest corner which is just to the east of our proposed exit shaft. The clearance between the pile cap and the sheeting required to the support of excavation is less than one foot. To make the connection with the existing outfall there is no option to relocate the shaft. On the northwest side the shaft is also bounded by a conduit containing optical lines and on the south side there is an existing 30-inch drain line and manhole. The existing outfall that will be tied into the new Tingey Street sewer is a 75-inch outfall sewer that within the footprint of the shaft is partially supported on piles and flow is tidal.

Connection to Existing Active CSO Line

The purpose of CSO-013 Diversion Structure is to direct the overflow collected from the existing East Side interceptor at the new CSO-013 Diversion Chamber into the TSDS via a 36-inch diversion tunnel that will also be constructed using trenchless means. There is very little surface work area available at this location.

SOLUTIONS

Tunneling

The soil profile as previously described has the probability of encountering boulders as well as having to tunnel in soft soils. We felt that it was imperative to provide a MTBM that could address these potential problems as well as the issue of tunneling below the

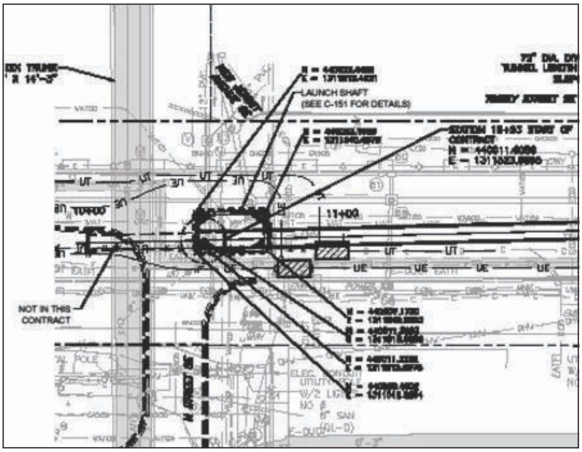

Figure 2. Contract drawing of circular exit shaft and proposed launch shaft

groundwater and controlling ground loss so we stay below the threshold limits established for surface settlement.

Our proposed microtunneling machine (MTBM) is a NRC-owned Herrenknecht AVND 1800 AB. This machine is designed for soft ground with mixed ground conditions such as cobbles and boulders; there is access to the cutting wheel for tool replacement; it has a very reliable guidance and steering system and the NRC MTBM operators are experienced in the use of this machine in weak soils. There is a variable flushing mode to suit different ground conditions. The medium pressure water system is compatible with tunneling in cohesive soil. It also has a highly effective cone crusher for boulders. It is completely remote controlled so manned entry into the installed pipe string during tunneling is normally unnecessary.

Our approach is to tunnel through CSO-013 MH shaft location and after completion of the entire tunnel to complete the construction of the manhole at this location. The construction sequence will be to install a 12 feet by 12 feet shaft with four tiers of wale support. The bottom tier will be replaced by backfill to tunnel springline elevation with a low strength cement grout. Using the guillotine approach for the sheeting break-in and break-out the tunnel will pass through the shaft. Upon completion of the tunnel shaft will be excavated to springline along with cutting and removing the upper half of the RCP and building a dog-house structure over the exposed tunnel. The shaft will be completed to ground surface. A third guillotine will be used for the break-in of the 36-inch MTBM used for the connecting diversion tunnel.

Launch Shaft

To address each of these critical elements the plan developed by the Design-Build team was to perform the tunnel in one continuous drive from west to east if it was deemed feasible. Detailed analyses of the effects to the Tiber Creek Sewer due to the change in tunneling operations were required. These analyses evolved from a very simple and basic closed form analysis consisting of a 2D hand calculation to a detailed 3D analysis (Figure 2).

After the sequential evaluation of this data with the 3D model and consideration of the utilities in the immediate area of the footprint of a launch shaft we decided to perform the work from the west end of the job and ensure the safety of Tiber Creek Sewer to relocate the launch shaft. To accomplish this, we decided to make the following changes in our design:

- Move the launch shaft 5 feet east, further away from the sewer so it is now 17 feet from the sewer.
- Design the sheeting to fully engage the soil.
- Use a variable moment hammer.
- Account for the resistance of soils beneath the base of the excavation due to the sheeting extending 5 feet below subgrade.
- Install instrumentation to confirm that the sewer is adequately protected before the full jacking load is applied.
- We also moved the shaft north approximately 5 feet to avoid interference with the south wall of sheeting and an abandoned steam tunnel.

Tiber Creek Sewer—3D Modeling

Soil parameters for the Linear Elastic (LE) constitutive model used in the 3D modeling include density and bulk and shear modulus values. These parameters were developed based on the available subsurface boring data, in-situ and laboratory test results, as discussed below.

For the purposes of modeling and based on the inspection reports, we assumed that the southern section of Tiber Creek Sewer was constructed as shown on the contract drawings, while the northern section was constructed as shown on the contract drawings up to the springline, and the arch section was constructed entirely of brick. The concrete was conservatively assumed to be 3,000 psi concrete.

The deformation behavior of the Tiber Creek Sewer under the anticipated loading conditions is primarily controlled by the soil modulus (stiffness) parameters, which vary with strain. Therefore, the selection of appropriate modulus values required an understanding of the strain that is anticipated under the anticipated loading conditions.

We conservatively selected modulus values equivalent to a 50 percent secant modulus (e.g., the equivalent modulus value for a stress level equal to 50 percent of the failure stress), which typically corresponds to shear strains in the range of 0.5 to 1.0 percent.

Table 2 summarizes the selected soil parameters. The Young's modulus, shear modulus, and bulk modulus provided in the table are related by functions of Poisson's ratio, and all four parameters can be defined by any two. FLAC3D uses Shear and Bulk Moduli as input values and the other parameter values are provided for reference.

We developed a single model that establishes the *in-situ* stress conditions, including installation of the Tiber Creek Sewer and the steam tunnel adjacent to the launch shaft location, evaluates the impact of excavations in the launch shaft, and incorporates the jacking forces that will be imparted during the microtunneling operation.

Table 2. Summary of soil parameters

Soil Strata	Density, γ_t (pcf)/ Mass Density ρ (slug/ft^3)	Poisson's Ratio, ν	Young's Modulus, E (ksf)	Shear Modulus, G (ksf)	Bulk Modulus, K (ksf)
Sand fill	100/3.11	0.32	600	230	550
Clay fill	100/3.11	0.45	230	79	770
Alluvial clay	110/3.42	0.45	200	69	670
Alluvial sand	125/3.89	0.31	380	140	320
Potomac clay	125/3.89	0.45	1,500	520	5,000
Potomac sand	130/4.04	0.28	2,800	1,100	2,100

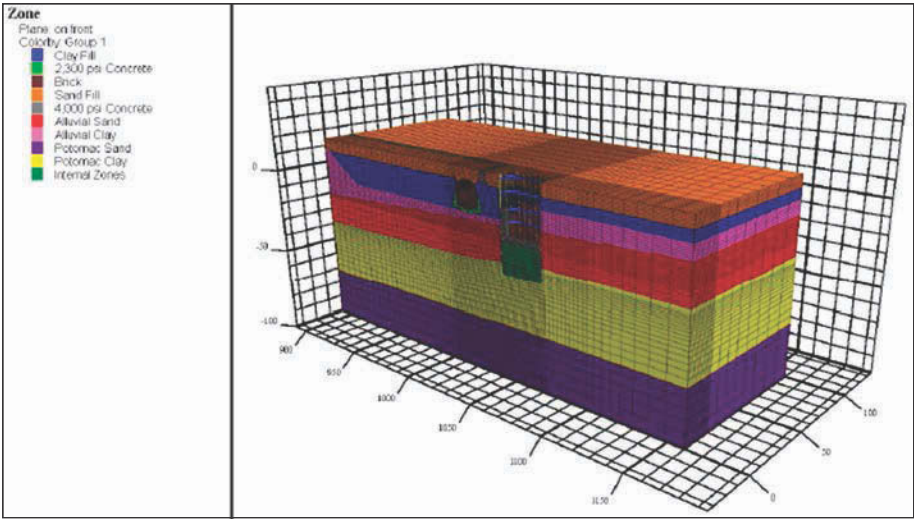

Figure 3. Cut through center of excavation to show interior details of model

The modeled soil stratigraphy (Figure 3) was based on the geotechnical profile, provided in the GDR. The model covers the portion of the profile roughly between Stations 8+90 and 11+60. The model also includes the existing Steam tunnel, the existing Tiber Creek Sewer and the proposed Launch Shaft.

The mesh consisted of 1-foot cubic elements near the Launch Shaft, with the mesh getting coarser towards the edges. Boundary conditions on the sides and bottom of the model are set as roller boundaries. The abandoned steam tunnel lies immediately adjacent to the launch shaft, turns 90 degrees to cross between the shaft and the Tiber Creek Sewer, then turns back 90 degrees and crosses above the Tiber Creek Sewer. It is likely on the order of three feet wide by five feet tall and constructed of normal strength reinforced concrete. We modeled the steam tunnel with inside dimensions of 3 feet by 5 feet, and 1-foot thick walls constructed of 4,000 psi concrete.

In addition, a portion of the steam tunnel will be backfilled with 1,000 psi grout prior to the installation of the sheeting. In the model, 1,000 psi grout was included for all elements within three feet of the launch shaft.

The launch shaft has been designed using SCZ23 sheet piles driven to a tip elevation of –52 and supported by five levels of walers with corner bracing. The wales are located at elevations +12, +4, –4, –11, and –20. The base of the excavation is at elevation –22. The base of the launch shaft included a 1-foot thick slab of 4,000 psi concrete. After the base slab is placed and allowed to cure, the lowest level of bracing will be removed to allow for the launch of the microtunnel. In addition, a concrete reaction block measuring 16 feet high × 12 feet wide × 2 feet thick will be placed to spread the jacking load out to a larger portion of the sheeting.

The construction sequence and application of the jacking load were carried out in a series of 14 stages as follows:

- **Stage 1—In-situ (pre-1700s) State:** The in-situ stresses were established assuming that the current strata boundary between the Fill and the underlying Alluvium was the ground surface.
- **Stage 2—Current State:** The Tiber Creek Sewer, steam tunnel, and all fill are added in this stage.

- **Stage 3—First Excavation Level:** The steam tunnel is filled with 1,000 psi grout near the excavation, the sheet piles are installed to full depth, and an excavation is made within the sheets to one foot below the first wale level. Displacements in the model were initialized to zero at the beginning of this stage and all subsequent displacement values in the model are cumulative and relative to this point.
- **Stages 4 through 8—Second through Sixth Excavation Levels:** In each of these stages, a single level of wale beams is added to the model, and the excavation proceeds to one foot below the next waler level. No pre-stress load is added to the walers in the model.
- **Stage 9—Concrete Poured:** The base slab of the excavation and the reaction block for the jacking frame are added to the model. The base slab is one foot thick, and the reaction block is 16 feet high X 12 feet wide X 2 feet thick.
- **Stages 10 through 14—Microtunnel Jacking:** Jacking loads of 200, 400, 600, 800, and 1000 tons are applied sequentially as two point loads on the face of the reaction block to simulate the application of loading through the two relatively small-diameter jacks. These loads represent a range of values that the jacks can operate at, and were chosen to provide coverage of all possible jacking scenarios. By applying the loads as point loads on the face of the reaction block, the model can be used to confirm the adequacy of the reaction block to spread the loads over a larger portion of the shoring system.
- As shown in Figure 4 strains imposed on the Tiber Creek Sewer during excavation and jacking Stages 7 through 11 will be negligible as the modeling results show that the Tiber Creek Sewer Invert would deflect less than 0.1 inches during the proposed excavation and jacking operations. Due to the conservative assumptions we have made in the preparation of this model, it is likely that these values represent an upper bound on the expected deformation of the sewer. Due to the interaction between the entire shaft and surrounding

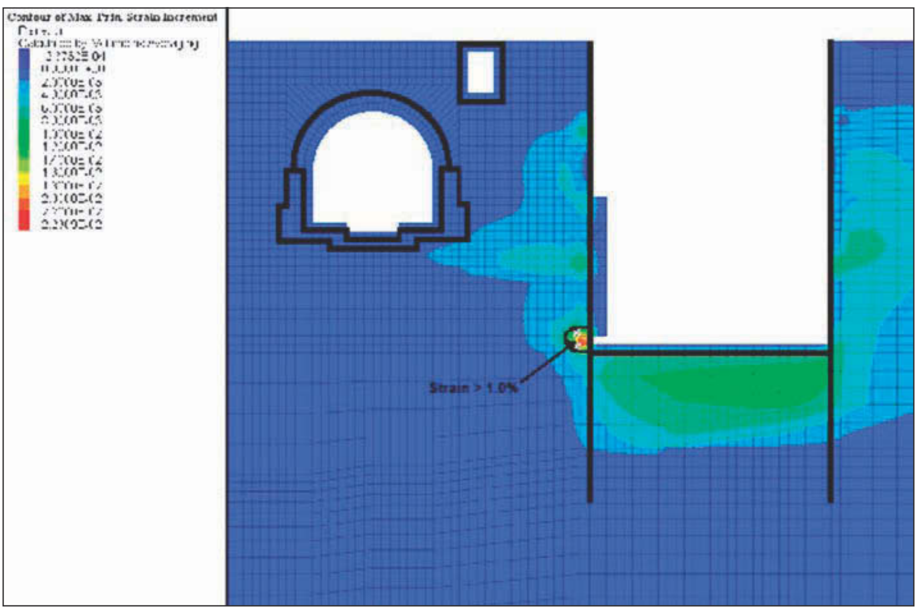

Figure 4. Maximum axial strains during Jacking Stage 11

soil in conjunction with the relatively deep application of jacking loads, the model indicates that a majority of the movement in the soil will occur at depths deeper than the Tiber Creek Sewer.

In addition to having a very small magnitude, the displacement of the sewer will be spread over a wide area, causing a very small amount of distortion. For example, the maximum rotation of the sewer is about 0.003 degrees (under a 1,000 ton jacking load). Figure 5 shows a deflection profile along the invert of the sewer to illustrate this for Stages 9 through 14.

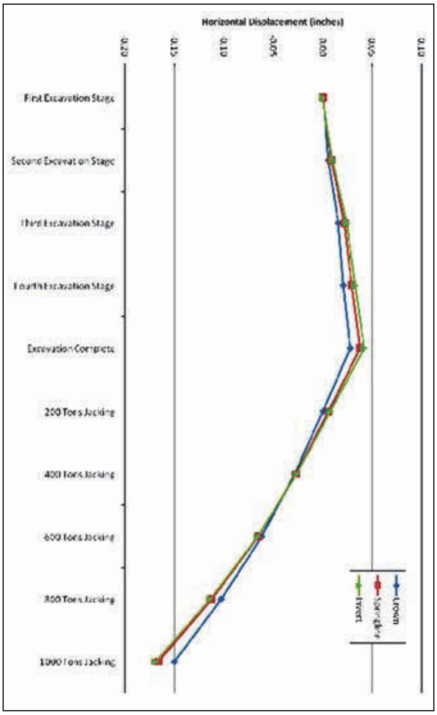

Figure 5. Predicted movement of Tiber Creek sewer due to shaft construction and jacking loads

Based on our approach, we will limit the jacking load to approximately 500 tons. This thrust reduction can be achieved by using intermediate jacking stations (IJS) immediately behind the shield and along the tunnel alignment and using an automated lubrication system to reduce overall skin friction on the pipe.

EIS PERMEATION GROUTING

Permeation grouting with a sodium silicate grout was selected based on a risk/benefit analysis of different methods of providing ground improvement to protect the ESI. In making this decision we recognized that chemically grouting soil with 15 to 25 percent fines would be challenging. We consulted grouting specialist regarding this issue. The basic response was that where the grout does not take because of the higher percentage of fines is also the ground that has the greater probability of providing sufficient stand-up time to allow for the annular void outside of the jacked RCP to be adequately stabilized with the bentonite lubrication when tunneling and subsequently grouted with a cement grout at the completion of the tunneling. Settlement calculations were based on this expected reduction in volume of lost ground (V_l) to about ¼ percent. And where the permeation grouting does penetrate into the soil, the resulting stand-up time of the soil, which is mostly sand by definition, will be substantially increased, so its V_l will also be about ¼ percent as well.

This grouting would be performed from the street surface using the lance method of injecting grout starting at the lower elevation of the grout zone and injecting grout as the lance is withdrawn. Grout injection pressures will be limited to values only slightly in excess of the in-situ total load on the soil at that location to avoid causing damage to the existing interceptor by the grouting process.

We estimated that movement at the ground surface and at the ESI invert will be negligible. The surface settlement in the vicinity of this crossing under the ESI was estimated at about 0.03 inches. With the ground modification described above we estimated that the ground movement at the invert of the ESI will be .04 inches. The angular distortion to the ESI caused by this maximum settlement will be less than 0.05 percent.

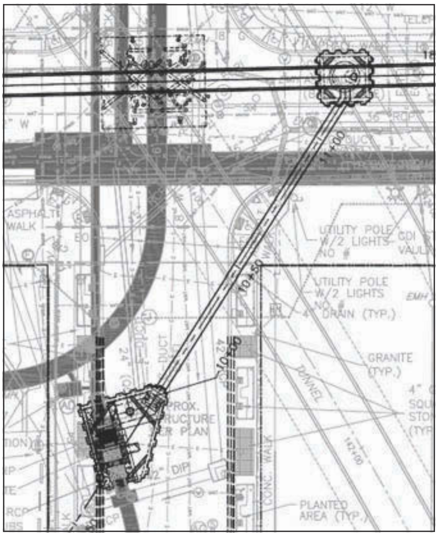

Figure 6. Ground modification at East Side Interceptor

WMATA CROSSING

The change in tunneling using the shaft in the vicinity of the Tiber Creek Sewer provides the benefit to the WMATA tunnels of a greatly mitigated risk of any movement to these tunnels. With this approach the Tingey Street Tunnel only passes through the CSO-013 MH structure, Figure 6, after the ground has been replaced with a weak cement grout, eliminates any detrimental effects of jacking loads on the tunnels, reduces the time that the excavation for 013 MH is open and the subsequent loading change on the under lying tunnels.

Conventional driver/extractors produce a spike in energy on startup and shutdown, which can result in vibrations that are as much as three times as strong as the vibrations experienced during driving.

To further mitigate any disturbance to the WMATA tunnels NRC will drive sheet piles to construct the temporary excavation support systems for the construction of the CSO-013 DS and the CSO-013 MH using a variable moment driver/extractor hammer. We undertook a vibration analysis to evaluate potential effect of the critical case of construction induced vibrations and distance between source and structure. The smallest separation between outside crown of the WMATA tunnel and toe of the sheeting is approximately 29 feet.

The largest variable moment driver/extractor produced by American Pile Driving is capable of delivering 4,500 inch-pounds of energy at a frequency of 2,300 cycles per minute (38 Hz) during pile driving. Based on this energy input and the distance to the WMATA tunnels, we calculated that the amplitude of the displacement at the WMATA tunnels will be less than 0.00021 inches and the peak particle velocity will be less than 0.05 inches per second. These values are well below any established threshold for damage.

WEAK SOILS

Contract documents required that ground modification via support of the tunnel be provided to address the issue of long term settlement potential of the soft soils at the east end of the project. Our proposal of a single row of 6-foot-diameter jet grout columns at approximately 10 feet center-to-center spacing along the centerline of the new sewer was accepted based on the predicted settlements. The columns will penetrate at least 2 feet into the underlying Potomac formation and will extend into the tunnel profile sufficiently so that the full width of the column will engage the underside of the new sewer as indicated in Figure 7. The estimated settlement of the jet grout column supported tunnel in this zone is expected to be less than ¼ inch. The criterion used for column placement in consideration of the utilities throughout this segment of the alignment was to allow for a maximum deviation of two feet from the design locations. At the terminus of the jet grout columns at the exit shaft we will install two columns to serve the dual purpose of tunnel support and also as a ground modification at tunnel break-in.

HISTORICAL STRUCTURE AND CONNECTION TO EXISTING ACTIVE CSO LINES

Figure 8 shows the proximity of the historical Building No. 74, existing utilities and jet grout columns that will be used for tunnel support as well as break-in ground modification support as well as support for the 60-inch connection sewer to the distribution structure. The same approach to the installation of the support of excavation using variable moment driver hammer will be used to reduce vibration for the support required at Building No. 74. The building will be monitored using both surface monitoring points on the sheeting and an inclinometer between the building and the sheeting.

The sequence of construction to tie the new diversion structure to the existing 75-inch sewer will require use of a flume rather than by-pass pumping because of the size of the sewer and the fact that it has a tidal flow in it from the river. The new diversion structure was also designed to avoid the existing piles that support a portion of the 75-inch sewer within the SOE footprint. Because of the grade change within the SOE we opted to support the new 60-inch sewer on jet grout columns rather than driving steel pipe piles for this structure. This decision allows for all piles to have the same top elevation.

CONCLUSIONS

The ability to work a design directly with a contractor where construction input with regards to equipment and materials available to the contractor can be used in the design is a major savings to the project. Also the design effort is more efficient when preferences for types of support can be discussed and opening evaluated. As was the case of type of pile support for the CSO014 Diversion Structure, the initial stated preference was a wood pile by the NRC and steel pipe pile by CDM Smith. With responses to their concerns the decision to use steel was made. At the same site an evaluation was made by NRC regarding the length of the steel sheets either with or without jet grout columns to shorten the length. The decision is still be evaluated.

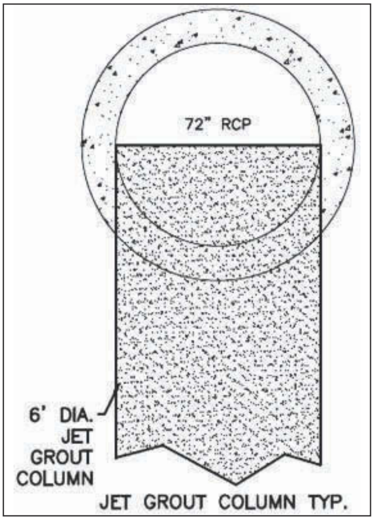

Figure 7. Tingey Street tunnel supported on jet grout columns in soft fill

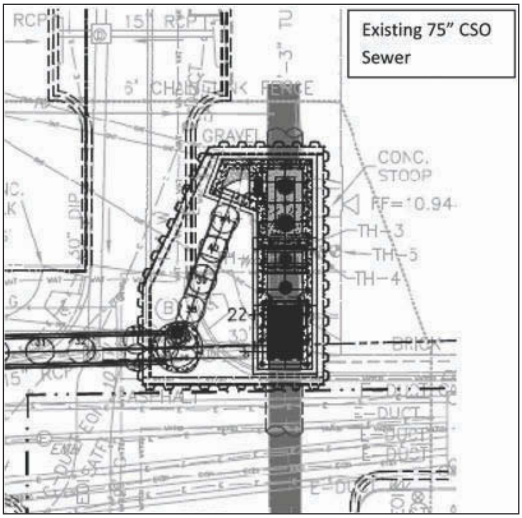

Figure 8. Exit shaft at historical Building No. 74

MICROTUNNELING IN GRAVEL, COBBLES, AND BOULDERS

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ABSTRACT

Perhaps the most challenging ground condition for microtunneling is a full face of wet, cohesionless, high permeability gravel with cobbles and boulders (GCB). This ground condition increases the risk of potential impacts such as: a jammed excavation chamber; high torque and microtunnel boring machine (MTBM) stalling; excessive overmining resulting in lost ground settlement damage or sinkholes; significant MTBM advance rate reductions; excessive abrasion damage to cutters, cutterhead, rock crusher, intake ports, slurry mucking system; and impact vibration damage to MTBM gears and bearings. The risk of these potential impacts may make microtunneling inadvisable, but at least necessitates use of special measures to help make microtunneling more manageable. This paper elaborates on the challenges of microtunneling in GCB and provides potential solutions to mitigate the most significant risks.

INTRODUCTION

Microtunneling in gravel with cobbles and boulders is significantly more challenging than common microtunneling. Previous papers by the authors and others have addressed the particular demands associated with microtunneling through boulders—see Hunt and Del Nero 2010 and Hunt and Del Nero 2012. The focus of this paper is specifically on microtunneling in a gravel matrix with or without cobbles and boulders.

Some tunnel engineers believe that microtunneling in gravel with cobbles and boulders is too risky and should be avoided. The authors believe that with special measures it is feasible, and that it may be the only practical solution in some situations. The special measures are needed to:

- Control the rate of flow of cut ground past the cutterhead into the crushing chamber relative to the advance rate to reduce risk of chamber choking, high torque and MTBM stalling.
- Maximize available MTBM torque for rotating the cutterhead and rock crushing.
- Provide a durable cutterhead, cutters and rock crusher that can adequately excavate, fracture and commutate cobbles and boulders into gravel size (<75 mm or 3 inches) for pumping.
- Provide a bentonite or a bentonite-polymer-additive slurry with properties to help provide face stability, prevent excessive slurry losses, control of muck flow into the crushing chamber; lubricate muck to minimize mixing and crushing friction and wear of rock crusher, intake ports, slurry lines and slurry pumps; and to stabilize the slurry to help maintain suspension of clasts in pumped slurry to prevent clogging and jammed slurry lines.

Control of flow of excavated material (muck) into the crushing chamber is essential for successful microtunneling in GCB. If the rate of MTBM advance is too high relative to the flow rate of GCB into the chamber, the rock crusher will not able to commutate clasts fast enough for pumping and the chamber will become jammed. As the chamber jams, MTBM torque will become excessive and eventually the MTBM will stall. If the rate of advance is too slow relative to the rate of flow of GCB into the chamber, over-mining will likely occur resulting in excess ground loss, settlements or sinkholes.

HIGHLIGHTED RISK AND POTENTIAL MITIGATIONS

Ground and Groundwater Conditions

Wet, cohesionless, high permeability gravel or GCB tends to cause several important geotechnical challenges. This ground condition has one of the lowest standup times possible. As the groundwater head increases, the standup time decreases and challenges of providing face stability increases. Unbalanced groundwater heads as small as 1 m (0.1 bar) can cause flowing ground with overmining and excessive settlements. Contractors often attempt to compensate for flowing ground by increasing MTBM thrust and advance rate. However, ingesting more ground with cobble size clasts than can be crushed and pumped for the advance rate may create a high commutation energy demand (see Hunt and Del Nero 2012) and excessive torque resulting in a stalled MTBM. Flowing ground and overmining must be prevented by reductions in the cutterhead opening ratio (COR) and by applying effective face pressure using an engineered slurry that forms a filter cake and application of slurry pressurize equal to groundwater pressure and active earth pressure.

To properly characterize GCB for determining MTBM components and operation, the subsurface investigation program should be designed to indicate:

- Percentages of ground types anticipated for the tunnel zone and stratigraphy including three dimensional extent of gravel and high permeability zones.
- Groundwater heads and ranges in porosity and permeability of all aquifers and gravel zones.
- Cobble and boulder conditions including: sizes, shapes, distributions, cobble volume ratio (CVR) and boulder volume ratio (BVR), rock unconfined compressive strengths, and abrasivity of rock clasts (e.g., Cerchar abrasivity index, CAI).
- Grain size distributions, abrasivity range, percentage of fines, cohesiveness or unconfined strength (extent of cementation, if any) and Atterberg Limits of tunnel zone matrix soil.

In order to successfully microtunnel in GCB, the subsurface investigation program must focus on obtaining reliable data for the properties listed above. A significant subsurface investigation challenge in GCB is sufficient recovery of gravel, cobbles and boulders to allow reasonable baselining. Special subsurface investigation methods are needed to supplement normal rotary wash, hollow stem auger and STP sampling methods. Those that have been found to be effective in ground with GCB include rotosonic borings, bucket augers, test pits, and caissons with windows (Hunt and Del Nero 2010, Hunt and Del Nero 2012).

Face Pressure and MTBM Muck Conveyance Slurry

Presuming that permeability and groundwater head are adequately known, the next challenge is to apply an effective face pressure that minimizes flowing ground and overmining. To be effective, the face pressure must be at least equal to the groundwater pressure at invert plus a component for active earth pressure. This is a very difficult task in very high permeability ground, which can generally be assumed to have a permeability of 10^{-2} cm/sec or more. Counterbalancing the water pressure can be readily achieved by pressurizing the excavation chamber to the required level regardless of the slurry mixture being used. Resisting flowing ground and overmining is more difficult.

To provide face stability, the muck conveyance slurry must have sufficient viscosity and other properties to form a “filter cake” at the heading. A thorough discussion of slurries for microtunneling and recommendations for slurry properties for various soil types are given in Boyce et al. 2011 and Camp et al. 2011. Kim and Tonon 2010 provide a detailed discussion of filter cake formation by slurries and face stability for various soil types including high permeability cohesionless soils. Fritz 2003 provides a thorough discussion of mix designs and additives needed for slurry shield tunneling in ground with high permeability.

Contractors like to use a water-soil slurry to reduce slurry costs and improve separation plant efficiency. Water-soil slurry may be suitable for microtunneling in clayey or silty ground, but it is not suitable in high permeability, cohesionless soils with less than 10 percent fines (<10 percent passing the no. 200 sieve). The muck conveyance slurry for microtunneling in a gravel matrix with less than 10 to 15 percent fines, should not be water-soil only slurry. If the slurry is too thin and a filter cake is not formed, the slurry will excessively flow into the ground and only provide resistance to flowing ground from seepage pressure. Adequate face pressure to resist ground flow will not develop and large volumes of slurry will be lost. A properly designed bentonite slurry or bentonite-polymer-additive slurry should be used when microtunneling in high permeability gravel or GCB to resist uncontrolled flowing ground and mitigate risk of overmining, a jammed excavation chamber and a stalled MTBM.

In addition to face control, use of bentonite or bentonite-polymer-additive slurry is also important for lubrication to reduce muck shearing and pumping friction and to reduce abrasion of the cutters, cutterhead, rock crusher, intake ports, slurry lines, pumps and separation plant. These factors are discussed below. Despite all these potential benefits, often too little attention is given to the muck conveyance slurry design to achieve successful microtunneling in a gravel matrix. While the slurry design is very important, another critical element is the MTBM cutterhead opening configuration and extent.

Cutterhead Opening Ratio

The cutterhead opening ratio (COR), which is the percentage of open area on the cutterhead, and size and distribution of openings from the center are critically important considerations for microtunneling in GCB. MTBMs typically have CORs ranging from 20 to over 50 percent and may be as high as 80 percent. Larger CORs are generally desired in cohesive soils (firm or slow raveling ground) to improve muck flow and help prevent clay clogging at the cutterhead opening. Smaller CORs are generally desired in cohesionless soils (flowing or fast raveling ground) to help restrict muck flow.

Where the ground has sufficiently low permeability, no active groundwater head and sufficient strength to be stable in an open face condition, a larger COR is also generally desired in GCB with a total clast volume ratio less than 2–3 percent (clast volume ratio is the total volume of cobbles and boulders as a percentage of excavated volume). Larger cutterhead openings increase the size of clasts that can be passed and minimize the amount of cobble and boulder fracturing required by cutters to be passable into the excavation chamber, which may or may not be desirable. While a larger COR helps reduce cutter and cutterhead wear and damage, it increases risk of a jammed MTBM and may increase rock crusher wear since more commutation energy must be expended to reduce rock clasts to gravel size for flow through intake ports and the slurry piping system. To determine the best COR for a project with variable ground with GCB, the contractor and MTBM manufacturer should determine:

- What COR is needed in combination with properly pressurized engineered muck conveyance slurry to provide adequate face stability (avoid flowing ground);
- Whether more cobble and boulder fracturing should be completed by the cutters or more by the rock crusher depending on cutter types, cutterhead opening sizes and available torque; and
- Impacts of the COR on advance rates and lost ground settlement risk for the entire tunnel drive(s).

In high-permeability GCB, a meter or more of groundwater head may result in potentially flowing or fast raveling ground. A smaller COR may be needed to help reduce the flow of ground into the MTBM chamber and thereby reduce the risk of torque overload, choking and stalling. The use of locally thicker bentonite slurry or bentonite slurry with polymer or fiber additives may be suitable for controlling the face in small pockets (mixed face or full face) of gravel or GCB, but it may not be sufficient to prevent the excavation chamber from getting jammed with excessive cobbles and boulders that must be crushed. Furthermore, use of a smaller COR helps reduce dependence on the slurry mix design and ability to rapidly adjust it in changing ground conditions.

Considerable energy is required to crush cobbles and boulders to gravel size for passage through intake ports, slurry lines, elbows, valves and pumps to a separation plant. MTBMs have limited power and torque for use in turning the cutterhead and in crushing rocks within the excavation chamber. In GCB, the volume of cobbles and boulders that enter the chamber should be limited to reduce the commutation energy demand and prevent excessive torque and stalling. To minimize the energy required for commutation, the bentonite slurry should be designed to have a lubrication benefit. In addition, the intake ports and slurry lines should be designed as large as possible to allow passage of larger clasts than normal to reduce commutation energy.

Based on experience from several projects, Hunt and Del Nero 2012 suggested that cutterhead opening ratio limits should be specified when clast volume ratios over 10 percent are expected. Where the anticipated clast volume ratio exceeds 10 percent, the COR should be reduced to 25 percent or less and may need to be in the range of 10 to 20 percent. Where the ground permeability is over 10^{-2} cm/sec, the COR should be lower and closer to 10 percent to help minimize the risk that bentonite slurry will be thick enough for face stability. Where the ground permeability is lower, the risk that the bentonite slurry viscosity will be inadequate is lower and the COR may be higher and closer to 20 percent.

Cutterhead opening size and configuration should be optimized for the size, distribution, and geometry of the clasts anticipated and the range of soil matrix conditions expected. For instance, if the clasts tend to be planar, then several smaller openings may not be the best geometry even though the cutterhead opening ratio is suitable. Several long openings at the face may still meet the requirement for a reduced COR, but may permit the passage of too many large clasts that may ultimately clog the crushing chamber and stall the drive.

While reduced CORs for microtunneling GCB have been found effective at reducing risk of overmining and a jammed excavation chamber and stalled MTBM, COR reductions will likely reduce MTBM advance rates. Reduced advance rates are the price that must be paid to successfully microtunnel in GCB. However, reduced advance rates may improve MTBM cutter ability to fracture boulders at the heading while reducing cutter impact damage and wear (Hunt and Del Nero 2012). Reduced impact vibrations and torque spikes also help reduce risk of MTBM cutterhead bearing and gear damage.

MTBM Torque

The thrust and torque required for an MTBM to effectively advance through GCB is dependent on many factors including: soil density; gravel content, clast volume ratios; clast sizes and strengths; energy required to fracture, pluck and crush clasts; muck flow friction in the MTBM chamber, intakes and slurry mucking system; and friction between the ground and MTBM and jacked pipe. GCB with higher density or that is weakly cemented tends to increase the torque required to cut, pluck and pass cobbles and boulders. As the clast volume ratio increases, the commutation energy and MTBM torque demand increase. In addition to clast volume ratios, the size and unconfined compressive strength of the clasts also influences torque demand (Hunt and Del Nero 2010). A boulder will generally require more torque to cut and pluck than scattered cobbles for the same clast volume ratio. Torque spikes above that required for general excavation will result when the cutters impact boulders at the face. The sustained energy and torque required to cut and fracture or pluck clasts at the heading increases as the unconfined compressive strength of the rock increases (Hunt and Del Nero 2012).

After cobbles and boulders are partially cut, plucked and passed into the MTBM excavation chamber, the energy required to crush the clasts to a gravel size for slurry mucking is very high and increases with both increasing clast volume ratio and increasing unconfined compressive strength of the clasts. Torque spikes are also likely to occur when one or more, large, high strength clasts are engaged by the rock crusher.

When microtunneling in GCB with a clast volume ratio greater than 10 percent, the selected MTBM should be provided with the highest torque available from manufacturers for the excavated diameter and operating cutterhead speed. Specifying or at least strongly suggesting use of the maximum available torque for the planned excavated diameter is strongly recommended to help reduce the risk of stalling in this ground condition. MTBM shield skin-up in GCB should be avoided or only done with caution and after careful assessment of commutation energy need and available torque. Depending on the percent increase in diameter and the MTBM drive system, excessive skin-up most likely means more ground will be mined than the machine was designed for from a torque standpoint. Specifying a limit on how much skin-up, if any, of a MTBM will be allowed may be prudent for tunneling in GCB. The issue of skin-up is addressed in the Alameda Siphon No. 4 case history later in the paper.

Friction and Lubrication

The friction of the slurry and muck rotating through the MTBM crusher and flowing to intake ports is much higher than normal when boring in GCB with few fines. Friction increases as the gravel content and clast volume ratio increase. Friction also increases as the abrasiveness of the clasts and matrix increase. Higher friction and resistance to muck flow results in higher torque demand. Use of an appropriately designed bentonite or bentonite-polymer-additive slurry helps lubricate the muck and to reduce the muck resistance to flow and thereby torque demand. In addition to reducing friction, the bentonite also helps to reduce abrasive wear of the rock crusher, intakes and slurry mucking system (Milligan 2000). If ground conditions are such that GCB can be broken down outside of the cutterhead, that may also allow a reduction in torque as the clasts actually ingested would likely be smaller in size and therefore require lower commutation energy.

Abrasion and Wear

Another potential consequence of using water-soil only conveyance slurry in ground with a gravel matrix and cobbles and boulders is excessive abrasion, cutter impact damage and wear. GCB muck within water-soil slurry without bentonite is much more abrasive than in bentonite slurry. A more abrasive slurry results in higher MTBM torque and higher rates of wear of the rock crusher bars and arms, the chamber slurry intake ports, the slurry pump and slurry return lines, particularly at pipe bends (Milligan 2000). Severe intake port wear from crushed GCB may cause MTBM slurry lines to become jammed and advance stopped (Camp 2007b and Staheli et al. 1999).

When clast volume ratios are in the range of approximately 3 to 10 percent within a gravel matrix, measures such as use of engineered bentonite-polymer-additive slurry and cutterhead opening ratio reduction are likely needed. When clast volume ratios exceed 10 percent and very abrasive gravel and clasts are expected (Cerchar Abrasivity Index, CAI > 2 or 3), microtunneling should be avoided unless special measures are provided to manage the abrasion and stalling risks. These measures might include: a combination head with disk cutters, reduced COR, cutterhead and crusher armoring, use of bentonite-polymer-additive slurry; use of larger intake ports and slurry lines; use of intake port surface hardening and specified minimum TBM cutterhead torque requirements.

SELECTED CASE HISTORIES

Woods Trunk Sewer Replacement Project; Portland, Oregon

The Woods Trunk Sewer Replacement Project in Portland, Oregon is an example of repeated MTBM stalling followed by successful microtunneling after MTBM and conveyance slurry modifications. The case involves a 249 m (817 ft) long drive using a 741 mm (54-inch) diameter Soltau RVS 600 MTBM to jack 914 mm (36-inch) ID reinforced concrete pipe (Hickey and Staheli 2007). The initial ~58 m (~190 ft) of drive from launch was described as “very dense poorly graded gravel [GP] in a silty sand matrix with [cobbles] and boulders up to 24-inch [600 mm].” The groundwater head was not reported, but is estimated at ~6 m (20 ft) based on the geologic profile (Staheli 2008).

At launch, the MTBM had a mixed-face cutterhead with triple kerf disc cutters and scrapers. The COR was approximately 20 to 25 percent. The MTBM bentonite slurry properties at launch were not reported, but characterized as “thin” or watery with little viscosity. Approximately 15 m (50 ft) into the drive, the MTBM stalled and upon cutterhead exposure, no boulder obstructions were found. The MTBM chamber was described as “packed with gravel”—see Figure 1 Left. The jammed gravel and cobbles were removed from the chamber, the MTBM head was reburied and tunneling resumed. After little additional advance, the cutterhead torque became excessive again and the MTBM stalled a second time. As before, no obstructions were found, but the chamber was packed with gravel and cobbles. This time the MTBM was removed for modifications.

MTBM cutterhead was modified to reduce the opening sizes and COR. Steel plates with hard facing were welded to reduce the size of the three largest cutterhead openings. The modified cutterhead had a COR of approximately 10 to 12 percent—see Figure 1 Right.

After modifications and servicing, the MTBM was buried again and relaunched. In order to improve face stability and minimize overmining, “the amount of bentonite in the slurry was increased.” These modifications “proved to solve the problem and the microtunnel [drive] was successfully completed” (Hickey and Staheli 2007). In summary, this case history shows that microtunneling in gravel with cobbles requires a smaller than normal cutterhead opening ratio and use of a thicker than normal bentonite conveyance slurry.

Alameda Siphon No. 4; Sunol, California

The Alameda Siphon No. 4 Project in Sunol, California is an example of MTBM stalling, and gearbox failure followed by successful microtunneling after MTBM and conveyance

Figure 1. Woods Trunk Sewer Replacement Project. Left: Crushing chamber packed with gravel after MTBM stalled. Right: MTBM cutterhead after modifications including closure pieces to reduce COR. (Staheli 2008)

slurry modifications. The microtunnel drive involved jacking ~178 m (584 ft) of 2400 mm (94.5 inch) inside diameter, 2438 mm (96-inch) outside diameter Permalok steel casing within alluvial sand and gravel below a predominately dry creek bed.

The GBR indicated that the tunnel zone "predominantly consisted of layers of poorly graded to well-graded, medium dense to very dense stream gravels containing sub-angular to rounded gravel, cobbles and boulders, with varying amounts of sand, silt and clay." The geological profile in GBR indicated that the tunnel zone soil was predominately within soils with a USCS symbol of "GP-GC." The GBR did not specifically baseline cobble and boulder quantities, but it indicated that "A substantial number of cobbles and boulders were encountered" and "Therefore, during the microtunnel construction, dense to very dense gravelly, clayey materials containing cobbles and boulders, with boulders up to 3 feet [~1 m] in size, should be expected."

In preparation for a Disputes Review Board (DRB) hearing, a study was made of Geotechnical Data Report Standard Penetration Test N-value data and sample descriptions along the tunnel alignment (Hunt 2011). In addition the alignment data was compared to N-values and reported cobble and boulder volume ratios within an adjacent, long, deep test trench in the same alluvial unit. The test trench data showed that alluvial gravel deposits contained cobbles and small boulders (up to ~500 mm or 18-inches in size) with total cobble + boulder volume ratios ranging from 5 to 40 percent. Based on all the available data at the time of bidding, a reasonable interpretation of the data indicated that bidders should have expected a total average cobble and boulder ratio of ~20 percent with a CVR of ~16 percent and BVR of ~4 percent. In addition, local zones had an expectable total cobble and boulder volume ratio of ~40 percent with a CVR of ~32 percent and BVR of ~8 percent (Hunt 2011). These expectable cobble and boulder quantities were very high for both the average and locally concentrated zones and indicated high risk of MTBM problems with chocking and cutter wear and damage.

The contractor elected to utilize an Akkerman SL-74 MTBM having a ~1880 mm (74-inch) diameter shield and upsize it to 2438 mm (96-inch) outside diameter. The upsized cutterhead had a COR of ~20 percent. The cutterhead had diametrical row with 8 double kerf disc cutters and had 4 triple kerf disc cutters at gage positions. Scraper cutters lined the cutterhead openings—see Figure 2 Left. The upsizing reduced the available torque to diameter ratio meaning that the upsized MTBM had less available torque than would generally be available with a non-upsized MTBM of the same diameter.

The Contractor elected to launch and operate the MTBM using a water-soil only conveyance slurry—no bentonite was added. During the first 75 m (254 ft) of tunneling, the recorded cutter head torque exceeded 75 percent of the maximum continuous available torque 20 percent of the time (Abbott 2011). At this point the torque became excessive and forward motion and ground excavation of the MTBM stopped. During the next 4 days, site reports indicate that the contractor made repeated attempts to reestablish cutterhead rotation. After approximately 40 hours of total effort (2, 16, 16 and 6 hrs) without being able to either fully rotate the cutter head or make forward progress, a decision was made to sink a rescue shaft.

After excavating to expose the cutterhead, the MTBM crushing chamber was found jam packed with cobbles and gravel with varying amounts of sand, silt, and clay—see Figure 3. Field engineers estimated that the soils within the tunnel horizon at the heading contained approximately 5 to 10 percent boulders by volume with sizes up to 460 mm (18 inches), 10 to 25 percent cobbles by volume, and that the remainder was a clayey sand and gravel matrix. Four representative cobble samples were selected and transported to a laboratory for strength testing. Laboratory unconfined compressive strength (UCS) test results indicated that UCS values ranged from 40 to 160 MPa (5,880 to 23,150 psi) with an average value of 83 MPa (12,000 psi).

The MTBM cutterhead was found to be in good condition with no broken cutters and minimal cutter and cutterhead wear—see Figure 2 Center. From the photographs and observations in the field, the wear to the crushing chamber was also quite minimal.

The cutter head drive gearbox was removed and disassembled. It was found to be very extensively damaged. The gearbox was sent to a structural laboratory for further investigation. The investigating specialist found that the gears and gearbox had little wear but the cast iron planetary carrier casting had failed and cracked at multiple locations (Abbott 2011).

Before tunneling could resume, MTBM repairs and modifications were made. The gearbox was replaced and the MTBM cutterhead opening ratio was reduced by 30 percent from an initial COR of 20 percent to a reduced COR of 14 percent—see Figure 2 Right.

Modifications to MTBM operation were also made upon relaunch for the completion drive of 85.5 m (~287 ft). Bentonite was added to the slurry system to provide thicker, more viscous slurry to increase face pressure effectiveness, reduce abrasion and reduce muck conveyance blockages. In addition, the MTBM cutterhead rotation speed was reduced from a previous average of 7.3 rpm to 4.3 rpm. This helped increase torque available for crushing rock clasts. The MTBM instantaneous advance rate prior to the stoppage averaged 13.2 mm/min. This rate reduced slightly to 11.7 mm/min. on

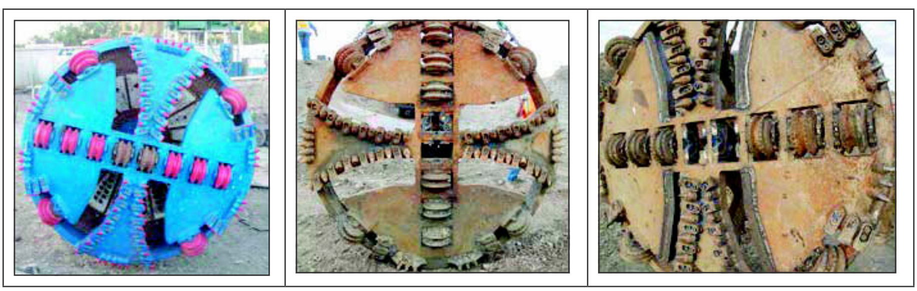

Figure 2. Alameda 4 Siphon. Left: Cutterhead before initial launch. Center: Cutterhead after removal from rescue shaft. Right: MTBM cutterhead after modifications before relaunch. (Finney and Del Nero 2010)

Figure 3. Alameda 4 Siphon. Left: MTBM crusher chamber with cobbles and boulders in sand and gravel matrix. Right: MTBM crusher chamber with cobbles in sand and gravel matrix. (Finney and Del Nero 2010)

the completion drive. The reduced advance rate was appropriate for the dense, cobbly ground being encountered to help reduce the high levels of torque. Prior to the stoppage, the recorded cutter head torque exceeded 75 percent of the maximum continuous available torque 20 percent of the time. For the completion drive, this reduced to 10 percent of the time (Abbott 2011).

The MTBM cutterhead and operation modifications were successful and allowed microtunneling to be completed in very difficult, dense gravel with high cobble and boulder volume ratios. The success on the completion drive was attributed to the reduced cutterhead opening ratio, use of bentonite slurry, reduced cutterhead rotation speed and slightly slower advance rates. These changes kept the crushing chamber from being jammed and the MTBM from stalling. The DRB evaluation team also concluded that upsizing should not be allowed or stringent limits specified in similar GCB ground in order to maximize the available MTBM torque.

International Paper Springfield Intake Facility; Springfield, Oregon

The Springfield Intake Facility Project in Springfield, Oregon is an example of MTBM stalling in gravel and cobbles due to a large cutterhead opening ratio and use of water-soil only muck conveyance slurry without bentonite. After the MTBM was recovered, this microtunneling drive was never completed.

Microtunneling was selected as the trenchless method to install 168 m (550 ft) of 1524 mm (60-inch) outside diameter, 12.7 mm (0.5 inch) wall Permalok steel casing for a water pipeline from a river intake to a paper mill in Oregon (Del Nero 2008). The project was completed to 30 percent design and was originally intended to have final design completed before bidding, but instead was bid as design-build. The proposed final design subsurface investigation program and preparation of a GDR and GBR were not completed and specifications for tunneling were not written.

The planning phase geotechnical report described the tunnel zone ground as dense, cohesionless sandy gravel with cobbles up to 150 mm (6 inch) size with potentially larger cobbles. The alignment extended from the bank of a river under a divided highway and below a slough resulting in a shallow water table and groundwater heads ranging from 2 to 5 m (7 to 15 ft). The design-build contractor elected to not complete any additional borings or install piezometers. The owner decided to forgo most of the “design” and submittal review part of the design-build process. The contractor did have access to outcrops of alluvial deposits within the stream bank and was able to observe the ground conditions excavated at the launch shaft and reception pit. Figure 4 left shows a stockpile of mostly gravel and cobbles excavated from the launch shaft.

Figure 4. Left: Springfield Intake Facility. Gravel and cobbles excavated from the launch shaft. Right: Gravel and cobbles excavated from the rescue shaft. (Del Nero 2008)

The contractor mobilized a 1524 mm (60-inch) diameter Soltau RVS 600AS MTBM. The MTBM had a large cutterhead opening ratio of ~35 to 40 percent. It had 5 button bit strawberry cutters and 3 multi-kerf disc cutters—Figure 5 Left.

The Contractor elected to launch and operate the MTBM using a water-soil only conveyance slurry–no bentonite was added. The MTBM had progressed approximately 15 m (50 ft) from the pump station launch shaft towards a reception pit at the river intake structure before cutterhead torque became excessive and the MTBM stalled. The contractor discontinued tunneling operations and constructed a rescue shaft to retrieve the machine.

The ground encountered in the rescue shaft was cobbly gravel with some sand. A geotechnical engineer estimated that total cobble volume ratio ranged from about 20 to 50 percent. Approximately 95 percent of the cobbles ranged from 75 to 150 mm (3 to 6-inches) in diameter, with an estimated 5 percent of the cobbles between 150 to 300 mm (6 to 12 inches) in diameter.

The MTBM crushing chamber was found packed full of sand, gravel and cobbles—Figure 5 Right. After further assessment of the ground conditions, the contractor and their consultant claimed differing site conditions (DSC) and indicated that Soltau RVS600AS was not capable of microtunneling in this ground and furthermore that the ground was not suitable for microtunneling by any MTBM. The contract was terminated and the drive never completed. The owner concluded that the risk of continued microtunneling challenges was too high even with recommended mitigation measures.

Engineers retained by the owner determined that a DSC did not exist based on a comparison of the ground encountered with those indicated in the planning phase borings. They also concluded that the drive failed because the MTBM cutterhead opening ratio was too large for the GCB ground encountered and because the water-soil only slurry used was ineffective at forming a filter cake at the face to restrict the flow of cut ground into the MTBM crushing chamber. The owner's engineers also concluded that microtunneling was viable with the right MTBM configuration, use of an engineered bentonite slurry and proper MTBM operation.

Folsom East Interceptor IIB

The Folsom East Interceptor IIB near Sacramento, California was completed in extreme GCB conditions using an open-face rotary wheel TBM with ribs and lagging (Castro et al. 2001) following an unsuccessful attempt to prove the viability of microtunneling during a "Construction Methods Microtunneling Proving Project" (Staheli et al. 1999). During the proving project, a conventional slurry shield MTBM with a high

Figure 5. Springfield Intake Facility. Left: Soltau RVS 600AS MTBM prior to launch. Right: MTBM after recovery from rescue shaft. (Del Nero 2008)

cutterhead opening ratio and scraper only cutters was unsuccessful. A hybrid rotating cutter arm MTBM also failed. Both machines were unable to complete their test drives. The researchers concluded that microtunneling was not viable in this ground. While their conclusions were valid for the types of MTBMs tested, they are not necessarily valid for the more robust MTBMs available today that have mixed-face cutters, small CORs, wear protection, high torque and more.

Snohomish River Crossing

The Snohomish River Crossing for the Clearview project near Seattle Washington is example of both failed and successful microtunneling in GCB. The crossing involved a 340 m (1,115 ft) long drive to jack 1524 mm (60-inch) Permalok steel casing below the Snohomish River, mostly within Old Alluvium. The GBR described the Old Alluvium as "saturated, mostly clean, oxidized, medium- to coarse-grained sand, gravel, cobbles, and boulders with many zones of open graded gravel and cobbles." The cobble and boulder quantities were not baselined.

Performance specs allowed the contractor to select a 1575 mm (62-inch) diameter Iseki Unclemole for MTBM (Figure 6 Left). The MTBM had four cutter arms fitted with scraper cutters and a very large COR of ~80 percent. Soon after launch, abrasive ground with cobbles and boulders impacted performance. Between 142–169 m (465–555 ft) the ground was 65–75 percent GCB with clast volume ratios as high as 20 percent. Between 169–177 m (555–582 ft), the ground was over 90 percent GCB with an estimated 30 percent clast volume ratio. The MTBM experienced extremely high, erratic torque along with steering problems and refusal to advance. The MTBM and 171 m (560 ft) of installed steel pipe were permanently abandoned.

After an approximately 11 month delay, a new Lovat MTS 2000 MTBM (Figure 6 Center) was launched from a new launch shaft located about 6 m (20 ft) offset to the East in the same geology. The new MTBM had higher torque, a much lower COR of ~25 percent and was fitted with multi-kerf disc cutters and wear resistant scrapers. The new MTBM with careful control of bentonite slurry properties successfully completed the drive in approximately 14 days with an average advance rate 22 m/d (75 ft/d). As reported by Staheli and Duyvestyn 2003, "The machine had little to no difficulty excavating the materials it encountered." The new MTBM cutters and cutterhead experienced very little wear—Figure 6 Right.

This case history shows how a robust MTBM fitted with disc cutters and a reduced COR with proper control of bentonite slurry properties can be successful where a MTBM without these features was a failure.

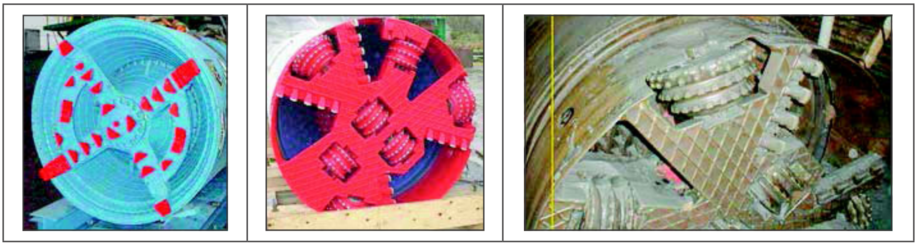

Figure 6. Left: Snohomish River Crossing. Iseki Unclemole before launch. Center: Lovat MTS 2000 before launch. Right: Lovat MTS 2000 after breakthrough with minimal cutter and cutterhead wear.

Other Case Histories

Many additional microtunneling case histories within high permeability gravel or GCB exist. Hunt and Del Nero 2012 provide an extensive bibliography for microtunneling and tunneling in cobbly and bouldery ground—many with gravel matrix. New references are published each year. Some of the many additional case histories on microtunneling in GCB worth noting include several projects in San Diego—see Camp and Murray 2005, Camp 2007a, and Camp 2007b.

Many microtunnels in Ireland have been completed in GCB. Curran et al. 2010 provide a summary of many of them and emphasize the importance of properly designed bentonite slurry for filter cake formation and pressure application along with use of a robust MTBM. Reilly and Orr 2011 provide case history overviews of three of these Irish MTBM projects.

Additional case histories for microtunneling in GCB can be found for Portland, Oregon; Sacramento, the San Francisco Bay area, and Los Angeles areas in California; Las Vegas, Nevada; and the Denver, Colorado area.

SUMMARY AND CONCLUSIONS

The risks of microtunneling in gravel with cobbles and boulders are significant but can be appropriately managed by special measures. In particular, special measures are critical when the gravel zone permeability exceeds 10^{-2} cm/sec or when the total clast volume ratio exceeds 10 percent. To microtunnel in high-permeability GCB and to reduce risk of choking and stalling along with risk of severe overmining and sinkholes, the flow of ground through the cutterhead into the excavation chamber must be restricted to a rate that the rock crusher and slurry mucking system can handle by one or more of the following methods:

- Pre-excavation grouting to reduce permeability and increase matrix strength.
- Use of an engineered bentonite or bentonite-polymer-additive slurry that is thick enough to form a “filter cake” and function as an effective lubricant, which helps to minimize clast separation in slurry lines.
- Application of a slurry pressure at the face equal to at least the groundwater pressure and estimated active earth pressure.
- Reduction of cutterhead opening sizes and cutterhead opening ratio with particular attention given to the orientation and sizing of the individual openings.
- Use of robust disc and scraper cutters with hardened inserts and wear protection on the cutterhead, rock crusher and slurry intake ports.

- Avoidance of advancing the MTBM too fast or over rotating the cutterhead. Allow the cutters to do their work in reducing the clast size prior to entry into the crushing chamber.
- Avoidance of high pressure water jets to loosen muck in the chamber. It only acts to dilute the slurry and remove the filter cake from the formation.
- Utilization of an MTBM that is not upsized and has the maximum torque available from manufacturers for that diameter.

These and other special measures discussed above should help make microtunneling in GCB more manageable and minimize the risk of MTBM stalling, getting stuck or encountering other severe impacts. Even with the best of mitigation measure, this ground condition should be considered one of the most challenging an MTBM could ever face.

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HIGHLY SUCCESSFUL GROUND SUPPORT FOR HIGH COVER: A CASE STUDY OF THE WEST QINLING RAIL TUNNELS

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ABSTRACT

China's West Qinling Rail Tunnels, located in the remote Qilian Mountains, are being excavated under high cover of at least 1,000 m for the entire length of the bored tunnel drives. To combat difficult ground conditions including limestone and phyllite with fault zones, a unique system of ground support has been designed. Two 10.2 m diameter Main Beam TBMs were engineered around the concept of these versatile support systems, which allow a variety of types of support to be installed in varying conditions. The design of the system and performance will be discussed, along with an analysis of the machines' world record advance rates and lessons learned. Applications of this type of ground support system on future projects will also be covered.

INTRODUCTION

Ground support for open-type, hard rock TBMs is always of critical importance, but even more so in high cover tunnels at larger diameters. Tunnel projects underneath more than several hundred meters of rock demand versatile ground support that can handle a wider range of problematic conditions, from rock bursting to over-break, squeezing ground, and fault zones.

The L1 zone, which extends from the face to several meters back from the cutterhead and directly behind the cutterhead support, is a critical area for ground stabilization. Primary ground support should be installed while boring to avoid deterioration of the rock over time and distance from the face.

Crew members may apply, depending on the ground conditions, shotcrete, wire mesh, steel straps, ring beams, rock bolts, or combinations of these types of support in the L1 area. As ground pressure and diameter increases, however, new measures must be taken in order to ensure a stable tunnel. This was the challenge before engineers designing the two Main Beam (open-type) TBMs for China's West Qinling Rail Tunnels.

PROJECT BACKGROUND

At up to 1,400 m of cover and more than 10 hours from the nearest town, China's West Qinling Mountains are a remote location for a rail tunnel. The West Qinling tunnels are part of the Chinese Government's Lanzhou to Chongqing Railway, a massive 820 km long scheme that will link the capital of Gansu Province (Lanzhou) with southwestern Chongqing, a mega-city of over 35 million people. The new railway, at a cost of USD $11.3 billion, will shorten transport times from 17.5 hours to 6.5 hours and enable an annual freight capacity of 100 million metric tons. Trains will run on the double track lines at 160 km per hour, with a 50-train daily maximum. The entire railway is expected to open to traffic in 2014 (see Figure 1).

The West Qinling Rail Tunnels are being excavated through ground consisting of limestone and phyllite, some siliceous phyllite, and fault zones in breccias and clay. An

Figure 1. Lanzhou to Chongqing Railway (Courtesy of ENR.com)

initial geological study for the project identified three large faults ranging in width from 190 m to 310 m wide, consisting of fragmented limestone and sandstone with gravels and breccia. Some karstic features were also identified. Broken, fractured rock is also expected in a 915 m wide section of tunnel.

The two parallel 16.6 km long routes are just 40 m apart and are located approximately 1,000 m above sea level, about halfway up Qinling Mountain. The 18th Bureau of China Railway Tunnel Group (CRTG) is managing the Left Line Tunnel, while China Railway Construction Corporation (CRCC) is managing the Right Line Tunnel.

Detailed Geology

The main rock type encountered on this project is Phyllite. Phyllites are strongly foliated metamorphic rocks similar to slates but slightly coarser in texture. Phyllites have a shiny luster due to the presence of large amounts of fine flakes of Mica. They gradually pass into slates which are the first stage in the metamorphism of shale. There is little use for Phyllite, as it is too soft to use as crushed stone and it is too weak for structural use.

In addition to the problems associated with boring through Phyllites under higher cover, problems are often seen when using hard rock boring machines through Phyllites, particularly when the gripper pads are removed to reset the machine in preparation for the next boring stroke. Due to the pressures associated with applying and removing the grippers this can cause the ground to appear to unload. Unloading is the release of pressure due to the removal of an overburden. When the pressure is reduced rapidly, the rapid expansion of the rock causes high surface stress and spalling. As such, the TBM and ground support system must be designed with these potential problems in mind.

TBM DESIGN

Two 10.20 m diameter hard rock main beam type tunnel boring machines with back-loading 19-inch disc cutters are being utilized on the project. The TBMs, manufactured by The Robbins Company, have cutterheads with installed power of 3,960 kW. Each machine can produce a maximum cutterhead torque of 10,322 kNm and produce a thrust force of 21,148 kN. Power to the machines is supplied at 22kV (see Figures 2–3).

Boring of the tunnels commenced in June 2010 for the first machine and July for the second machine.

Figure 2. Shop assembly of the main beam TBMs

Figure 3. TBM launch

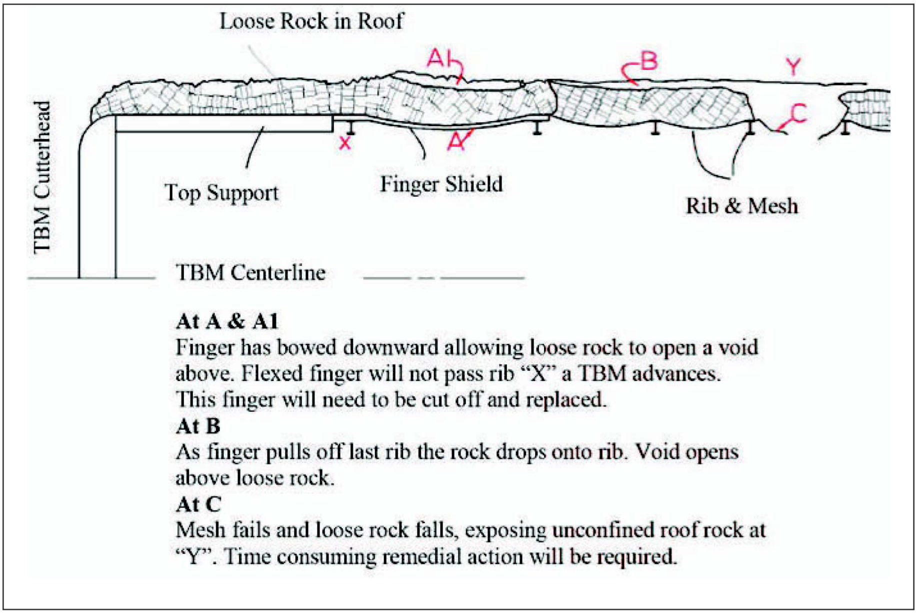

Figure 4. Loose rock behind roof shield fingers (Courtesy of C&M McNally)

GROUND SUPPORT SYSTEM DESIGN

The two TBMs are some of the more recent Main Beam machines supplied without roof support fingers. In the past, roof support fingers provided limited protection to the crew working at the front of the machine and also to a degree prevented damage to cutterhead drive motors and other equipment installed on the front of the machine. However, when poorer or blocky ground conditions were encountered these fingers would simply bend out of shape and often the contractor would end up removing them altogether (see Figure 4).

With the removal of the roof support fingers, the bored tunnel is exposed at the back of the roof support where more effective ground support is easier and quicker to install. The setup offers significant benefits:

a. Ground can be treated sooner as there is no need to wait for the bored tunnel to pass the roof support fingers to allow rock bolting or mesh installation, etc.

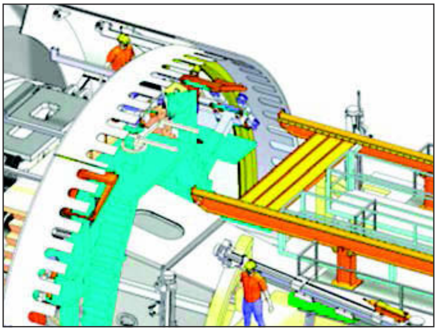

Figure 5. Drawing showing removal of shield fingers and protected area for installing ground support

Figure 6. Continuous pour concreting behind the TBMs

b. Bad ground can be held in the roof of the tunnel and does not (unless required by the contract) fall in to the invert creating a lot of cleanup operations.

Ground support systems on the West Qinling machines include modified mesh installation, ring beam installation, work platforms, and materials handling (see Figure 5). During tunneling, ground support consists of continuous mesh and rock bolts, with either ring beams or steel straps for the length of the tunnel. In addition the contractor opted to install a 50 mm thick wet mix shotcrete lining for the length of each tunnel in a zone some 50 m behind the primary (L1 zone) in what is known as the secondary or L2 ground support zone. The whole tunnel will eventually be lined with concrete to a thickness of some 300 mm (see Figure 6).

Safer Mesh Installation

Mesh windows, installed in the roof shield, allow workers to slide panels of mesh in the annular space between the shield and the tunnel crown. The panels are then pinned or secured with rock bolts. Traditional ground support includes no specific provisions for mesh installation and little cover from falling rock.

Streamlined Ring Beam Installation

Ring beams are installed using an erector consisting of the assembly ring and expander. The rotating assembly ring is fixed axially and used to loosely assemble five ring beam components. Once the components are loosely assembled and pinned to the assembly ring, the expander, which moves fore and aft, expands the components to a preset pressure against the tunnel wall. A sixth Dutchman piece is installed in the resulting space, and the ring beam with tightened connections is bolted to the tunnel wall (see Figure 7). The assembly and expander can also be converted to use steel straps, rather than full rings.

Previous assembly methods required that the fully assembled ring beam be transported to a pocket before being expanded against the tunnel wall. The method is not as fast, and does not give the flexibility often needed in changing ground that may require steel straps.

A Variety of Work Platforms

Accessible work platforms are located throughout the machine, including two in front of the ring beam erector and under the roof shield for mesh installation. Other work

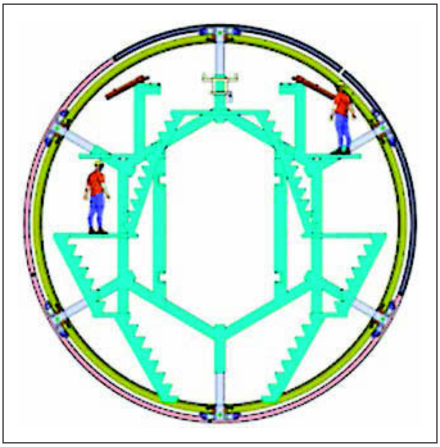

Figure 7. Ring beam erector at West Qinling

platforms are located at various levels around the circumference of the machine for ring beam and rock bolt installation.

Efficient Materials Handling

Streamlined materials handling allows the ring beam components to be transported efficiently to the L1 area. The system reduces the number of transfer points, and ultimately reduces the number of crew members required to transport materials.

Ground support components are loaded onto the back-up using a crane, and placed onto a carriage riding on an electric transport car. The carriage is designed to hold a stack of mesh panels, ring beams, rock bolts, and lagging materials for the McNally system, a specialized form of ground support. The remotely operated trolley carriage transports the materials to a rack located in front of the ring beam guide rollers where they can be easily placed.

Increased Range of Motion for Drills

Instead of a combination roof/probe drill, the setup utilizes separate roof and probe drill canopies. The system allows a wider range of motion and better access from work platforms.

Optional McNally Ground Support

If more difficult ground is encountered, the mesh pockets can be relatively easily converted to use a modified form of the McNally Ground Support System, developed by C&M McNally of Ontario, Canada for exclusive use on Robbins TBMs.

The McNally system requires modifying or replacing the roof shields. The curved finger shield plate is replaced by a curved assembly of pockets with rectangular cross-sections. The pockets extend axially aft from the rear side of the cutterhead through the cutterhead support, in the area where roof drills can work. Before a TBM stroke, crews slide pre-fabricated steel slats (consisting of four pieces of steel reinforcing bar welded together) into the pockets, such that the slats are two rows deep inside each pocket. The ends of the slats protrude from the pockets and are bolted to the roof of the tunnel using a steel strap. As the machine advances, the slats are pulled from the pockets and continuously bolted to the roof using subsequent straps. Slats are reloaded and used throughout excavation to prevent deformation and rock falls (see Figure 8.1–8.4,

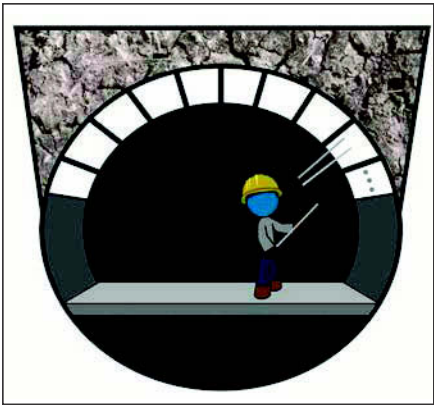

Figure 8.1. A worker loads slats into the McNally Pockets

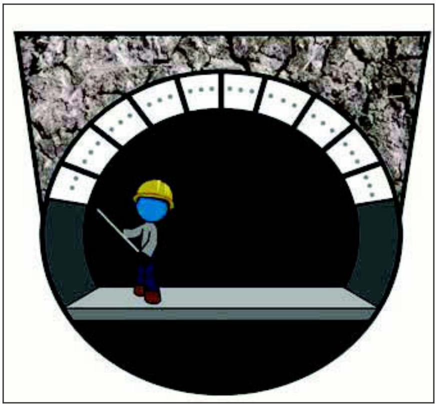

Figure 8.2. The slats are loaded around the tunnel circumference

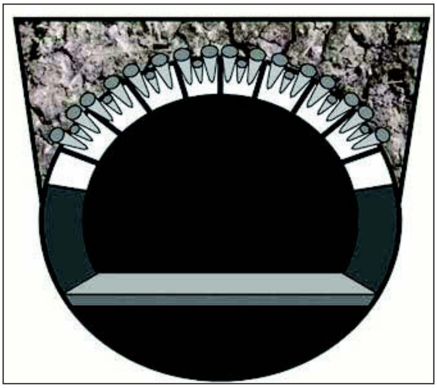

Figure 8.3. The slats are extruded as the TBM advances

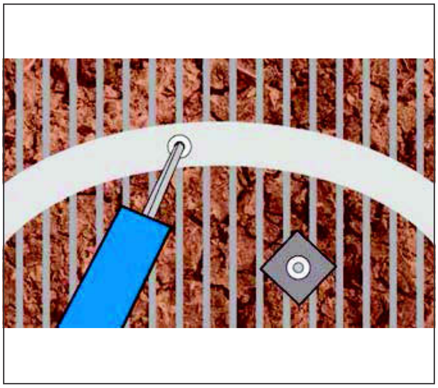

Figure 8.4. The slats are secured to the tunnel crown using rock bolts and straps

which shows a simplified cross section of a TBM and McNally pockets). Crews at West Qinling are able to bolt McNally pockets inside the mesh pocket structures, allowing a space to slide short slats of steel or wood into the area where roof drills can operate.

TUNNEL EXCAVATION

The machines were launched in June and July 2010. By October 2010, the TBMs were excavating at good production rates of up to 595 m per month. The high rates of advance continued into 2011, with a record month of 841.8 m being achieved during 30 days of continuous boring between March and April of that year. The record-breaking Left Line machine then broke through into an intermediate adit tunnel on May 28, 2011 at the 5.5 km mark, where it underwent planned maintenance and inspection.

The contractors at West Qinling have so far encountered mainly rock type III & IV ground conditions with a Rock Mass Rating of 41-60 & 21-40 respectively. Ground support consists of systematic bolting, mesh, ribs and ring beams as required. Temporary shotcreting is also done on an as needed basis before the final concrete lining is poured in the L2 zone (see Figures 9–10).

Though McNally slats have not yet been needed, a number of other factors are credited by the contractor as having a significant effect on the good advance rates. These include the robustness of the cutterhead and cutters, which have required minimal maintenance and repair, the continuous conveyor system operating behind the TBMs, and the efficient ring beam erector system. Since excavation began, small modifications have been made to the hydraulic rock drill, and a material crane was replaced with a lifter platform.

Into 2012, production rates have continued to be high for the Left Line machine, and average rates have increased year over year (see Figures 11–12).

Meanwhile, the Right Line machine also achieved high advance rates but was stopped in 2012 for refurbishing and replacement of the main bearing. This work was recently completed and the machine began excavating again as of December 2012. The advance rates reflect this stoppage (see Figures 13–14).

EXPANSION OF THE VERSATILE GROUND SUPPORT SYSTEM

With the success of the versatile ground support at West Qinling, as well as other projects such as Peru's Olmos Trans-Andean Tunnel (below 2,000 m of cover), Robbins has begun building nearly all of its Main Beam machines without roof shield fingers and with McNally Pockets.

Figure 9. Wire mesh in the tunnel crown

Figure 10. The ring beam erector expanding a ring against the tunnel wall

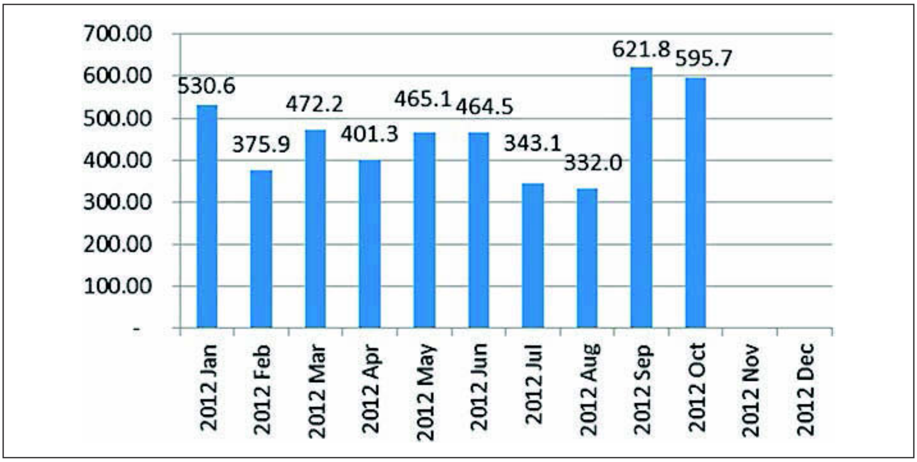

Figure 11. Monthly advance of Left Line machine (in meters)

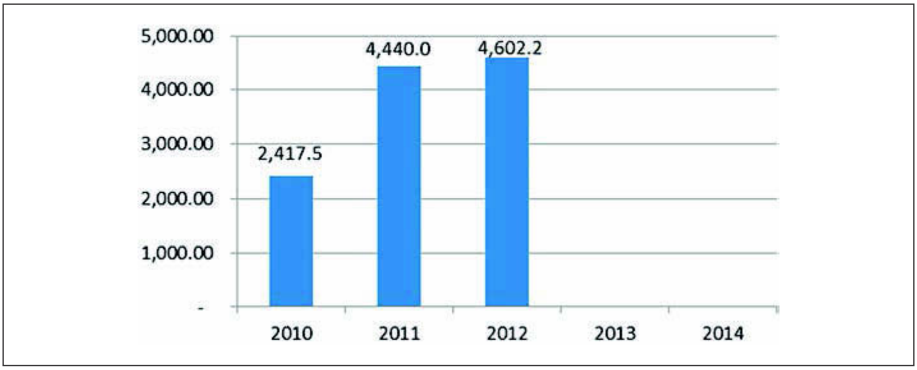

Figure 12. Averaged yearly advance rates of Left Line machine

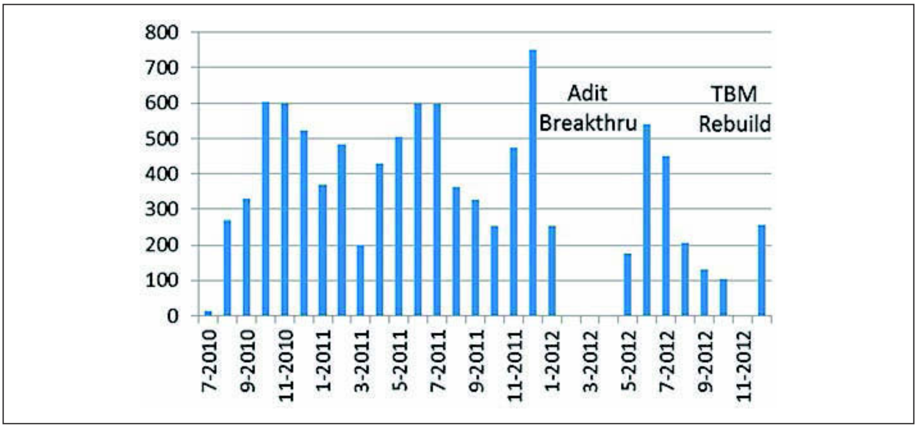

Figure 13. Monthly advance of the Right Line machine (in meters)

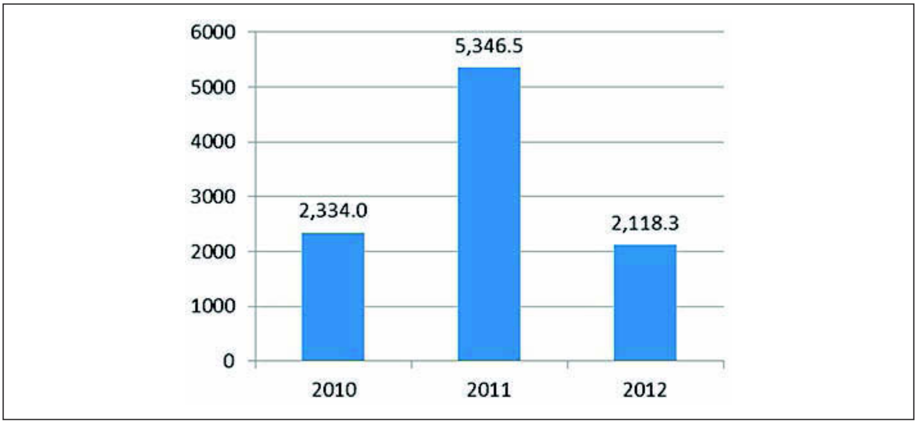

Figure 14. Averaged yearly advance rates of Right Line machine (in meters)

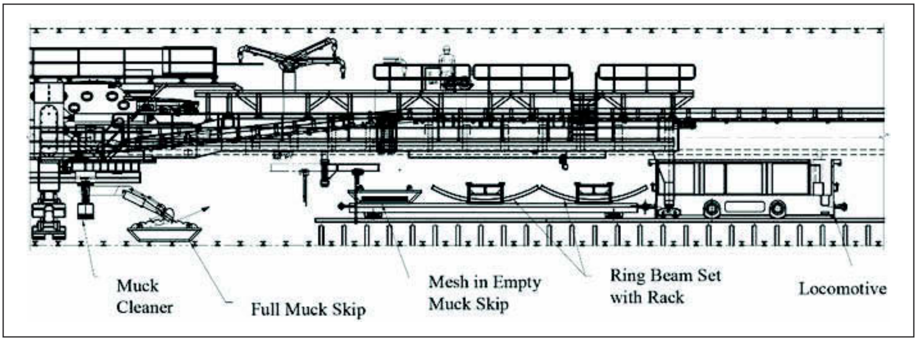

Figure 15. Invert materials handling

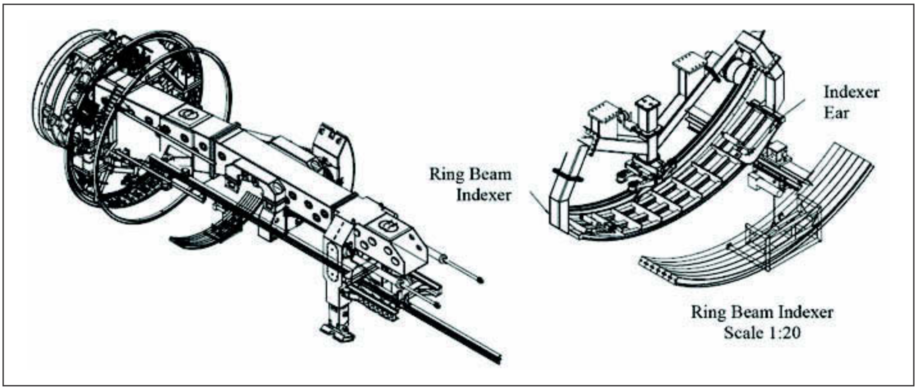

Figure 16. Ring beam erector and indexer system

New High Cover Project in China

A variation on this ground support system is now in development on 5 large diameter (8 m) Main Beam TBMs for a project in northeastern China. The ground support system will be similar to that at West Qinling, with several modifications due to the more limited space of an 8 m vs. 10.2 m diameter machine. Materials handling will take place in the tunnel invert, requiring a 180-degree rotating backhoe scoop that can be moved out of the path of the cart. A bridge crane and jib crane will pick up materials such as mesh panels, new disc cutters, etc. and transfer it to the bridge area. Invert cleaning will be ongoing when the cart is not in place.

The ring beam erector and roof drill system will both be mounted on the same rail system, but will be capable of independent movement. The ring beam erector will be similar to West Qinling, only the ring beam indexer—a loading tray—will be located at the bottom of the erector rather than at the top. Designed in six sections, the erector will pull one section out at a time to complete the entire ring from the tray.

The ground support system offers flexibility just like the West Qinling system, from McNally slats to ring beams and wire mesh, and will be provided on 5 machines (see Figures 15–16).

CONCLUSIONS

Versatile ground support systems allow Main Beam TBMs to better excavate challenging conditions, from squeezing ground to rock bursting. A more adaptive approach to ground support, which allows crews to place ring beams, rock bolts, wire mesh, ring beams, and McNally slats better arms the contractor for unexpected conditions. In projects where complex ground is predicted, these types of ground support systems have been proven successful. As such, we recommend that these types of systems are designed and installed on the TBM from the project launch, in order to avoid costly and time-consuming in-tunnel modifications.

LARGE-DIAMETER SHAFT CONSTRUCTION THROUGH DIFFICULT GROUND IN COLUMBUS, OHIO

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Mike Hall ▪ HR Gray

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Greg Fedner ▪ City of Columbus Department of Public Utilities

Bob Rautenberg ▪ Kenny Construction

Ron Hall ▪ Nicholson Construction

ABSTRACT

The 20 ft diameter OARS CSO Tunnel, currently under construction, will traverse 23,300 ft from the Arena District in downtown Columbus to the Jackson Pike Wastewater Treatment Plant (JPWWTP). The project includes 6 shafts up to 220 ft deep, as well as flow diversion structures, pump station, river overflow structure, screen building, and existing combined sewer relief structures and overflow connections. Challenging geotechnical conditions were encountered during pre-excavation grouting as well as shaft, starter tunnel, and tail /connector tunnel construction. The paper will discuss the issues encountered during pre-excavation grouting, slurry wall, cutter soil mixing (CSM), jet grouting and drill and blast excavation of Shafts 1, 2 and 6 while emphasizing lessons learned thus far on the project.

INTRODUCTION

The OARS (OSIS (Olentangy Scioto Interceptor Sewer) Augmentation Relief Sewer) came to be as a result of the City of Columbus's CSO Consent Order with the Ohio EPA. Identified in the 2005 Wet Weather Management Plan, OARS is a key component in the City's effort to control the combined sewer overflows (CSOs) along the Scioto River through downtown Columbus. The tunnel will serve two functions: (1) collect and store CSO's and prevent contamination of the Scioto River; and (2) convey flows to the Jackson Pike and Southerly Wastewater Treatment Plants. When operational the OARS Tunnel will prevent over 1 billion gallons of CSOs annually from entering the Scioto River.

DESIGN INTENT AND BID DOCUMENTS

The design of the OARS project focused on risk management relative to the mitigation of groundwater inflows that were anticipated to be encountered during construction. Shaft excavation involved soils ranging from non-cohesive and granular to cohesive. The depths of soil ranged from 40 feet at Shaft 3 to 100 feet at Shaft 1. In addition the static ground water elevations ranged from EL 670 feet to EL 700 feet, on average approximately 150 ft above the tunnel alignment, which made groundwater a major consideration during design development.

In order to excavate the tunnel in a consistent geology, the invert of the tunnel was designed to be approximately 180 ft below ground surface. The tunnel will flow by gravity from north to south and has a consistent slope of 0.13% for the entire 23,300 ft alignment. The shafts and tunnel were required to be constructed within bedrock consisting generally of shale, limestone and dolomite. While the shale bedrock is relatively impervious, the limestone and dolomite are characterized by solution features that transmit high groundwater flows. To mitigate groundwater inflows in the overburden, it was determined that a water-tight support of soil excavation would be required for all shafts drop structures. For rock excavations, a pre-excavation rock grouting program was required.

Geotechnical Baseline Report and Geotechnical Data Report

As geotechnical field and laboratory investigations were completed, a Geotechnical Data Report was compiled to document data obtained. Evaluations were completed to prepare a Hydrogeological Report utilizing field investigations that included slug tests, packer tests, and longer term pump well testing. In addition, a groundwater flow model was developed to estimate potential ground water inflows into the tunnel and shaft excavations. Because groundwater was expected to be a key design parameter and solution features were anticipated to be a major ground water contributor, down-hole videos were completed to further define/document solution features within the bedrock.

The Geotechnical Baseline Report included a description of the project, a summary of geologic and geotechnical information reviewed for the project, the geologic project setting, previous construction experiences in similar/local geology, ground characterization, subsurface conditions at construction sites, subsurface conditions and foundation recommendations for near surface structures, and construction considerations. Baselines were developed for several items using bid quantities. These items included modified grout for tunnel segment backfill, additional grout for shaft, tail tunnel, and screen chamber voids, downtime due to gas in tunnels and shafts, and cut-off grouting for interventions.

The Geotechnical Baseline Report was developed in concert with the project specifications to help define the grout requirements that would be required to manage groundwater into shaft and tunnel excavations. One major concern with the pre-excavation rock grouting plan was the potential that grout would migrate long distances from the zone required to cut-off groundwater. To address this concern a baseline void ratio was specified on the drawings for each shaft and drill and blast tunnels, adits, and deaeration chambers. The contractor was required to measure or quantify voids encountered during the execution of the pre-grouting operations and a contingency pay item for additional grout was provided if the documented void ratio exceeded the theoretical void ratio. The intent was to encourage the contractor to limit “runaway” grout takes since the goal was to only “plug” groundwater flow and not to grout all joints and solution features entirely.

Shaft Design Intent

The intent of drawings and specifications was to prescribe minimum criteria for excavation and support and allow the contractor to choose its means and methods. While a water-tight soil support system was specified, the contractor was required to select the method and provide a formal submittal that would detail its equipment, materials, and construction methods. Similarly, the pre-excavation grouting was a required submittal. As noted previously, a suggested method was shown on the contract drawings; however, the contractor was permitted to submit an alternative plan that met the intent of the pre-excavation grouting program as suggested.

PRE-EXCAVATION GROUTING AND SUPPORT OF EXCAVATION

As-Designed Grouting Plan, Design Intent, and Specification Requirements

The design professional developed minimum suggested pre-excavation grout methods that included a three stage grout plug with inclined grout holes. In addition a soft ground/rock interface pre-grouting zone was recommended. Pre-grouting of the tail tunnel, starter tunnel, and deaeration chambers/adits was also required and a horizontal grouting method was depicted in the contract documents as a suggested method to accomplish the pre-grouting for the horizontal excavations.

Pre-excavation grouting was also depicted on the contract drawings and a suggested method was defined that included a staged approach with inclined grout holes. The spacing of grout holes was also suggested and minimum grout limits (15 feet beyond excavation limits) were noted on the drawings. A specification was included that prescribed minimum grout strengths, cement and aggregate requirements, fluidifiers, anti-washout agents, grout pipe and fitting requirements, and admixture requirements. The contractor was required to submit grout mix designs for review by the design professional. Minimum equipment standards and methods were also specified. The contractor was also required to submit working drawings of its grouting equipment, subcontractor experience requirements, grout pressures, sequencing and procedures to be utilized, and reports and records documenting the drilling and grouting operations.

Contractor-Approved Alternative Grouting Plan

The Contractor, Kenny/Obayashi, JV (KOJV) submitted an alternative grouting plan developed specifically for the Project by its grouting subcontractor, Nicholson Construction Company (NCC) that was approved by the design professional as conforming to the design intent. The Contractor's alternative grouting plan used a double-layer grout curtain instead of the grout plug that was included in the contract documents. A plan view from Shaft 2 is shown in Figure 1. The double-layer grout curtain consisted of split-space installed primary, secondary and tertiary grout holes all extending 25 ft below the shaft invert to a depth of nearly 230 ft at the Shaft 1 pump station. The outer layer holes known as Line A were drilled at predetermined angles to angle and batter the grout holes around the shaft to intersect anticipated vertical and horizontal features in the bedrock. The inner layer known as Line B was drilled vertically through casing installed in the diaphragm/slurry walls. For both layers of the curtain Nicholson planned on using upstage grouting unless it was determined in the field that downstage grouting would be required.

Shaft 1 and 2 Pre-Excavation Grouting

The Contractor's grouting program began with installation of an 8 ft wide, 6 inch think unreinforced concrete ring around each shaft. The concrete ring served three purposes: allowed for clear identification of grout holes, provided safe working conditions for laborers and equipment and eased the removal of drill cutting from the grout holes. Once the work platform was completed the Contractor proceeded by setting casing through the overburden, socketing 2 feet into rock, for all grout holes in Line A. After casing installation the Contractor proceeded to drill, pressure wash, water test and downstage grout the 1st stage (25 ft) for all primary holes. Grouting in the interface was done at 0.5 psi per vertical foot of cover. This process was repeated for the secondary holes, and then for the tertiary holes—establishing a cap in the soil/rock interface.

After the first stage was completed for all primary, secondary and tertiary holes the Contractor proceeded to drill the primary holes to full depth, pressure wash, water test and upstage grout each stage until all primary holes were completed. Grout pressures in rock were set not to exceed 1.0 psi per vertical foot of cover. This process was repeated for secondary and tertiary holes until all holes in Line A had been completed.

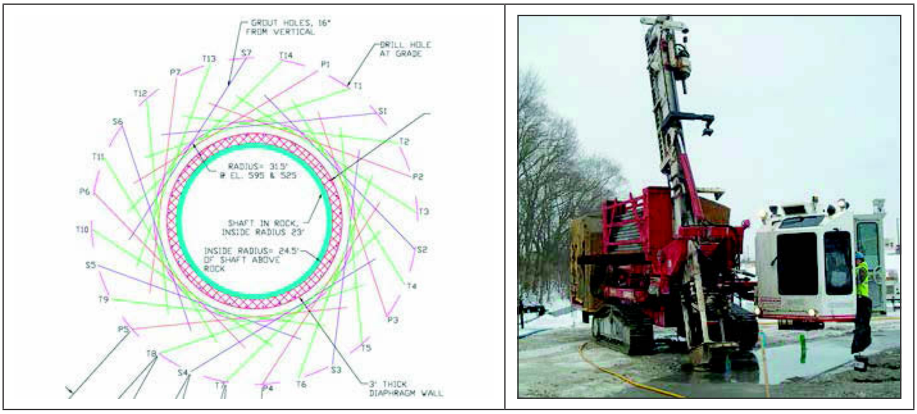

Figure 1. Pre-excavation grouting typical hole layout and rock drilling set up

The Contractor's submittal indicated that a special attachment would be used to pressure wash grout holes, but in the field they chose to utilize drilling water through the drill bit to clean holes before grouting.

During diaphragm/slurry wall installation a PVC casing was placed in with the steel reinforcing cage as a casing for the Line B grout holes. After the diaphragm/slurry wall installation was completed, the Contractor began drilling through casings in slurry wall, through rock to full hole depth. Holes were then pressure washed, water tested and upstage grouted. Secondary and tertiary holes were completed in split-space pattern.

The project specifications defined refusal for a grout hole as a grout take of less than 1 cubic foot over two minutes with the grout pump operating at 100 percent of the maximum injection pressure. This equates to approximately 3.75 gpm at the maximum pressure. The specifications did not specify a maximum grout injection pressure or closure criteria in the form of a maximum grout placed per stage. The project plans call for a performance test to be completed after pre-excavation grouting was completed. As specified by a note on the plans, 4 probe holes were required to be drilled in each pre-grouting zone and if groundwater inflow exceeds 5 gpm from a single hole or 10 gpm from all 4 holes combined additional grouting is required until the requirement is met. Although the requirement for probe holes was based on a zoned grouting approach, they were applied to the Contractor's grout curtain even though the entire shaft would be tested at the same time. In order to meet this requirement the Contractor voluntarily reduced its refusal flow rate to as low as 0.1 gpm. The Contractor experienced very high grout takes in excess of 1,000 gallons in a 25 ft zone in numerous primary and secondary Line A holes. The highest grout takes were frequently found in the upper portion of the bedrock, but occurred in every grout zone.

In a continued effort to meet the performance criteria for the grout curtain, the Contractor drilled and grouted 8 full depth quaternary holes in Line A at Shaft 1. The quaternary holes allowed Nicholson to reach the refusal flow rate with a much lower amount of grout placed per zone. Nicholson utilized an automated and computerized recording and monitoring system called GROUT I.T. to measure, record, and graphically display in real time the total volume of fluids placed in the rock as well as the gage and effective injection pressures, the rate of injection, the apparent lugeon value, the start and stop times of grout injection, and the total time and volume of fluid placed.

Shaft 6 Pre-Excavation Grouting

Grouting operations at Shaft 6 by Nicholson employed the same equipment, procedures, and means & methods as Shaft 1 and 2 with a few differences. At Shaft 6 Nicholson installed the diaphragm wall prior to drilling and grouting the Line A grout holes. This appeared to benefit the grouting operation because it allowed the grouting crew to place grout from the Line A holes into any windows in the diaphragm wall panel joints caused by offsets in verticality. This also enabled the grouting of the slurry wall panel joints vertically from the surface. The result is a more water-tight excavation support system. After the diaphragm wall was installed the Line B holes were installed as they were at Shaft 1 & 2.

Shaft 6 also required the use of quaternary holes in a few locations to tighten up voids and ensure intact soft grout/rock interface as well as zones just above invert were closure was doubted. As of the date this paper was published the shaft excavation has not reached this elevation, so water-tightness has not been verified. Nicholson placed more grout at Shaft 6 than was placed at Shaft 1 and 2 combined, nearly 700 cubic yards.

Cutter Soil Mixing (CSM) Wall at Shaft 1

A CSM wall was installed outside the perimeter of Shaft 1 using a technique developed by Soletanche Bachy including soldier piles as structural reinforcement. Cutter Soil Mixing (CSM) wall is an effective solution for rapid construction of retaining and cut-off walls by mixing soil in-situ with a cement/bentonite grout. The precise positioning and verticality of the wall is achieved using a kelly bar to a depth of up to 40 meters. The CSM method consists of cutting and mixing soil by means of drums mounted on compact hydraulic motors, a method derived from diaphragm wall cutter technology. The drums are essentially designed to combine high penetration rates and improve soil/cement mixing. Precise positioning and verticality is controlled by using a Kelly bar and real time monitoring and processing. CSM provides an adaptable and cost-effective solution with many advantages: mixing the soil in place considerably decreases the spoil volume compared to traditional walls, savings are achieved on natural materials incorporated into the wall, guide wall is cheaper or in certain situations not required, and the CSM equipment does not require a dedicated rig. It can be mounted on standard high torque rotary piling machines, cranes, or carriers equipped with telescopic leaders.

The CSM process consist of two phases: penetration of the tool with outward rotation of the drums whilst injecting a “drilling in” fluid (bentonite slurry or cement bentonite slurry), followed by reversal of the drum rotations and, withdrawal with continued injection employing a binder suspension. The CSM wall is made of successive primary and secondary panels, just like a diaphragm wall. The secondary panels may be excavated through fresh primary panels or hardened ones (a couple days after primary panel installation).

For this project cutter soil mixing (CSM) technology was chosen for the installation of the temporary support of excavation (SOE) at Shaft 1 between El. 700 and El. 650. The CSM wall was necessary in addition to the diaphragm all at Shaft 1 due to the geometry of the pump station piping that required a larger diameter opening near the surface. The selection of this technique was based on Contractors interpretation of geotechnical information provided in the Contract. The CSM wall panels had a width of 2' 8" and were reinforced by driving W18×76 steel soldier piles into fresh mixed material to resist bending moments. The soldier piles were installed in the soil cement mix under their own weight although in some cases vibrators were required. After placing the soldier piles into the cement soil mix, the pile was hung from an installation level “hanging and positioning” device, one foot above the bottom of the excavation.

Figure 2. Cobbles and boulders encountered in panel excavation

During the construction of the SOE with the CSM equipment, high concentrations of boulders and cobbles and nested boulders and cobbles were encountered in the majority of the panels as shown in Figure 2. Although the presence of high concentration of boulders and cobbles, and nested boulders and cobbles were indicated in the Geotechnical Baseline Report (GBR) and Geotechnical Data Report (GDR), they were anticipated to be encountered below CSM wall elevations. GBR and GDR stated boulders and cobbles to be sporadic and occasional from El. 700 to El. 650. Nicholson provided notification to the Construction Management Team (CMT) of a differing site condition (DSC) due to encountering unanticipated boulders, cobbles and a cemented gravel layer. To mitigate cost and schedule impact a revised installation procedure was submitted for approval. The new procedure consisted of excavating the panels using bentonite slurry using a clamshell, backfilling the trench with a cement-bentonite mix after removing the in-situ soils and blending the mix with the CSM equipment while verifying verticality of the trench. Additional equipment and crews were mobilized and a second shift operation was started to maintain schedule. The revised installation method using the clamshell allowed removal of the boulders and cobbles, but resulted in over excavation of the panels in the process. Furthermore, all in-situ material was removed, increasing off-site disposal and additional bentonite and grout were required to backfill the panels. This new installation procedure mirrored the modified slurry wall procedure utilizing the mechanical clam for excavation and filling trench with bentonite until final concrete placed instead of mixing soil in-situ with a cement/bentonite grout as typical with CSM methods.

The DSC caused an increase in both cost to perform the installation of the CSM wall and time of contract performance. To resolve the claim, the CMT negotiated equitable cost and time with KOJV & NCC because the CMT acknowledged increased presence of boulders and cobbles over what was presented in the Contract Documents, but also argued that Nicholson's means and methods (specifically the CSM) would have been impacted even if only sporadic and occasional boulders and cobbles were encountered.

Diaphragm/Slurry Wall at Shaft 1, 2, and 6

OARS Phase 1 consists of the construction of support of excavation for three large diameter shafts through approximately 100 ft of overburden. The contract documents required that the overburden support method extend at least 5-ft into the underlying bedrock to provide structural support and minimize groundwater infiltration through the transition zone. Kenny/Obayashi JV and Nicholson Construction selected diaphragm wall excavation to provide the support of excavation.

Diaphragm walls were constructed using a mechanical grab "clam bucket" and a hydrafraise. The excavation operation is shown in Figure 3. During the excavation

Figure 3. Diaphragm wall panel excavation

process, bentonite slurry is pumped into the excavation trench and maintained within 3ft of the top of the guide wall. The verticality of the trench is monitored by visual inspection of the grab cables during the successive lowering of grab into trench or by data recording system built into hydrafraise. Shaft 1 and 6 are comprised of 9 panels each and Shaft 2 contains 8 panels. Panels can be classified into 3 types according to their excavation sequence—Primary, Follower, and Secondary or Closure Panels. A primary panel is constructed initially prior to installation of an adjacent panel. The joints are formed by end stops installed at both ends of the panel. A follower panel is a panel that has only one adjacent panel. One end of the panel is formed by end stop, and the other end of the joint is cast directly against the previously constructed panel. A secondary panel or closure panel has two previously cast adjacent panels. Both ends of the panel are cast directly against the previously constructed panels. In all cases, the joints are keyed together with the "Coffrage avec Waterstop" (CWS) developed by Nicholson's parent company Soletanche Bachy. The CWS forms a water-tight joint between the panels of the diaphragm wall while providing the required shear key to prevent relative movement between adjacent panels. The CWS end stop casts a continuous water barrier within the concrete and provides a positive water barrier at the joint.

The rock socket/toe was formed cutting the rock with the hydrafraise. The excavation depth was determined by visual inspection of the excavated rock samples taken from the desanding unit when the hydrafraise was in operation. Typically, the operator had extended 10-ft ± into the underlying bedrock by the time engineers confirmed rock samples were sound and the panel desanding & cleaning was completed since the hydrafraise is prone to continue excavating downward during the desanding and cleaning sequence. While excavating, control of trench verticality is maintained by measuring the position of the suspension wire of the clamshell or chisel in relative to guide walls or recording excavation parameters with a data recording system when using the hydrafraise. Nicholson found that in addition to installing a cast-in-place concrete guide wall at the surface, utilizing the mechanical clam bucket for approximately first 30-ft of excavation helped the hydrafraise maintain verticality for remaining excavation to bedrock.

Maintaining verticality required careful attention by the operator as the hydrafraise typically "kicked" or penetrated at an undesired angled when the teeth first encountered bedrock. Additionally the presence of boulders and cobbles in the overburden impacted verticality as well as production. Large granite boulders were unable to be crushed or ripped apart by the hydrafraise and became trapped between the cutting wheels, clogging the system. This required removal of hydrafraise from the excavation so the boulder could be removed. This resulted in very low productivity. In order to achieve the required production through the overburden Nicholson began utilizing a mechanical clam bucket that was able to scoop up the boulders and surrounding overburden.

Since the diaphragm wall serves as the support of excavation for the shaft nominal reinforcement was required. The reinforcement cages, including PVC pipe conduit for Line B grouting, were pre-fabricated off-site and arrived in two sections. They were spliced together and then lowered into the trench. In order to ensure 3-inches of concrete cover around the reinforcing cage, spacers were attached to both sides of cages. Concrete placement was done by tremie from the bottom of the panel, displacing the bentonite. A highly workable mix of concrete was placed directly from a concrete truck into the tremie pipe. Tremie pipes were removed section by section as the concrete was placed, always maintaining at least 6-ft of tremie pipe below the concrete level.

A differing site condition (DSC) claim was filed by the Contractor for impacts suffered during the installation of diaphragm walls at Shaft 1 and Shaft 2. The method to install the support of excavation and the support of excavation itself were left up to the Contractor and based on contractor's interpretation of geotechnical information provided in the contract. As discussed previously construction of the SOE with the diaphragm wall equipment, high concentrations of boulders and cobbles and nested boulders and cobbles were encountered in the majority of the panels. The claim stated that the higher concentrations of cobbles and boulders could not have been anticipated based on information provided in the Geotechnical Baseline Report (GBR) and the Geotechnical Date Report (GDR).

To minimize schedule impacts the contractor had proceeded to work a 12 hour-shift 6 days/week for both hydrafraise and clamshell equipment. A heavier clamshell had also been mobilized to the site to aid the removal of large boulders from within the excavation. It was the CMT's opinion that whatever method selected by the Contractor should have accounted for downtime associated with dealing with these problematic zones. The CMT negotiated an equitable adjustment with the Contractor taking into account the equipment selection and other issues that had impacted diaphragm wall production including: higher than anticipated mechanical downtime suffered by the hydrafraise, other associated equipment downtime and inefficiencies, and standby waiting for replacements parts. In addition, it was understood that many of the obstructions encountered were part of the two distinct zones identified as potentially problematic in the GBR—an upper zone with sporadic, occasional, and nested boulders and the lower zone with numerous very hard cobbles and boulders.

Jet Grouting at Shaft 1

Jet grouting was utilized on the project at Shaft 1 to fill the space between the outside of the diaphragm wall and the inside of the CSM wall to maintain a water tight support of excavation from the surface to the overburden/rock interface. The jet grouted zone was approximately 7 ft thick and extended from a depth of 38 ft to 45 ft below the ground surface.

Nicholson selected a double fluid jet grouting technique. This system uses a double-tube jet of water/grout and air which is introduced at high pressure in order to form the columns in the soil below. The double jet rod is composed of specially designed rod with two conduits: the inner for the high pressure grouting and the outer for the compressed air. Nozzles through which grout and air are jetting out are located in the jet or "monitor." Once the monitor reaches the required depth, it is rotated and the jetting fluids are pumped at high pressure to the jetting tip as the monitor is withdrawn from the hole at a controlled rate to form an in-situ column.

The diameter of each jet grout column is a function of the pressure utilized by the Contractor and the soil properties. In a consistent soil the column will appear relatively homogenous; however, when the soil is inconsistent and when there are cobbles and boulders present the column diameter can vary significantly. The design diameter for each column was 8 ft. Columns were spaced at 6'-6" c/c for Line-A, 5'-6" c/c for Line-B and 4'11" c/c for Line-C. During the jet grouting, Nicholson closely monitored the overflow of spoil at the hole collar to verify no blockage or heaving occurs in vicinity.

Based on the differing site condition (DSC) encountered during the CSM and the diaphragm wall excavation, both the CMT and the Contractor believed there to be a high concentration of rock fragments, boulders and cobbles in the jet grout plug zone. To mitigate the potential issue, the CMT compensated the contractor to mobilize additional resources to pre-drill the jet grout holes through the overburden and backfill the holes with a cement/bentonite mix. By pre-drilling these holes the Contractor did not have to worry about encountering a boulder with his jet grouting equipment. The Contractor simply drilled through the placed cement/bentonite with the jet grouting equipment and placed the columns. The means and methods modification was successful and the jet grouting activities were completed faster than anticipated, thereby offsetting some of the cost of the pre-drilling

SHAFT AND STARTER TUNNEL EXCAVATION

Overburden Excavation at Shafts 1 and 2

As discussed, a significant number of cobbles and boulders were encountered by the contractor while constructing excavation support perimeter walls utilizing cutter soil mixing and slurry (diaphragm) wall methods. While the presence of cobbles and boulders was problematic for the support of excavation installation, once completed the overburden was able to be excavated with minimal issues. The Contractor utilized a larger CAT 320C excavator lowered into the shaft by crane and a 12 cubic yard muck box. The muck box was lowered into the shaft by crane, filled by the excavator, and then removed by crane. Shaft 1 was large enough in diameter to allow two excavators working simultaneously which significantly reduced the cycle time.

Overburden Excavation at Shaft 6

Overburden excavation a Shaft 6 proceeded without incident as well. Cobbles and boulders were not encountered at Shaft 6 during diaphragm wall installation and when excavated it was found that the overburden consisted of a 10 foot granular fill material, followed by 80’ of cohesive soils (medium and dense clay) over underlying bedrock.

Bedrock Excavation at Shaft 2

Limestone bedrock was encountered at approximately EL 600 ft in Shaft 2, just over 100 ft below the ground surface. After removing all overburden the Contractor proceeded with a Drilling, Blasting, & Mucking (DBM) cycle. The Contractor proceeded with drilling an approved blasting pattern with one or more rock drills. Hole depth varied from 6 ft to 12 ft and both full shaft and half shaft blasting (benching) was performed.

Issues were quickly encountered after the first blast at Shaft 2. The first 12 foot lift had been divided into two halves of the shaft and after muck had been removed the Contractor proceeded drilling for the 2nd 12 ft deep blast round. During this drilling operation a significant water inflow event occurred. The crew from the previous day had noted that they had encountered numerous voids, but did not see any water inflow. Flow initially began coming through the shaft wall, then moved to the shaft floor, where the flow rate increased substantially. The crew was forced to evacuate the shaft as they feared the water inflow may destabilize the excavation. Water inflow was estimated to be approximately 1,400 gpm by measuring the rise in water level in the shaft at around 1 inch per minute. In order to stabilize the excavation the Contractor began pumping water into the shaft to equalize the water level in the shaft with the groundwater level (Figure 4).

Figure 4. Excavator in Shaft 2 surrounded by diaphragm walls, and drilling for blast round

After stabilizing the groundwater level, the Contractor re-drilled three Line B interior layer grout holes at the surface on the side of the shaft near the water inflow, and checked each hole for its ability to hold water. Three of the holes did not display a hydraulic connection to the shaft; however the 4th hole displayed a hydraulic connection to the shaft water level as well as a nearby monitoring well. The Contractor placed 12,500 gallons of grout into in this hole and another hole that was also re-drilled nearby. After grouting was completed and allowed to set the Contractor began dewatering the shaft with high capacity high head pumps. As the shaft was pumped down, pumping was stopped intermittently to check for recharge and when none was seen dewatering continued. The shaft was fully dewatered approximately 2 weeks after the inflow event. After dewatering the shaft it was inspected and a clay seam approximately 3 foot thick filled with cobbles was identified below on the diaphragm wall panels. The panel had terminated on rock, but the rock was underlain with a very large clay seam which dipped from right to left across the panel. In an effort to stabilize the excavation the Contractor made preparations to shotcrete across the clay infilling after installing welded wire fabric and rock dowels for support, however due to difficulties placing shotcrete the Contractor chose instead to install a concrete ring around the entire shaft. After installation of the concrete ring the Contractor continued his DBM cycle in the shaft.

The next blast cycle was all drilled at once, but the inner rings were loaded and blasted prior to the two outer rings. The Contractor did this to provide more relief for the outer ring blast and prevent encountering addition water inflow. However after blasting the perimeter during the mucking operation the crew again experienced a water inflow into the shaft of approximately 1,200 gpm, but from a different location. The Contractor responded by again flooding the shaft with water to equalize the shaft water level with the groundwater elevation. After the water level was equalized two Line B interior layer grout holes were re-drilled near the water inflow, communication to the shaft was noted in one of the holes. As had been done with the previous inflow the Contractor began pumping grout into the re-drilled Line B holes. Over 2 days over 30,000 gallons of grout was placed. After grout placement the water level in the shaft was raised above the groundwater elevation to check for closure, but water was measured to be flowing out of the shaft at approximately 90 gpm. Since a conduit was still open in and out of the shaft the Contractor mobilized a chemical polyurethane grouting operation. The grout used was a two-part Mountain Grout brand product with an added accelerator. Varying the quantity of accelerator allows the set time of the grout to be quickened or delayed. The chemical grout was placed down the same hole that had been previously grouted with 30,000 gallons of cementitious grout. Prior to grouting the water level in the shaft was raised to 3 feet above static level. While grouting at this level some communication was noted into the shaft (floating chemical grout), so the water level was increased to 6 ft above static. After increasing water level no communication was noted. 115 gallons of chemical grout was placed. After placement no water loss was observed over an hour period and since the chemical grout sets up nearly instantaneously the Contractor began dewatering the shaft in 10 foot increments. At each increment the pumping would stop for 2 hours to check for recharge into the shaft. After successfully pumping down for nearly 12 hours, recharge of approximately 100 gpm was noted during a check. This flow rate continued to increase to 400 gpm, and at that point the Contractor abandoned the attempt to fully dewater the shaft without additional grouting.

As grouting from the surface with both cementitious and chemical grouting had not been successful, the Contractor contacted Pro-Dive Construction Contractors, to proceed with a plan of inserting a grouting hose into the void while the shaft was full of water. During the dive constant contact was maintained with the diver through audio feed and live video feed. The diver was able to quickly identify the void area. Initially it was determined that the hose was not flexible enough to enter void, however after cleaning the opening the diver was able to insert hose over 6 ft into the void. The diver noted the void was about 14 inches deep before turning down, then to the right behind some set-up chemical grout. When he stuck his hand into the void he could feel significant flow entering the shaft and also noted the shaft was "smooth" at the elevation of the void likely due to grout that had flowed back into the shaft, filling the shaft floor. Prior to grouting the Contractor brought the water level in the shaft up to 11 ft above static to prevent grout from flowing back into the shaft. Grouting was done simultaneously through two hoses, one pumping cementitious grout and one pumping chemical grout. A total of nearly 14,000 gallons of cement grout and 165 gallons of chemical grout were placed in the first day of grouting and as this was being done on a Friday no other operations were conducted over the weekend, allowing the grout to set. Two days later inflow was measured at 40 gpm, so additional grouting was done. An additional 6,000 gallons of cement grout and 20 gallons of chemical grout were placed. After placement of this grout the water level in the shaft held constant. After allowing the grout to set, the shaft was dewatered.

Prior to proceeding with further drilling and blasting the Contractor began drilling holes around the perimeter of the shaft, angled outwards 10–15 degrees and approximately 12 feet long. These holes were then grouted with a single part Sub-Technical, Inc. polyurethane grout with a 60 second set time. This perimeter cut off grouting was designed to intercept any other water making features before they were encountered by blasting. This method proved to be successful as no other water making features were encountered in the Shaft 2 excavation, although it is possible that no other features were encountered.

Bedrock Excavation at Shaft 1

Limestone bedrock was encountered at approximately EL 592 ft in Shaft 1, just over 100 ft below the ground surface. After removing all overburden the Contractor proceeded with a Drilling, Blasting, & Mucking (DBM) cycle. Due to the groundwater issues encountered in Shaft 2, the Contractor proceeded cautiously excavating Shaft 1.

The Contractor successful drilled, blasted and mucked two lifts from EL 592 ft to EL 580 ft, although significant communication was noted between drilled holes and clayey and silty return water was found while drilling blast holes. Drilling for the 3rd lift at EL 580 ft drillers noted even greater communication and a 5 to 6 ft thick soft area approximately 3 ft deep on the south side of the shaft, and water was seen bubbling up through a seam running east-west across the shaft. Grouting with chemical grout was done in the wall, but not in the floor prior to the blast. When the lift was blasted and smoke cleared a large water inflow could be seen on the north side of the shaft, seeming to come out of the shaft wall. Inflow was estimated at over 2,000 gpm. Since the muck had yet to be removed and the excavation was deemed too dangerous to enter with the water inflow, the Contractor brought in a large clam bucket from off-site and proceeded removing the muck with the clam bucket. Material removal was focused to area local to inflow on north side of shaft.

Due to the significant water inflows at Shaft 2, the Contractor had already mobilized high capacity and high head pumps on-site. Five pumps running simultaneously were used to control the inflow in Shaft 1, and equalize the groundwater coming in with pumped water going out. This allowed the Contractor to lower an excavator into the shaft and remove additional material around the inflow. A two part polyurethane grout with a set time between 5 and 10 seconds was used to attempt to stop the water inflow. Initially packers were drilled around the inflow but the inflow was of much too high a volume for the chemical grout to set up. The chemical grout was too diluted and expelled back into the shaft too quickly. Eventually the operation was called off and grout piping was placed into the void, and piped to the surface so that the shaft could be flooded and grouted under positive pressure as had been done at Shaft 2. Three 1 inch lines and two 2 inch lines were installed to allow for simultaneous chemical, cement, and sanded cement grouting. Cement grout was pumped initially to slick the lines, followed by sanded cement grout. Contractor had difficulty keeping consistent flow through the line and it was suspected that the sand was separating from the grout, therefore sanded grout was terminated. Contractor continued grouting with cementitious grout for approximately 3 hours, after grouting the shaft water level started to stabilize. At this point single part chemical grout was introduced as well as anti-washout agent in the cementitious grout to assist in closure of the void.

After allowing the grout to set for a few days over the weekend the Contractor began dewatering the shaft, stopping every 25 ft to check for recharge. Shaft successfully dewatered over next 24 hours with high capacity pumps. With pumps running shaft water level could be maintained at invert. Once dewatered the crew mobilized a drill rig to drill holes into the seams to place two part polyurethane chemical grout. Grouting was unsuccessful and inflows increased as silt and clay was washed out from the seams. Flows increased from 20 gpm to 150 gpm over a few hours, and 12 hours later flow had increased to over 1,000 gpm. After shoving new grout pipes into void, the Contractor removed his dewatering pumps and began flooding the shaft a second time. The water level in the shaft was raised to 6 feet above the static water elevation to achieve a positive flow out of the shaft in the range of 250–300 gpm. Grouting was done for two days and over 11,000 gallons of cementitious grout and 385 gallons of one part polyurethane chemical grout. After allowing the grout to set again over the weekend, dewatering was conducted in a similar manner, checking recharge every 25 ft.

Shaft was successfully dewatered and although multiple leaks still existed around the shaft perimeter flow rates had been greatly reduced, and shaft bottom remained stable with pumps operating. Over the next few days crew worked on drilling for chemical grout packers and grouting with the two part chemical grout with the 5–10 second reaction time to plug up any remaining leaks. As leaks began to be cut-off by the chemical grout around the shaft, a leak that had been previously noted at 2–3 gpm on the north side began flowing at substantially higher flow rates, in excess of 1,000 gpm as shown in Figure 5. The Contractor immediately turned his attention to this leak but even after pumping 220 gallons of chemical grout the leak had not attenuated even slightly. Probing of the leak indicated the void went up to 12 ft back into the shaft wall and the shaft opening had eroded to approximately 3 feet across by 2 feet high. In order to stop the leak the Contractor was able to control the water by shoving timber and burlap into the void and then pumping chemical grout, the timber and burlap help reduce the size of the flow path and catch the chemical grout, allowing the void to slowly clot.

Once the leaks were attenuated, the Contractor set up steel forms and poured a concrete collar around the entire shaft to prevent future water inflows in the area.

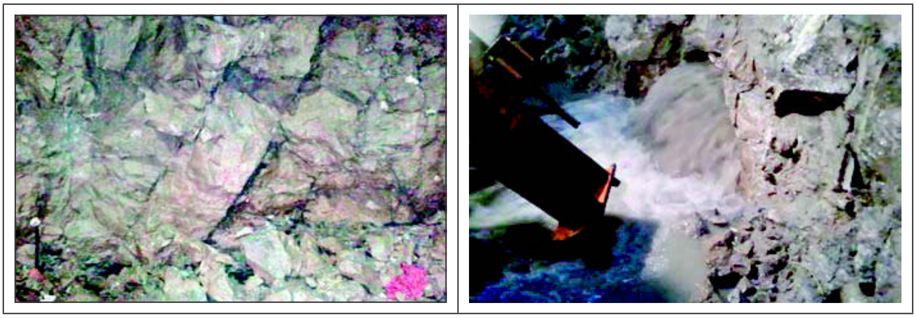

Figure 5. High angle joint in limestone with clay in-filling, and water inflow through joint

Once the concrete collar was poured, the Contractor continued his DBM cycle with the additional of perimeter chemical grouting. No other major water inflow events occurred through the remainder of Shaft 1 excavation.

Rock Excavation at Shaft 6

To date, the Contractor has done four blasts in rock and has yet to encounter any water infiltration. To mitigate risk and reoccurrence of Shaft 1 and Shaft 2 inflows due to clay seams, the CMT is compensating the Contractor to perform chemical grouting around perimeter of shaft before each blast as was done at Shaft 1 and 2. Shaft forms are also readily available to pour concrete collars for additional mitigation if necessary.

LESSONS LEARNED

Chemical Grouting to the Rescue

The use of polyurethane grouts to retard water inflow was critical to the successful shaft sinking. While an excellent tool, proper use and storage are critical. As most of the grouting was being done in the winter we found that the set time was very temperature dependent. Product literature for the polyurethane grout stated that it should not be used below 60 degrees Fahrenheit and we found this to be true. When grout was at or near freezing temperatures the chemical reaction was slowed by 5 or 10 times. Additionally the experience of the grouting crew was of paramount importance to avoid wasting large quantities of the very expensive chemical grout. If pumped too quickly or if the leak to be grouted is not properly prepared, thousands of dollars of material can be quickly washed back into the shaft and wasted.

Clay Filled Joints Were Undetected

Although a significant geotechnical exploration was conducted as part of the design including hundreds of borings along the alignment and over thirty borings around Shafts 1 and 2, clay filled high-angle joints went undetected. The clay infilling prevented the features from being effectively grouted, and when excavated and unconfined the clay infilling was washed out leading to major water inflows. Because of the high angle nature of the features the vertical borings done during the geotechnical excavation likely did not contact the features at an angle that allowed the clay to be sampled. If high angle joints are suspected angled borings could be considered to attempt to get a better sampling of infilling. In addition, prior to grouting, pressure washing and flushing of drilled holes is extremely critical to wash out as much infilling as possible.

Importance of Clear and Concise Baselines

Bid items were developed that functioned as baselines. Geotechnical conditions that could not be properly quantified (during design or construction) were not developed into baselines. It was also believed that specified construction methods eliminated the need to baseline some conditions. For example, a baseline was not stated for water inflow into the shafts because it was believed that the specified impervious soil support of excavation and pre-excavation grouting of rock eliminated the need for developing a baseline. Numerous summary tables were included in the baseline report including soil tests (granular and cohesive), rock laboratory tests, hydraulic conductivity tests for soils, hydraulic conductivity tests for rock members, soil strata information (ranges), and bedrock strata information (ranges).

During construction cobbles and boulders were encountered during the construction of slurry walls and cutter soil mix walls. The contractor submitted a claim that stated the number and concentration of cobbles and boulders impacted its production and requested compensation for the impacts. Due to the absence of baselines for cobbles and boulders in the Geotechnical Baseline Report, the contractor and construction manager were required to interpret baseline “statements” concerning the presence cobbles and boulders. The interpretation efforts may have been avoided if concise baselines were developed and the quantity/concentration of boulders were able to be determined during construction.

USE OF A DIGGER SHIELD TO SUCCESSFULLY COMPLETE TUNNEL AFTER GROUND CONDITIONS PROVED TOO ADVERSE FOR A TBM

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ABSTRACT

Construction of the 1,829 m long (6,000 ft) Portsmouth Force Main Tunnel in Portland, Oregon, met with difficulties after the tunnel boring machine (TBM) was unable to advance through running soils containing cobbles and boulders. Significant ground loss occurred during efforts to break up and pass the boulders through the cutterhead. Efforts to stabilize the ground and advance the machine were unsuccessful, so the tunneling method was re-evaluated. This paper discusses construction challenges and efforts undertaken to resume tunneling. A plan was implemented to remove the TBM and replace it with a digger shield. TBM retrieval was through a jacked steel casing enveloping tunnel support system and machine.

INTRODUCTION

The Portsmouth Force Main (PFM) Project includes approximately 4,877 m (16,000 ft) of a single 1.68 mm (66 in.) force main that conveys up to 454 million liters (120 million gallons) per day of combined sewage from the Swan Island CSO Pump Station (SIPS) to the existing 1.83 m diameter (72 in.) Portsmouth Tunnel in North Portland. The project was one of the last major components of the City of Portland, Bureau of Environmental Services' (BES) program to minimize combined sewer overflows (CSO) into the Willamette River. The project was divided into two segments, Segment 1 and Segment 2, as shown in Figure 1.

Segment 1 consisted of 2,134 m (7,000 ft) of open-cut construction, and 914 m (3,000 ft) of large-diameter, 2.13 m (84 in.) microtunnel. Segment 2 consisted of 1,829 m (6,000 ft) of conventionally excavated soft ground tunnel through the highland bluff that borders the east bank of the Willamette River. The tunnel contains a 1,676 mm ID (66 in.) fiberglass reinforced polymer (Hobas) force main pipe. The tunnel extends up to 42.7 m deep (140 ft) through sandy Catastrophic Glacial Flood Deposits and Troutdale Formation gravel between the South Portal Shaft and the North Connection Shaft. The tunnel was mined and lined from a shaft at the south end of the alignment. Figure 2 shows the South Portal Shaft configuration and the railroad runs between the shaft and the base of the bluff.

Construction of the Segment 2 tunnel met with difficulties after the tunnel boring machine (TBM) was unable to advance through running soils containing cobbles and boulders. Significant ground loss occurred during efforts to break up and pass the boulders through the TBM. Efforts to stabilize the ground and advance the machine were unsuccessful, so the tunneling method was re-evaluated. A plan was implemented to

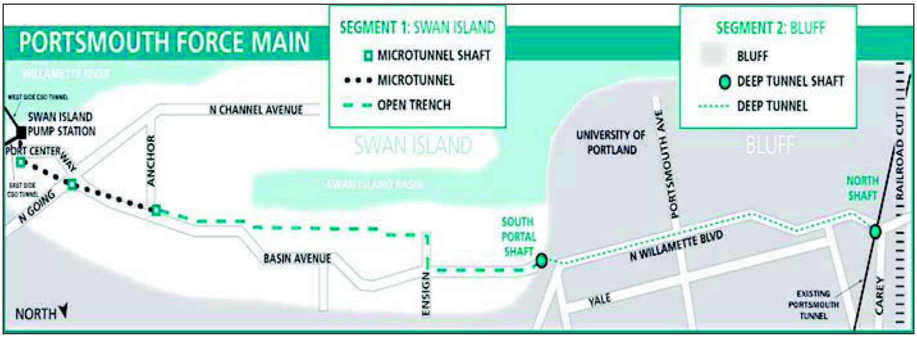

Figure 1. Project alignment

Figure 2. Aerial view of south portal shaft and railroad crossing

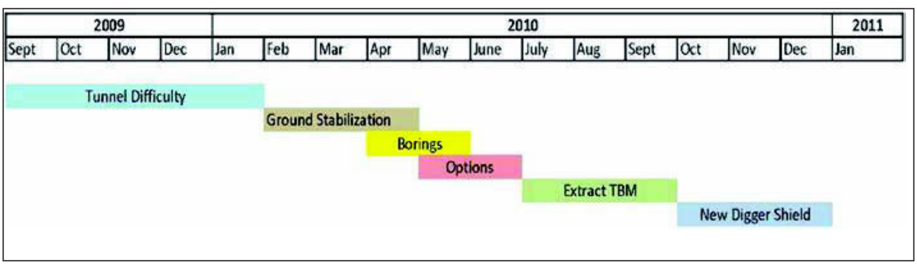

Figure 3. Project time line

remove the TBM and replace it with a digger shield. TBM retrieval was through a jacked steel casing enveloping tunnel support system and machine. Figure 3 indicates the general timeline for the project elements discussed herein.

SUBSURFACE CONDITIONS

Geologic conditions along the alignment vary dramatically between the lowland and bluff highland intervals, as discussed below. During tunnel design, six mud rotary and four rotosonic borings were advanced to evaluate soil engineering properties and develop a geologic profile for the tunnel alignment (shown in Figure 4). Boring locations varied

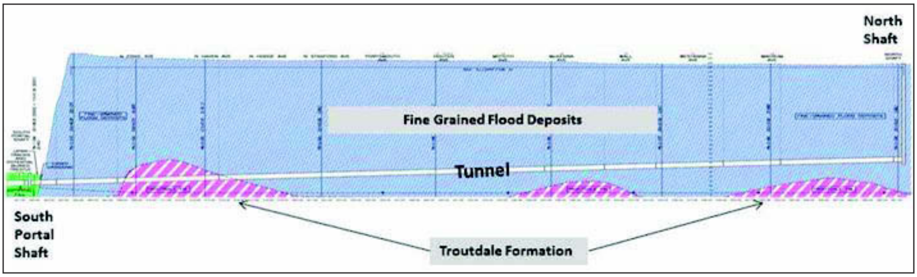

Figure 4. Interpretive geologic profile along tunnel alignment

slightly from the final tunnel alignment due to traffic control concerns. The rotosonic boring on the south end of the tunnel alignment is offset approximately 250 feet from the alignment. Although geologic contacts were interpreted between borehole locations, the geologic profile served as the baseline for geologic conditions within tunnel and shaft excavations. Groundwater was expected to be below the tunnel for the entire alignment.

The Mocks Bottom lowlands in the vicinity of the South Portal Shaft are underlain by fine-grained alluvium that was deposited in an abandoned meander of the Willamette River. Artificial fill, predominantly consisting of sandy dredge spoils, was placed across Mocks Bottom starting in the 1930s to raise the ground surface above the 100-year flood elevation.

Although the bluff slope adjacent to the South Portal Shaft and above the railroad crossing has shown no historic evidence of instability, it was considered marginally stable. Historic and active slope failures in the sandy bluff slope have been identified to the east and west of the tunnel alignment.

The bluff and highlands are underlain by fine-grained flood deposits. These flood deposits generally consist of interbedded sand and sand with silt, with isolated lenses of small gravel. The deposits are on the order of 30.5 to 43.6 m (100–140 ft) in the alignment corridor. Ice-rafted boulders are known to have been deposited along with the fine-grained flood deposits during flood events. These boulders consist of igneous or metamorphic rock (granite, gneiss, or quartzite). Boulders were not encountered in the project borings, but one boulder was removed from the North Shaft excavation.

Troutdale Formation gravel deposits and interbedded sand lenses underlie the fine-grained flood deposits. The upper contact of the Troutdale Formation has been eroded, forming an irregular geologic contact. Cobbles and nested cobbles are present within the Troutdale Formation. A probable 0.6 m diameter (2 ft) boulder was encountered in the Troutdale Formation below the tunnel zone in one project borehole. Troutdale Formation gravel is commonly cemented, rarely contains boulders, and has good stand-up time. In contrast, more recent Coarse-grained Catastrophic Flood Deposits, which underlie Mocks Bottom lowlands, are rarely cemented, contain frequent boulders, and have poor stand-up time.

Based on the mud rotary borings, the geology in the bluff was interpreted to be fine-grained catastrophic flood deposits over older Troutdale Formation gravel deposits for the entire alignment; however, standard penetration test (SPT) sampling is not ideal for identifying Troutdale gravel. Gravel deposits were not encountered in the design rotosonic borings, which are best for identifying Troutdale gravel, with the exception of the north shaft boring (below the tunnel).

In hindsight, the south end of the tunnel likely encountered Coarse-grained Catastrophic Flood Deposits; and not Troutdale gravel. This became apparent only after the tunnel encountered un-cemented, bouldery deposits and was ultimately confirmed when four additional rotosonic borings were drilled along the south end of the tunnel alignment during construction.

DESIGN REQUIREMENTS

Because of the presence of predominantly sandy soils along the tunnel alignment, ground settlement resulting from ground loss was a major concern. The tunnel alignment crossed under a steep bluff and under a major roadway leading to only entrance to the University of Portland. A 305 mm diameter (12 in.) cast iron water line runs the entire length of the tunnel alignment, and a jet fuel line parallels and crosses the central portion of the alignment. These structures cannot tolerate settlement. The specifications provided two settlement thresholds, referred to as the “Action Trigger Level” and the “Maximum Allowable Movement.” The contractor was responsible for preventing settlement and was required to submit corrective measures, taken when thresholds were exceeded and, and verify that these corrective measures were effective.

A geotechnical instrumentation and monitoring program was incorporated into the project. Instrumentation included surface settlement control point arrays. Each array consisted of three points that are centered above tunnel centerline. At selected locations, the central settlement control point was replaced with a settlement casing to monitor ground settlement below the surface that could propagate to the surface. The first combined surface/subsurface monitoring array was located near the start of the tunnel.

In specifying requirements for the tunneling operations, there were numerous discussions on how prescriptive to be on the machine requirements. In the end, it was decided to allow flexibility for the contractor to select the machine it felt best suited the conditions. Both a digger shield and an earth pressure balance (EPB) TBM were allowed; however, provisions for face control were required for both options.

To help manage risk, the project included a geotechnical baseline report that described the materials expected to be encountered and included baselines for the ground behavior for the tunnel excavation. The anticipated ground behavior through the fine-grained flood deposits was predominantly slow raveling to fast raveling. Boulders were baselined for the Troutdale Formation and in mixed face conditions along geologic contacts.

The GBR also described the successful construction of the existing 2,179 m long (7,149 ft), horseshoe-shaped Portsmouth Tunnel. This tunnel was constructed between 1966 and 1967 using an 2.4 m diameter (8 ft) open-face pneumatic shield in the fine grained flood deposits. Figure 5 shows the shield that was used. Tunnel excavation took eight months to complete, and five men worked at the heading during each shift. This past experience confirmed that the fine grained flood deposits are favorable for tunneling.

RAILROAD CROSSING

A cased crossing was required by Union Pacific Railroad (UPRR) for the force main construction beneath its tracks adjacent to the South Portal Shaft. A minimum of 1.4 m (4.5 ft) of vertical cover between the top of the track tie to the top of the casing was required. The original crossing permit envisioned concrete casing pipe being installed using pipe jacking methods from the South shaft to the northern edge of the (UPRR) right-of-way. The contractor requested a modification to the crossing permit to install a 3,048 mm diameter (120 in.) steel

Figure 5. Portsmouth Tunnel open-face pneumatic shield (front view) in 1967 photograph

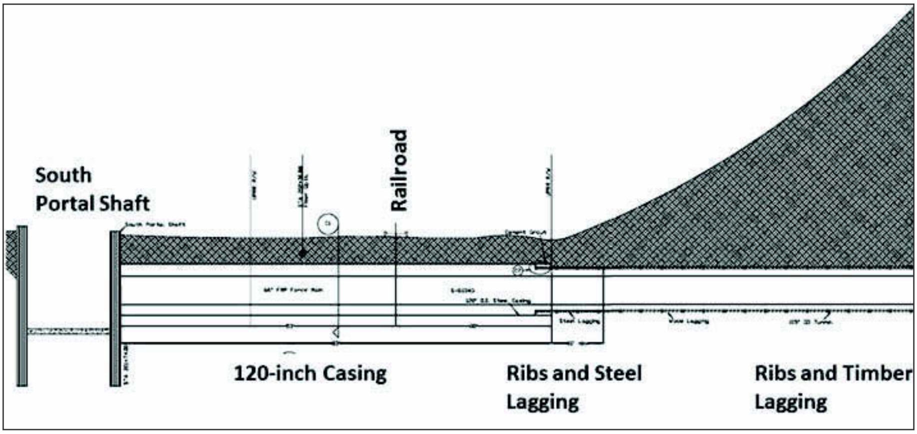

Figure 6. Railroad crossing profile for turning under the highland bluff

casing beneath the tracks using open-cut methods. This was approved by UPRR because traffic on the track consisted of a single train, no more than once per day Monday through Friday. The steel casing pipe was backfilled in the trench with controlled-density fill (CDF). UPRR personnel replaced the ties, ballast, and track. Figure 6 shows the crossing configuration. The sizing of the casing proved to be an essential criterion for being able to remove the TBM, as discussed later.

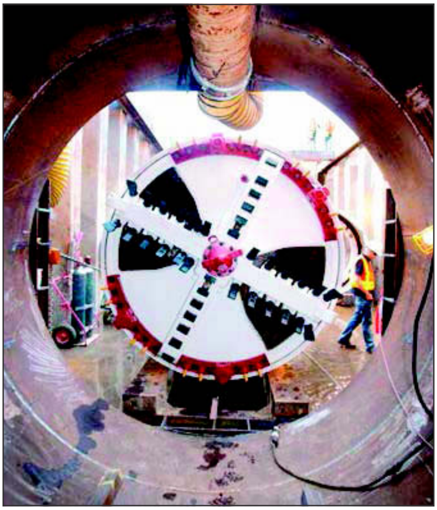

Figure 7. Lovat TBM in South Portal shaft

TUNNEL EXCAVATION

The TBM was launched through the casing in September 2009. The contractor selected a Lovat MP104PJ, Series 11200 TBM to excavate a 2,642 mm diameter (104 in.) tunnel. The TBM consisted of a conventional open face cutterhead with closure doors, internal pressure regulated gates (muck ring) and a conveyor system for the transport of spoil from the face to the muck cars. Figure 7 shows the cutterhead face of the machine looking out through the casing. The tunnel was supported by steel ribs and steel lagging for about 3 m (10 ft) beyond the end of the 3,048 mm (120 in.) steel casing, at which point wood lagging was utilized. A double track switch was constructed inside the casing and through to the shaft, where an empty train of muck cars and flat car with materials, was stationed for transport into the tunnel.

The TBM was run in open mode above groundwater with the ability to control fast raveling to running ground with the muck ring and pressure regulating gates. Temporary support consisted of steel ribs and timber lagging. Although extensive research was conducted to identify Segment 2 buried obstructions, unidentified cobbles and boulders were encountered beneath the bluff slope at the start of tunneling.

TUNNELING DIFFICULTIES

Coarse grained gravel deposits containing loose cobbles and boulders were encountered during the initiation of tunneling, leading to significant difficulties for the TBM. These deposits are not typical of the Troutdale Formation and were not identified in project design boreholes. In addition, the boulders were greater in size and greater in number than described in the GBR for the Troutdale Formation. The TBM cutting tools were not configured to break boulders, so the tunneling method consisted solely of pulling boulders through the face of the machine. Early on, the muck ring was damaged and the pressure regulating gates were removed to provide better face access to remove cobbles and boulders.

Several of the boulders encountered were too large to fit through the doors of the head of the TBM and had to be broken up by hand in front of the machine. Boulders that were small enough to fit through the doors also impacted the excavation, since to bring them into the TBM the doors had to be opened as wide as possible, allowing the fine-grained soils to run into the heading. The TBM cutterhead direction also had to be alternated to try to catch the boulders in the cutterhead opening and bring them into a position where they could be broken by hand. These efforts resulted in significant ground disturbance and overexcavation. Figure 8 shows a typical boulder in the cutterhead and the void that formed in front of the machine. Several large sinkholes formed in the bluff slope above the tunnel, as shown in Figure 9.

To keep the TBM going, a remedial grouting program, using polyurethane and cement bentonite grout, was implemented through horizontal holes in the bluff slope and through the tunnel face to help stabilize the fine grained soils. The process was marginally effective and resulted in extremely slow progress and numerous delays. The tunnel only advanced about 56 m (184 ft) in 8 months of active mining.

Midway through the remedial grouting efforts it was determined that grouting from the ground surface would be a more effective method for stabilizing the soils above the tunnel and preventing additional sinkholes. A surface grouting program was initiated in March 2010. The program utilized sodium silicate grout injected through vertical and battered sleeve port pipes to stabilize the granular soils (Harkins, 2012). The pipes were installed from a hiking trail located mid slope above the TBM. Figure 10 shows the grouting plan and section that was implemented. The grouting stabilized an approximately 21 m long (70 ft) zone behind and in front of the TBM. Upon completion of the grouting, the TBM was advanced an additional 12 m (40 ft) into the grout stabilized zone so that the TBM could be inspected and tunneling methods reevaluated by the construction team.

Figure 8. Boulder in cutterhead opening with ground loss void due to running ground

Figure 9. Sinkhole in bluff slope above tunnel

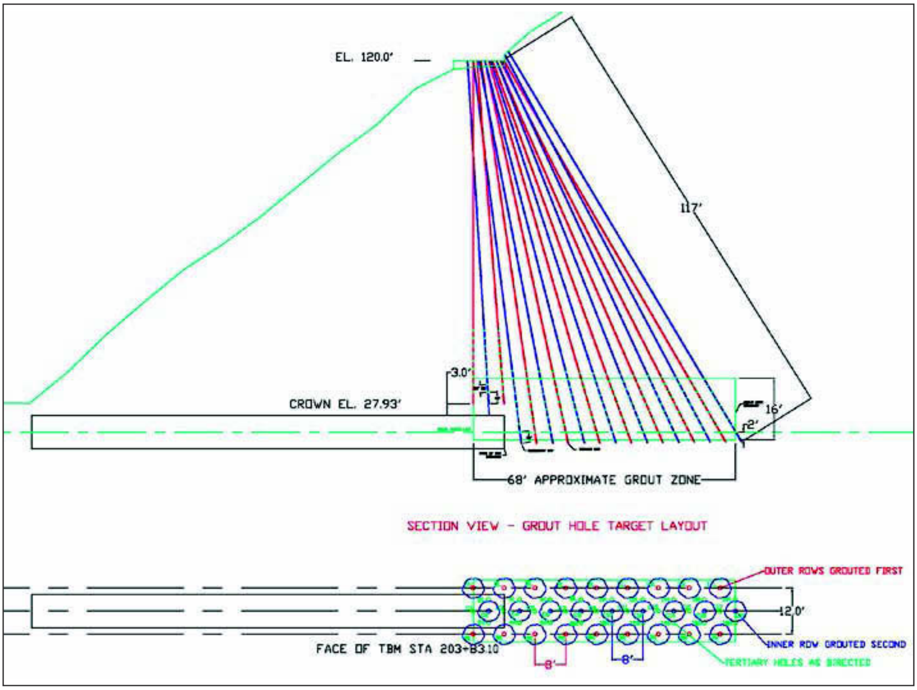

Figure 10. Grout port array to stabilize ground in front of TBM (Harkins, 2012)

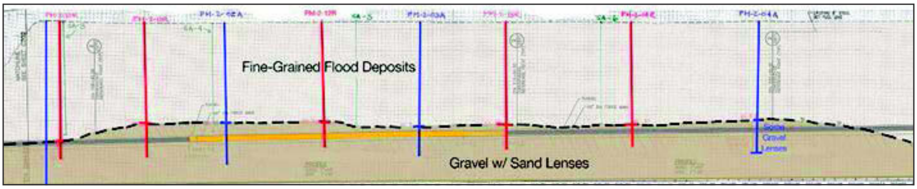

Figure 11. Revised tunnel profile based on additional investigations

Concurrent with the grouting program, additional rotosonic borings were drilled along the south end of the tunnel alignment in order to investigate the nature and lateral extent of the coarse gravel deposits. The results of the additional borings indicated that the gravel deposits and the upper contact with the Troutdale Formation could extend for another 610 m (2,000 ft). It was concluded that boulders could continue to cause excavation difficulties. The revised geologic profile is shown in Figure 11.

TBM EXTRACTION

Several options were considered, including a rescue shaft, retrieval of the TBM through the portal, retrofitting the TBM, changing the tunnel grade, and underground conversion of the TBM into a simple shield. summarizes the options that were considered. Underground conversion and retrofit options were eliminated because of the extensive ground improvement that would be necessary to stabilize the ground for working on the machine and continuing with the TBM. Options that involved a rescue shaft were eliminated because of cost and adverse impacts to the University of Portland, located at the top of the bluff. The selected plan involved TBM retrieval through a jacked steel casing that enveloped the tunnel support system and the TBM. It was decided that the TBM would be replaced with a digger shield of similar diameter. This option would be least cost and could be executed quicker compared to the other options. The option was feasible because the 3,048 mm (120 in.)casing size selected by the contractor was large enough to accommodate an inner casing that was larger than the machine.

Once the tunnel and casing were stripped of all materials and equipment, including the tunnel rails, a 2,997 mm (118 in.) diameter casing was jacked through the 3,048 mm casing and along the outside of the tunnel from the South Portal Shaft. The casing encapsulated the initial ground support and the majority of the TBM. The portion of ribs and lagging that was covered by the new casing was removed at the end of each advance. Since the tunnel began to curve in the area were the TBM was located, the casing joints were designed to "float" using connection tabs to allow the casing to negotiate the curve. Bentonite injection ports were included in the casing for lubrication.

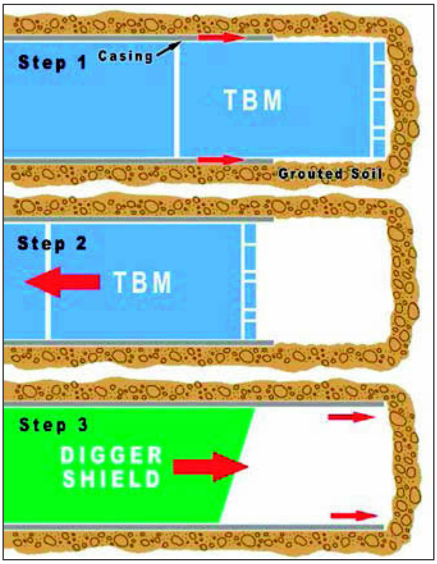

Figure 12. General sequence for TBM removal and digger shield launch

The TBM was placed on rollers and moved to the beginning of the horizontal curve section using the TBM thrust rams to pull the TBM. From there the TBM was removed from the casing using a crane and pulleys to tug on a cable attached to the back of the machine. After one of the TBM rollers broke 10 feet from the portal, the thrust cylinders on the pipe jacking frame was used to complete the extraction.

After the TBM was pulled back from the face, the casing was advanced to within 1 foot of the tunnel face. Figure 12 illustrates the three-step process of casing installation (Step 1), TBM retrieval (Step 2), and casing advancement to the tunnel face followed by the installation of the new digger shield (Step 3). Figure 13 shows the TBM exiting the casing in the South Portal Shaft.

Figure 13. TBM after extraction from casing

DIGGER SHIELD EXCAVATION

Used and new digger shields were considered. The key objective was to find a shield that could fit through the casing and that would be compatible with the steel ribs and lagging support materials already procured for the project. A Herrenknecht DA2640-MH2S tunneling shield was ordered in July 2010. The digger shield was equipped with a hood and an excavator arm and a hydraulic belt conveyor. Upgrade options were also selected, including a sand table, stabilizing fins, and a compact drilling system.

The shield and appurtenant equipment cost around $2.4 million. The digger shield arrived at the project site at the end of October and was underway in December (Figure 14).

Figure 14. Herrenknecht open face digger shield with excavator arm

Initially, the digger shield excavation progressed slowly. Concerns about overexcavation and sinkholes, carried over from the previous TBM experience, caused the operators to implement a procedure that required them to stop mining as soon as running and raveling ground was evident and perform remedial grouting from the face. Although the grout provided temporary relief from ground loss, it also bound up the digger shield, requiring significant efforts—steering and thrust—to break it free and continue mining. This resulted in an extremely low tunnel advance rate. Since running and raveling ground was anticipated throughout the entire alignment, these digger shield operational procedures were not sustainable. The start-stop procedures also made accurately monitoring muck volumes difficult.

Figure 15. Digger shield after hole through at North Shaft

The operational procedures were modified to place more emphasis on forward thrust and controlled muck removal, followed by systematic void filling behind the shield. This change required that the operators become confident in their abilities to operate the digger shield in ground that is not improved with grout. The sinkholes developed quickly because of shallow ground cover and significant overexcavation by the original TBM. In the deeper ground it took several days for settlement to propagate enough to be observed in a multilevel settlement monitoring point after an overexcavation event with the digger shield. This indicated that the material above the tunnel started to effectively bridge/arch and settlement risk could be significantly reduced by quickly filling (within 2 or 3 shifts) the overexcavated annulus or void above the tunnel's initial support. The deeper ground cover and improving ground behavior as the tunnel got into more uniform fine grained flood deposits increased the operators' confidence balancing muck removal and digger shield advance rate. The advance rate improved and settlement was not observed for the remainder of the tunnel.

The contractor used a VMT system to consistently log the location of the digger shield and provide data daily. As a quality control check, the contractor bored a 152 mm (6 in.) hole from the ground surface through the tunnel to ensure the tunnel was on the correct alignment prior to the last curve into the North Shaft for retrieval. The alignment of the tunnel was surveyed using this down hole, and minor adjustments were made to the machine heading prior to reaching the North Shaft. As a result, the machine entered the North Shaft on the correct alignment within the break-in window of the liner plate shaft wall (Figure 15).

The North Shaft was an elliptical shape supported by ring steel and steel lagging. The shaft was constructed over and around the existing Portsmouth Tunnel. The elliptical shape allowed for clearance to remove the tunnel shield without disturbing the exposed existing tunnel and thus eliminated a sewer bypass.

The contractor broke through the North Shaft on April 29, 2011. The advance rate ended up averaging 14.1 m (46.4 ft) per day, even considering the slow start and operating difficulties early in the tunnel drive. The total construction cost for the tunnel, force main, and shaft structures was around $30 million.

CONCLUSIONS

Good communication and partnering with the contractor helped the parties find an innovative solution to resolve the Differing Site Condition encountered during the tunneling. When it became evident that boulders were being encountered well above the conditions described in the GBR and the boulders could not be effectively extracted through the machine face without causing loss of ground in the surrounding soils, the project team agreed that the best way to retrieve the TBM from under the hillside was to grout a zone around the machine, then jack a steel casing into the hillside around the existing tunnel and pull the machine out through the South Portal Shaft. This method proved very effective, and the machine was successfully retrieved and replaced with the digger shield, a more appropriate tunneling machine and method for the boulders encountered. This method was successful because the contractor selected a starter casing with sufficient size to jack an inner casing and because the TBM had only traveled a short distance.

There were several lessons learned on this project. Since boulders were anticipated, it would have been beneficial to provide more specific requirements for the tunnel machine to handle the boulders and/or require to the contractor to submit a specific work plan for pulling boulders through the face in advance of tunneling, including measures for stabilizing the ground around the boulder as it is being removed. This in turn could have assisted with procuring a more appropriate machine for dealing with the boulders or for at least having a plan in place to reduce the time to implement the ground stabilization measures required to advance and ultimately retrieve the machine. Other considerations for projects facing similar ground conditions include the following:

- Rotosonic drilling methods with continuous soil core recovery are effective for characterizing coarse grained deposits, including identifying the size and quantity of cobbles and boulders.
- Select alignment with options for surface grouting.
- Install settlement casings early in drives and after change in ground to monitor for ground loss during the initial tunnel excavation learning curve.
- Consider aggressive cutterheads for EPB tunneling in ground with cobbles.

REFERENCE

Harkins, B.M., and Parmantier, D.M. 2012. Stabilization of running granular soils in TBM alignment with sodium silicate grout. In *Proceedings of the 4th International Conference on Grouting and Deep Mixing*. American Society of Civil Engineers.

PART 5

Future Projects

Chairs

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FREQUENT FLOODING RELIEF IN SIGHT—CHARLESTON'S STORMWATER TUNNEL TO "DRAIN" THE RAIN

Laura Cabiness ▪ City of Charleston

Steve Kirk ▪ City of Charleston

Jason Swartz ▪ Black & Veatch

Stephen O'Connell ▪ Black & Veatch

ABSTRACT

The City of Charleston, South Carolina has been actively engaged the last several years designing and securing funding for the largest infrastructure project in the City's history. Recent Federal and State financial assistance will finally allow the City to move forward with the highly anticipated U.S. 17 Septima Clark Parkway Transportation Infrastructure Reinvestment Project. This project will involve the construction of 8,500 feet of 12 foot diameter stormwater tunnel, nine drop shafts, four large diameter working shafts, high capacity pumping station, river outfall and near surface improvements to provide relief from frequent flooding in the heart of Historic Downtown Charleston. Once constructed, frequent flooding during moderate to heavy rains and high tides will be a thing of the past.

This project specifically addresses how many communities are turning toward innovative tunneling and other underground methods to improve the quality of life for their stakeholders. This paper will provide an overview of the project and specifically address the design behind the proposed tunnel system (145 feet below historic Charleston) and the associated river outfall. In addition, this paper will outline how the City was successful in securing funding to address a problem that is as old as the City itself.

INTRODUCTION AND BACKGROUND

Charleston has deep roots in the historic fabric of the United States. Founded in 1670 and located along the coast of the Atlantic Ocean in the southeastern United States, Charleston was a vital colony in America's early existence. Today many recognize Charleston for its historic significance, unique architecture, Southern charm, and coastal beauty. Charleston is a world class community and was recently voted by the Conde Nast Travel magazine as the number one City in the Country and the Top City in the World. The historic Charleston Peninsula is home to nearly a third of Charleston's 122,000 residents and the nucleus of Charleston's economy.

Since the city of Charleston's inception, stormwater flows have plagued the land. Charleston is located in the heart of what is regionally known as the Lowcountry, a designation which quite aptly describes the landscape of this historic city. Elevations barely above sea level have contributed to frequent flooding, especially when rain events occur at or around high tide levels, see Figure 1.

Charleston was among the first communities in America to begin separating sewer flows from the stormwater flows. This foresight is a credit to the city's forward thinking

Figure 1. Typical flooding in Charleston

founders. As Charleston continued to expand and push outward, the battle with tidal flows became a constant reminder of what has made Charleston great; easy access to major bodies of water. The Charleston peninsula is bounded by the tidally influenced Cooper River to the east, Ashley River to the west and Charleston Harbor to the south. Although Charleston's rivers and inner harbor have facilitated trade and travel, the tidal influences have severely hampered the landscape's natural ability (given low surface elevations) to effectively drain stormwater flows during high tides. The average elevation of the City is only a few feet above sea level with little to no change in topography. Engineers have struggled with these elevation constraints since the City's origin. Gravity systems have especially been difficult to construct as even moderate slopes push the infrastructure at or below the tidal zones.

In 1837 the Mayor of Charleston offered a $100 gold coin to anyone who could come up with a feasible solution to address the constant flooding. The offering of the gold coin spurred many ideas; however, it was quickly realized that there wasn't a simple fix to this problem. In the end no one received the golden coin and the Mayor took portions from some of the best ideas to provide a solution. The solution was to construct a network of interconnected brick arches that discharged stormwater to either the Cooper River or Ashley River on either side of the peninsula. Gates were installed on the outfalls to control the tidal waters. The limited amount of available differential head greatly restricted the system's ability to effectively flow by gravity at moderate to high tides. This facilitated the need for Charleston to divide the peninsula into a series of smaller basins to help limit the length and depth of the gravity runs. The system was also slightly undersized to help facilitate scouring velocities during storm events or high tidal exchanges. The objective was to provide a "self" cleansing system. Unfortunately, the system was never very efficient in conveying the stormwater flows to the rivers, particularly during high tide events. The system provided some minor flood relief but years of siltation, build up of other debris, and the lack of efficient scouring eventually clogged an already undersized system. Furthermore, many of the tidal gates have succumbed to the corrosive attacks of Mother Nature and the harsh marine environment with many of the gates since removed.

THE SOLUTION

In recent years, the City of Charleston has turned to more innovative approaches to address the flooding issues. Charleston Water System (separate entity responsible for water and wastewater service) has long relied on tunneling techniques to provide a reliable water supply and wastewater conveyance. Tunnels and trenchless technology have roots in Charleston dating back to 1928 when a system of water supply tunnels were constructed to bring a new water supply from the Edisto River and Foster Creek to the Hanahan Water Treatment Plant to supplement groundwater sources. Today nearly 50 miles of tunnels have been constructed or designed in and around Charleston for water, sewer and stormwater conveyance.

Tunnels and trenchless techniques have finally provided the City of Charleston (responsible for stormwater service) the relief it has sought to combat its flooding issues. The City of Charleston first implemented tunneling techniques in 1999 on their Meeting Street/Calhoun tunnel. The project was actually designed as a major open cut endeavor to fix the frequent flooding issues in one of the City's many stormwater basins. However, after input from various contractors the project was redesigned following award of the Contract to a deep underground conveyance tunnel. This change greatly minimized the impacts to existing utilities and significantly reduced disruptions to the general public. The tunnel project incorporated a new pump station to discharge the tunnel flows out into the Cooper River, thus providing alternative means to drain the rain whereas otherwise tidal conditions had previous controlled the rate of stormwater removal.

This concept proved highly successful for the City and resulted in similar designs for the next two critical basins plagued by flooding; the Market Street Drainage Improvements Project which is currently under construction and the U.S. 17 Septima Clark Parkway Transportation Infrastructure Reinvestment Project which until recently had been awaiting funding for construction to begin. Additional basins also have conceptual tunnel systems that may be implemented in the future.

THE NEXT PIECE OF THE "PUZZLE"

The U.S. 17 Septima Clark Parkway Transportation Infrastructure Reinvestment Project is the next piece of the "puzzle" to be constructed that will address flooding in two drainage basins locally referred to as the Spring and Fishburne basins, see Figure 2.

This Project is a multi-phased project that has several objectives; to improve the mobility, efficiency, emergency preparedness, and community livability; and, most importantly, to alleviate many of the flooding problems by reinvesting in the infrastructure. The transportation advancements will incorporate safer travel lanes for vehicles; improved intersections for pedestrian safety and vehicle efficiency; Intelligent Transportation Systems (ITS); and new, energy-efficient traffic signals. The infrastructure reinvestment will consist of constructing improved and additional surface collection systems throughout parts of the basins, drilling several shafts from the surface down as much as 150 feet, boring 8,500 feet of 12 foot internal diameter tunnels to connect the shafts, constructing a new pump station near the Ashley River, and constructing an outfall from the pump station to the Ashley River.

The Spring and Fishburne drainage basins incorporate approximately 20% of the land mass on the historic Charleston peninsula and are habitually flooded with a combination of storm and tidal waters. During times of moderate to heavy rainfall within a few hours of high tide, this area becomes impassable to vehicles, oftentimes for hours, cutting off access to vital entities. The list of some of the entities impacted by flooding in these basins include: U.S. Highway 17 (local Hurricane evacuation route), Medical University of South Carolina (the area's only Level 1 trauma center), Roper Hospital, VA Hospital, US Army Corps of Engineers Headquarters, Charleston Police Headquarters and Emergency Response Center, Charleston's Public Housing, Burke High School (Downtown's only high school), Julian Mitchell Elementary School, the Citadel, two fire

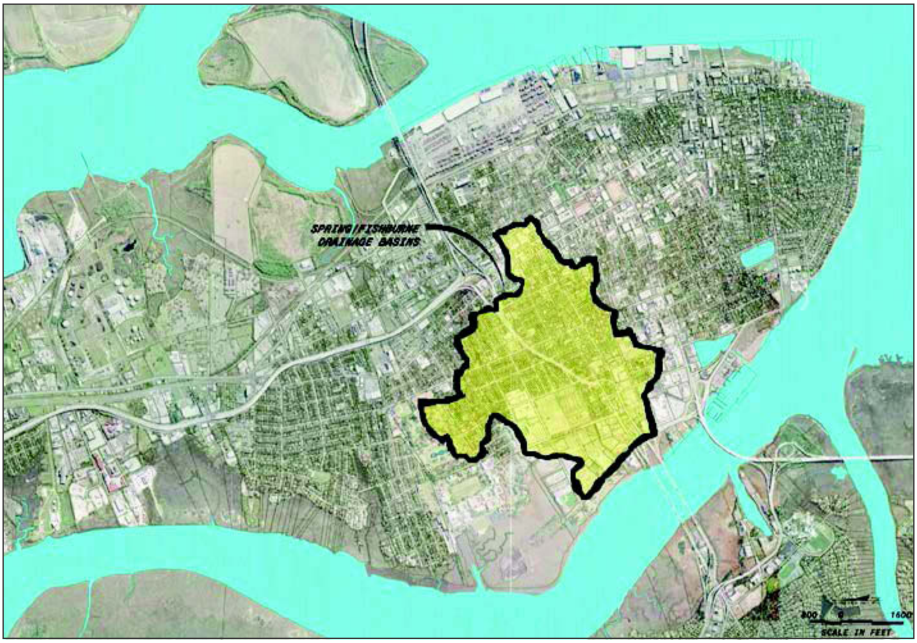

Figure 2. Spring Fishburne drainage basins

stations and the City Gym. Not to mention the countless other businesses, residences and daily travelers that utilize the infrastructure and facilities in these basins.

DESIGN OF THE TUNNELS AND SHAFTS

The geology of the Charleston peninsula is that of an estuary, and as such the shallow deposits (surficial soils) are influenced by a combination of marine and continental processes. The surficial soils were deposited in a wide range of sedimentary facies including fluvial, overbank, tidal marsh, tidal channel, tidal flat, lagoon, beach, barrier island, and shallow marine. Characterized by its extremely low shear strength and high clay content, the surficial soils are susceptible to significant consolidation and settlement over time. As a result, large portions of the peninsula including city streets are slowly subsiding. However, 50–70 feet below these surficial soils is a geologic formation locally referred to as the Cooper Marl. The Cooper Marl is a relatively thick layer (150–200 feet) of olive-green, calcareous, medium to stiff, sandy/clayey silt. It is an excellent engineering medium used extensively for its load bearing and self-supporting attributes. Large buildings are almost exclusively founded on pile supports that extend into the Cooper Marl. The Cooper Marl's strength and standup time can primarily be attributed to the calcareous bonds. However, these bonds are easily broken by standard tunneling methods including hand-mining. The Cooper Marl cannot be accurately classified by the Tunnelman's Ground Classification system as it generally behaves similar to that of a soft rock.

The Cooper Marl is the key to the success of Charleston's vast array of underground tunnels so it was no accident that the tunnels of the U.S. 17 Septima Clark Parkway Transportation Infrastructure Reinvestment Project were designed in this same formation.

The tunnel system was designed to transport the stormwater flows from centralized low points within the Spring and Fishburne basins via drop shafts to the Ashley

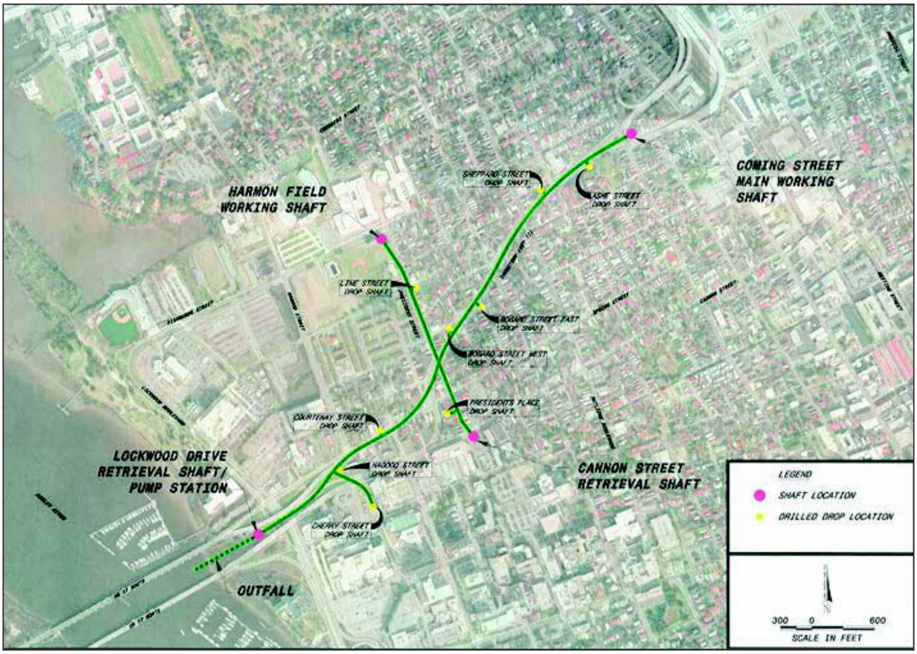

Figure 3. Project alignment

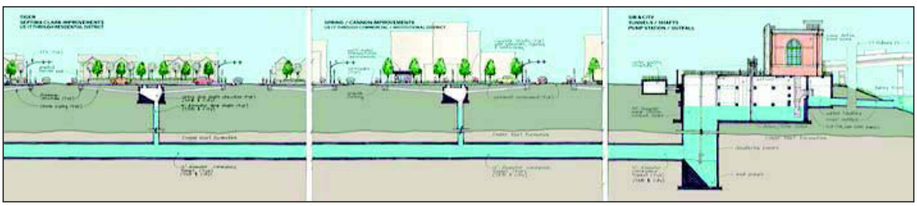

Figure 4. Schematic of drainage improvement (Courtesy of Davis & Floyd)

River. The tunnel was designed to a depth of 120 to 150 feet below the ground surface to ensure adequate clearance was maintained from the surficial soils and the existing building piles, bridge piers and existing wastewater tunnels in the project vicinity. The system will be comprised of 8,500 feet of cast-in-place 6 to 12 foot finished diameter concrete lined tunnel, 4 large diameter working shafts (20–30 feet ID) and 9 drop shafts (48–54 inch ID), see Figure 3 and 4.

Existing wastewater tunnels within the horizontal tunnel alignment determined the final vertical alignment of the tunnels. A minimum clearance of 20 vertical feet was maintained between the invert of existing wastewater tunnels and the crown of the proposed stormwater tunnels. The tunnel was sized so that the flooding from a 10 year storm event could be effectively removed from surface streets and conveyed through the tunnel and ultimately pumped into the Ashley River. The tunnel design assumed construction by means of a soft ground Tunnel Boring Machine (TBM) for the main tunnel reaches with roadheader and/or hand mining methods being utilized to construct the smaller tunnel adits connecting to the drop shafts. Support of the tunnel specified steel ribs and timber lagging for temporary support during construction with a final lining of cast-in-place concrete. Finished diameters of the tunnel conveyance system will range from 6 feet in the branch tunnels, and 8 to 12 feet in the main spines.

The location of the four main working and nine drop shafts were determined by hydraulic boundary conditions, disturbances to traffic and utilities, overall constructability and site locality. The main working shafts have been designed to be constructed by the caisson method. Construction by caisson method involves assembling lifts of reinforced concrete above grade as removal of soils is ongoing at the excavation floor inside the caisson. The caisson is then left to sink under its own weight and the process is repeated until final invert depth is reached. Typically excavation of material within the shaft is done in the wet through the surficial soils, meaning the shaft is full of water to balance hydrostatic loadings until the caisson is socketed into the marl, where the remainder of the shaft can be excavated in the dry. This method is ideally suited for minimizing impacts to nearby structures.

Drop shafts are anticipated to be constructed utilizing a vertical auger drilling method in the wet. A vertical drill rig with helical augers and a temporary steel casing is installed as the drill progresses through the soil to final invert depth. The process is completed in the wet to aid in maintaining wall stability and then a welded steel pipe with epoxy coating will be installed to form the final lining of the drop shafts. On the top of each drop shaft a vortex structure will be constructed to improve flow characteristics and limit the amount of air entrainment into the deep tunnels.

DESIGN OF THE OUTFALL

Selecting an appropriate outfall location along the western edge of the peninsula proved to be very challenging with several alternatives analyzed. The most appropriate location was determined to be between the northbound and southbound lanes of the U.S. Highway 17 bridges crossing the Ashley River. However, this provided additional challenges in that two different marinas existed on either side of the bridges. The design had to demonstrate that the outfall location would not scour the existing bridge piers while also not contributing to additional siltation of the marinas.

Detailed bathymetric surveys and a comprehensive hydrodynamic model were developed to aid in the analysis. This information was used to create both a 2D grid model and a Computational Fluid Dynamic (CFD) model. The model was run at both low and high tide conditions with all three pumps of the proposed pump station running to establish a worst case scenario. Each box culvert section is dedicated to one of the three pumps at a new pump station and is capable of handling flow up to the maximum possible output of 120,000 gpm. Sizing of the outfall was determined by the peak pumping capacity of the pumps, limiting the discharge velocities to within already natural velocity ranges occurring with each tide cycle. The outfall was also designed so the structure will be completely submerged at the Lower Low Level Water (LLLW) mark. This ensures the outlet of the outfall will be submerged at all times and help to control the discharge flow.

The models determined that the addition of an outfall to the Ashley River would have very minimal impacts to the bridge piers and marinas. The outfall design consisted of 8×10 foot box culverts extending from the proposed pump station 450 feet and into the Ashley River another 100 feet. To further dissipate the discharge velocities and reduce scouring of the river bottom, an enlarged box culvert section and a mat of riprap and gabion mattresses was designed at the terminal point of the outfall. The outfall will be supported on prestressed concrete piling and constructed entirely below grade.

Following design of the system, a physical hydraulic model was constructed by Clemson Engineering Hydraulics at a 1:11 scale, see Figure 5. The physical model was used to verify design assumptions and allowed the design team to run various flow scenarios through the model. The model also helped the design team evaluate the

Figure 5. Physical scale model of outfall and pump station

impacts of air entrainment in the outfall. Based on the physical model, modifications to the system were made and an air vent installed just downstream of the pumps for added air release. These modifications reduced the likelihood of massive air releases in the Ashley River and unnecessary reductions in hydraulic capacity.

FROM DREAM TO REALITY

The City of Charleston currently has two main sources of revenue for addressing the many stormwater challenges it faces. The first is a property tax levy set aside for ongoing City wide stormwater management and operations. The second is a stormwater utility fee based on the amount of impervious area associated with a particular property.

Approximately 58% of the revenue generated from these two sources goes to city-wide stormwater management, repair and operation expenses. The remaining funds are set aside for long-term maintenance, capital improvements, design services, and permitting for the stormwater system.

The total estimated construction cost for the U.S. 17 Septima Clark Parkway Transportation Infrastructure Reinvestment Project is $146.3 million. The cost magnitude of this project is more than the residents and rate payers of Charleston can manage alone. With the City's mean household income at 84% of the national average and 95% of the state average, the City is not in a position to add a significant tax or fee increase. If the City attempted to fund the entire Project through debt financing, the annual interest and principle payments would run about $8 million/year (based on issuing a 20 year, 5.00%, $145 million principle municipal bond). This payment would require the City to increase its city-wide stormwater revenues by more than 2½ times.

Even before the design efforts began, the City of Charleston had been looking for alternate funding sources for this project. The City's objective was to receive funding from as many sources as possible to supplement the little money already available. The City used a large portion of their available funding to pay for the design efforts of the project with anticipation that construction funding would eventually be secured.

Design efforts began in 2007 and were nearing completion by 2009. As the design completion neared, the national economy was attempting to avoid a national depression and various "stimulus" packages were being developed by the Federal Government. The City of Charleston found themselves with a "shovel ready" project but lacking funding. A program known as the Transportation Investment Generating Economy Recovery (TIGER) program was rolled out as part of the American Recovery and Reinvestment Act (ARRA) and the City of Charleston jumped into action preparing a thorough application and detailed Benefit Cost Analysis (BCA). The application sought full funding of the project from the $1.5 billion available through the program.

The grant was to be awarded on a competitive basis to projects that demonstrated an ability to provide a significant impact on the nation, region, or metropolitan area. The City of Charleston was ultimately successful in securing $10 million through this program and was one of only 51 projects (out of 1,380 applications) to receive funding through this program.

The TIGER Grant allowed the City to begin portions of the much needed U.S. 17 Septima Clark Parkway Transportation Infrastructure Reinvestment Project. However, local leaders continued the search for additional funding opportunities. In 2012 the City of Charleston was approved for a $25 million SCDOT matching grant to continue the improvements along U.S. 17 that were started with the funds received from the TIGER grant program. The City of Charleston committed $12.5 million through this program with the SCDOT providing the matching grant. Also in 2012, the City of Charleston was notified that the State Infrastructure Bank had voted unanimously to provide another $88 million toward this project to begin construction of the tunnels, shafts, outfall and pump station.

Repeated requests to legislators and many long hours completing applications had finally paid off and the City of Charleston now has the funds to continue the project. The current schedule has the City completing the current phase along US 17 and then turning its attention to the tunnels, shafts and outfall. The deep underground portion of the project is tentatively scheduled to be bid in early 2014 with contractor correspondence and prequalification updates beginning later this year.

Communities across the country are faced with a similar reality of implementing large scale infrastructure projects in urban environments, many of which require large conveyance systems. Charleston is a showcase for the increasing viability of underground construction techniques and other communities can benefit from the City's foresight and ingenuity. Charleston has demonstrated that determination and strong leadership do eventually pay off.

ACKNOWLEDGMENTS

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DC CLEAN RIVERS PROJECT NORTHEAST BOUNDARY TUNNEL AND NORTHEAST BOUNDARY BRANCH TUNNEL PROJECT OVERVIEWS

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ABSTRACT

The District of Colombia Water and Sewer Authority (DC Water) is implementing the $2.6 billion Clean Rivers Project consisting of six deep storage and conveyance tunnels to control combined sewer overflows (CSOs) to the Anacostia and Potomac rivers and Rock Creek and to meet the requirements of a Federal Consent Decree. To date, the first two tunnels, Blue Plains (BPT) and Anacostia River (ART), are in construction and procurement, respectively. This paper discusses two subsequent tunnels for the Anacostia River watershed, the Northeast Boundary Tunnel (NEBT) and the First Street Tunnel (FST). Besides providing additional CSO storage, these tunnels will alleviate chronic surface flooding in the highly congested residential areas of Northeast and Northwest Washington DC. This paper focuses on design considerations in the selection of tunnel alignments, shaft locations, and diversion facilities, as well as the challenges anticipated during construction. The projects will be procured using the Design-Build method with a Notice to Proceed date of late 2013 for the FST and 2016 for the NEBT.

BACKGROUND

The North East Boundary Tunnel (NEBT) and First Street Tunnel (FST) are the final two of the four large-diameter tunnels that make up the Anacostia River Project (ARP) portion of the District of Columbia Water and Sewer Authority's (DC Water) Long Term Control Plan (LTCP) (Figure 1). The LTCP is required by a consent decree signed by the US Environmental Protection Agency, the District of Columbia, and DC Water and is designed to reduce combined sewer overflows (CSOs) into the Potomac and Anacostia rivers and Rock Creek as well as to reduce flooding in flood-prone areas of the District. The first two tunnels of the ARP, the Blue Plains Tunnel (BPT) and the Anacostia River Tunnel (ART), are required to be in operation by March 23, 2018. The second two tunnels, the NEBT and FST, are required to be in operation by March 23, 2025. After the ARP is fully online in 2025, overflows into the Anacostia River will be reduced by 98%.

PREVIOUS CONTRACTS

The first two large-diameter tunnels, BPT and the ART, are 7 m (23 ft) internal diameter tunnels that will be constructed approximately 30 m (100 ft) below grade. The BPT will be 7,407 m long (24,300 ft) and will be constructed between the Blue Plains Advanced Wastewater Treatment Plant (BPAWWTP), at the southern tip of DC, and the DC Water Main Pumping Station, near Washington Nationals Park. The ART will be 3,810 m long (12,500 ft) and constructed between Poplar Point, near the South Capitol

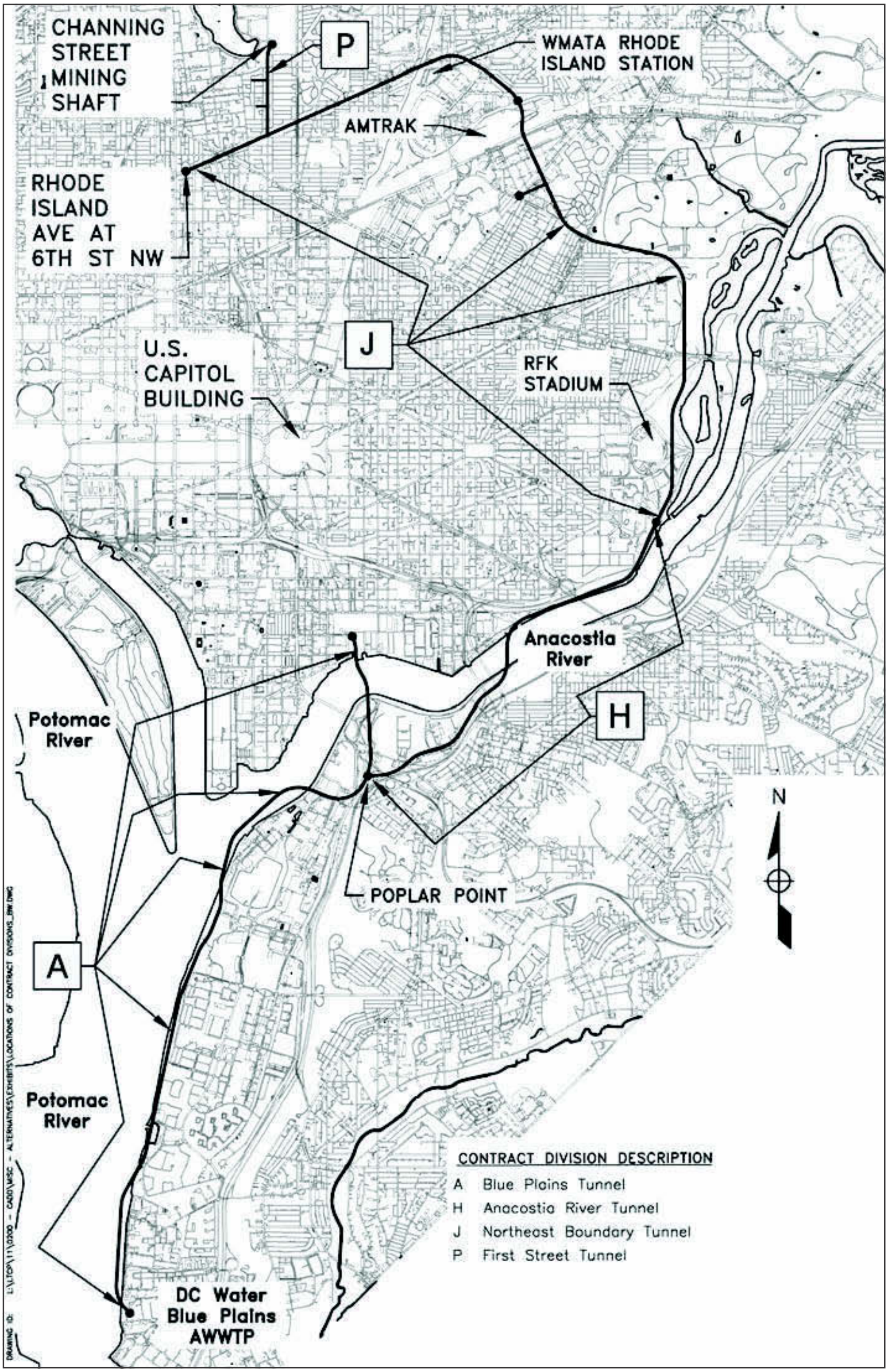

Figure 1. Location of DC Water's long term control plan contract divisions

Street/Anacostia Freeway Interchange, where it joins the BPT and RFK Stadium. Both the BPT and the ART, and their associated shafts and diversion structures, are primarily constructed on land owned by the District Department of Transportation (DDOT), DC Water, the National Park Service, or Joint Base Anacostia-Bolling (JBAB). The design and construction of the BPT and ART required coordination primarily with private third parties and not residential communities.

NORTH EAST BOUNDARY TUNNEL

The NEBT is planned to be a 7 m (23 ft) ID tunnel that will be constructed between RFK Stadium, the endpoint of the ART, and the intersection of Rhode Island Avenue NW and R Street NW. The NEBT will be approximately 8,108 m long (26,600 ft), with cover varying from 15 to 49 m (50–160 ft). Ancillary structures include four drop shafts and diversion structures, one maintenance and ventilation shaft (MVS), and one underground tunnel junction. The southernmost third of the NEBT will be constructed primarily under land owned by the National Park Service and the National Arboretum. The remaining two thirds of the NEBT will be constructed in medium-density residential neighborhoods along public rights-of-way. It is anticipated that the design process for the NEBT will require increased community outreach and coordination and increased structure protection analysis as compared to the BPT and ART. The NEBT will cross under two Washington Metropolitan Area Transit Authority (WMATA) aerial metro tracks, six CSXT/Amtrak tracks, and an Amtrak maintenance facility. Numerous water and sewer utilities of the 1880 to 1930 vintage are anticipated to required analysis prior to construction.

Utilities

It is anticipated that extensive coordination with utility companies will occur during the design and construction of the NEBT. The NEBT runs parallel to a six-lane street, Rhode Island Avenue, for approximately 2,862 m (8,800 ft). It is anticipated that the majority of the utility company coordination will focus on the water and sewer utilities that run parallel to, or cross, the alignment. The water and sewer lines in Northwest DC were typically constructed in the late 19th and early 20th centuries.

The Bryant Street Pump Station, a large potable water distribution center, is located near the terminus of the NEBT alignment, and several large-diameter, 914 to 1,219 mm diameter (36–48 in.) steel water mains exit the Bryant Street Pump Station and cross the alignment. The cover at the crossing is approximately 15 to 24 m (50–80 ft), or about two to three tunnel diameters. The water mains were originally constructed between 1880 and 1910, and some portions were rehabilitated in the 1950s.

Numerous sewers from the late 19th century run parallel to and cross the alignment. The sewer lines are typically brick or unreinforced concrete and are up to 2.6 m (8.5 ft) in diameter. The cover at the sewer line crossing is 15 to 24 m (50 to 80 ft), or about two to three tunnel diameters.

Large-diameter gas utilities are not anticipated to run parallel to or cross the alignment. Gas utilities near the alignment are anticipated to be small-diameter service or local distribution lines, with diameters of less than 305 mm (12 in.). The gas utilities are also anticipated to be located away from the tunnel centerline.

Power, phone, cable, and fiberoptic telecommunication utilities are known to exist in the area. Based on previous experience in the DC area, coordination with the power and telecommunication utility owners is not anticipated to be significant unless utility relocations are required at the near surface structure locations. Typically, power and telecommunication lines are able to accommodate a magnitude of displacement due to ground movements that exceeds that displacement amount allowed in the contract documents.

Nonutility Third Parties

The nonutility third parties with which the NEBT project will be required to coordinate during design and construction are primarily the District of Columbia and several railroad companies. The railroad companies include CSXT, Amtrak, and WMATA. The NEBT crosses WMATA lines at two locations and rail lines at two locations. The first WMATA crossing is located just north of RFK Stadium, where the NEBT will pass between two deep foundations for WMATA aerial piers. The next set of tracks is located just north of New York Avenue and includes four high-speed tracks used for CSXT and Amtrak service and an Amtrak maintenance facility. At the Amtrak maintenance facility, the NEBT will cross under approximately 16 low-speed tracks.

According to the Amtrak Union Station Master Plan (July 2012), Amtrak plans on constructing a tunnel from Union Station to the Amtrak maintenance facility in the Ivy City neighborhood of DC. It is anticipated that construction of the Amtrak tunnel will occur during or after construction of the NEBT. Therefore, it is anticipated that the design and construction of the two tunnels, NEBT and Amtrak, will require significant coordination between DC Water and Amtrak.

Just north of the Amtrak maintenance facility, a 17 m diameter (55 ft), 46 m deep (150 ft) maintenance and ventilation shaft for the NEBT will be constructed on land owned and operated by the District Department of Public works. The shaft is located at approximately the halfway point on the NEBT and will be used as a safe haven for maintenance on the tunnel boring machine (TBM). Its long-term use will be as a ventilation shaft to vent air when the tunnel is being filled during storm events.

Two more sets of railroad tracks are crossed when the tunnel runs parallel to Rhode Island Avenue NE. The first set of tracks are aerial WMATA tracks. The aerial piers that support the tracks are on 24.4 to 27.4 m (80 to 90 ft) centers and are shallowly founded. The NEBT crosses the aerial structure obliquely, and the tunnel centerline is within 7.6 m (25 ft) of the closest pier. The second set of tracks is owned by CSXT and passes over Rhode Island Avenue on a steel span with a central pier line. The foundation of the central pier line is unknown at this time and is located approximately 7.6 m (25 ft) from the tunnel centerline.

FIRST STREET TUNNEL (DIVISION P)

The FST will be designed under an emergency schedule to reduce flooding of the DC Bloomingdale neighborhood. During the summer of 2012, the Bloomingdale and LeDroit Park neighborhoods of Northwest DC experienced three major flood events. The flood events were caused by rainstorms that surcharged storm sewers and flooded street and basements. As a result of these flood events, the DC Mayor, Vincent Gray, established a task force to determine how to prevent future flood events. The recommendation of the task force was to accelerate the schedule of the FST, which was originally planned to be brought into service by 2025. The accelerated schedule requires that the FST be in service in 2016. Between completion of FST construction and completion of NEBT construction, the FST will be used as a storage tunnel to temporarily store storm water before it is pumped out into existing sewers. After completion of the NEBT, the FST will be used as a conveyance tunnel to convey flows south to the BPAWWTP.

The FST is planned to be a 5.8 m internal diameter (19 ft) to 8.8 m (29 ft) outside diameter tunnel, to be constructed between Rhode Island Avenue NW and Channing Street NW (Figure 2). The FST will be constructed under approximately 853 m (2,800 ft) of First Street NW, with cover varying from 24 to 49 m (80–160 ft). The variable diameter is designed to allow the Design-Build contractor flexibility in TBM selection. The

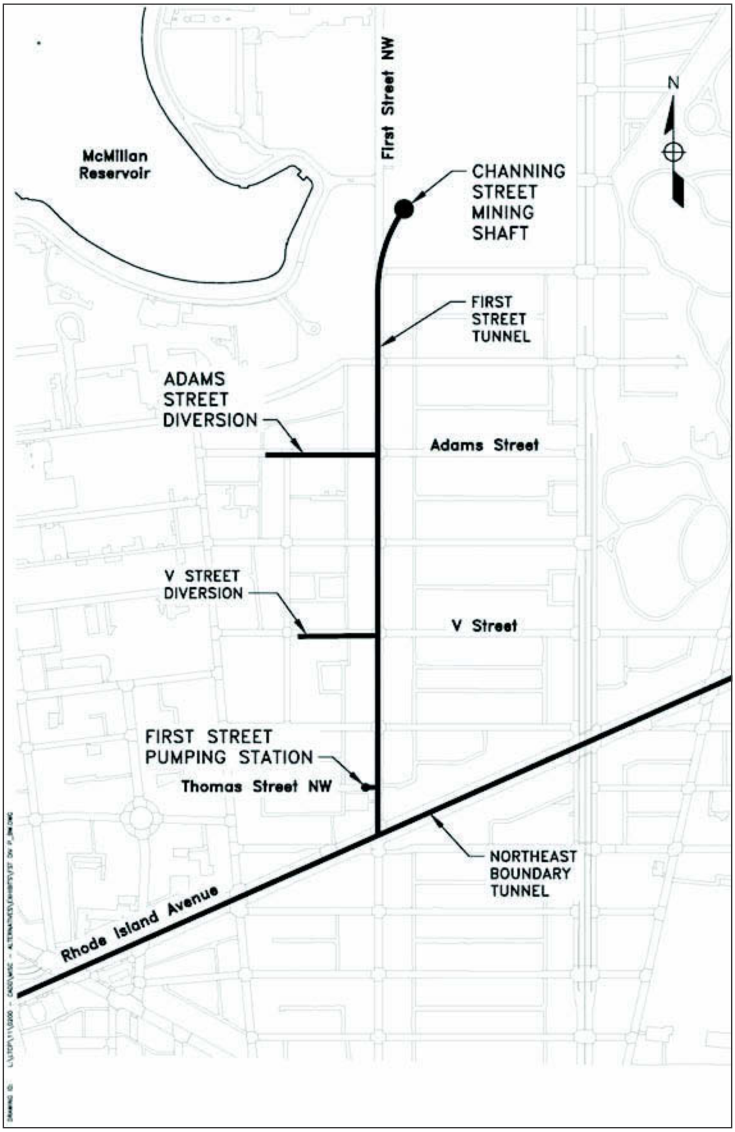

Figure 2. First Street Tunnel location plan

minimum tunnel diameter is controlled by hydraulic design criteria, and the maximum tunnel diameter is controlled by settlement concerns.

Protection of Structures

Approximately 120 sewer and 120 water lines are located in the FST project area. The majority of the wet utilities are less than 0.6 m (2 ft) in diameter, but approximately 20 of the sewer and 20 of the water lines are greater than 0.6 m (2 ft) in diameter. The majority of the structural impact analyses performed for the FST will focus on the large-diameter utilities.

The FST will build on the structural impact analysis approach developed on the ART and BPT. The ART and BPT specified a maximum volume loss above the crown of the tunnel, and the settlement trough width was estimated using charts developed by Peck (1969) and Cording and Hansmire (1975). Damage to utilities and structures was estimated using a combination of Boscardin and Cording (1989), Mair et al. (1996), Bracegirdle and Mair (1996), and Vorster et al. (2005). The estimated damage considered includes joint pull apart, horizontal strain, angular strain, bending strain, bending moment, joint rotation, differential movement, and segment slope.

It is believed that the settlement trough width estimated by Peck and Cording and Hansmire is overly conservative for the volume loss, type of tunneling, and depth of cover that Division P will be constructed under. To better estimate the settlement trough width for Division P, a numerical analysis model was developed in FLAC2D 5.0 (Itasca, 2005). The geologic stratigraphy is idealized as horizontal beds. The elastic material model is used, and a FLAC script was written that modifies the shear and bulk moduli of each zone of the model based on the shear strain of that zone. The shear modulus was reduced using shear modulus reduction curves developed for seismic analyses. An example of the shear modulus degradation ratios, after tunnel excavation and at equilibrium, is presented in Figure 3. The Poisson ratio was kept constant, and the bulk modulus was calculated from the shear modulus and the Poisson ratio. The initial and maximum shear and bulk moduli of the model were estimated by converting the shear wave velocity of the medium.

The nonlinear modulus model described above resulted in trough widths between those estimated by Peck and Cording and Hansmire and those estimated by a FLAC model that does not consider modulus degradation. It is believed that the empirical trough width is too conservative and the constant modulus numerical model trough width is not conservative enough. Therefore, it is believed that the nonlinear FLAC model will estimate a conservative, but not overly so, trough width.

The general approach to protection of structures for Division P will be:

1. Specify maximum volume loss above tunnel crown (0.5%).
2. Develop FLAC model at major utilities and changes in stratigraphy.
3. Fit empirical curve to FLAC settlement.
4. Analyze specific or typical structure and utility properties to determine anticipated damage.
5. If structure or utility will be damaged, recommend mitigation or postconstruction repair.

CONCLUSIONS

The challenges presented by the constraints of urban tunneling for these two upcoming tunnel projects have required the designers to perform a more detailed analysis and design than would typically be required to produce RFP drawings for the Design-Build contractors. The added value of the detailed design has allowed DC Water to better understand the potential risks and to identify potential construction difficulties for these urban tunnels. The Division J Northeast Boundary Tunnel and Division P First Street Tunnel contract will be procured using the Design-Build contracting method previously employed by DC Water on the Division A Blue Plains Tunnel and Division H Anacostia River Tunnel contracts.

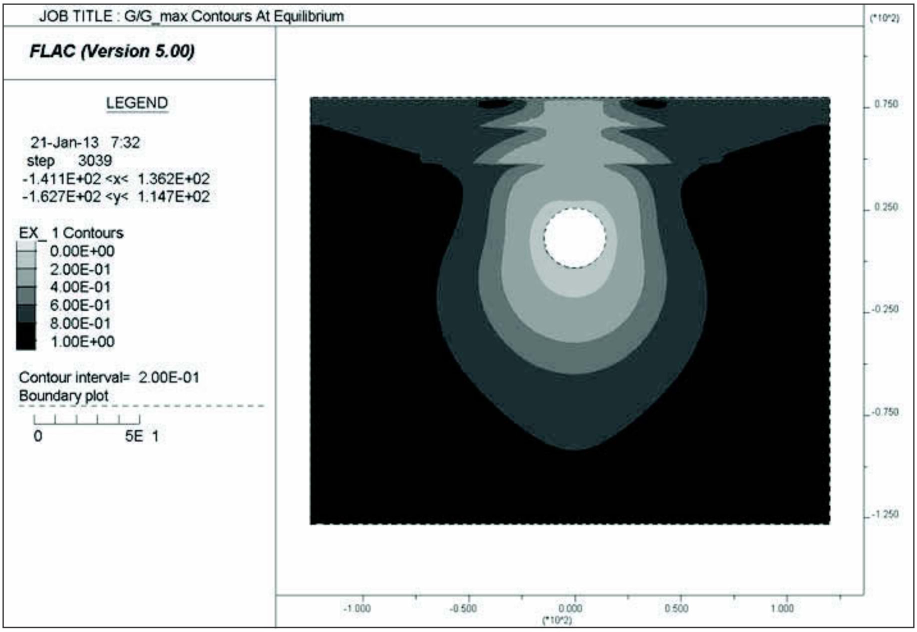

Figure 3. Shear modulus degradation contours at equilibrium

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SANITATION DISTRICTS OF LOS ANGELES COUNTY CLEARWATER PROGRAM—EFFLUENT OUTFALL TUNNEL

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ABSTRACT

In January 2012, the Sanitation Districts of Los Angeles County released a comprehensive, long-range draft Master Facilities Plan as part of its Clearwater Program. A major focus of the planning effort was assessing the need to provide sufficient capacity and redundancy for the existing 8- and 12-foot diameter Joint Water Pollution Control Plant effluent management tunnels that were built in 1937 and 1948, respectively. The recommended project is to construct an approximately 7-mile long, 18-foot internal diameter onshore tunnel that will tie into four existing ocean outfalls, ranging from 60- to 120-inches in diameter. The new tunnel will allow the existing tunnels to be inspected and will provide additional capacity and redundancy. This paper will discuss the preliminary design for this critical infrastructure project.

INTRODUCTION

The Sanitation Districts of Los Angeles County (Sanitation Districts) are 23 independent special districts serving approximately 5.3 million residents in Los Angeles County. Seventeen of the districts that furnish sewerage services to the metropolitan Los Angeles area are signatory to a Joint Outfall Agreement that provides for a regional, interconnected system of facilities known as the Joint Outfall System (JOS). The JOS service area is shown in Figure 1. The JOS serves an area that encompasses 73 cities as well as unincorporated territory and parts of the City of Los Angeles. The JOS provides wastewater collection, treatment, reuse, and disposal for residential, commercial, and industrial users, and it includes seven treatment plants, the largest of which is the Joint Water Pollution Control Plant (JWPCP), located in the City of Carson. Currently, secondary treated effluent is conveyed through two 6-mile long parallel tunnels, 8- and 12-feet in diameter, from the JWPCP to a manifold structure located at Royal Palms Beach, near White Point on the Palos Verdes (PV) Peninsula, from which four seafloor outfalls extend offshore. The two main outfalls, 90- and 120-inches in diameter, extend approximately 1.5 miles offshore and discharge at a depth of approximately 200 feet below sea level. The other two outfalls, 60- and 72-inches in diameter, are used for additional capacity during heavy rain events and extend a shorter distance offshore and discharge at shallower depths. A schematic of the existing system is shown in Figure 2. Both tunnels and main outfalls are required to be in service at all times.

The Clearwater Program is a comprehensive planning effort undertaken by the Sanitation Districts to develop a Master Facilities Plan (MFP) and accompanying

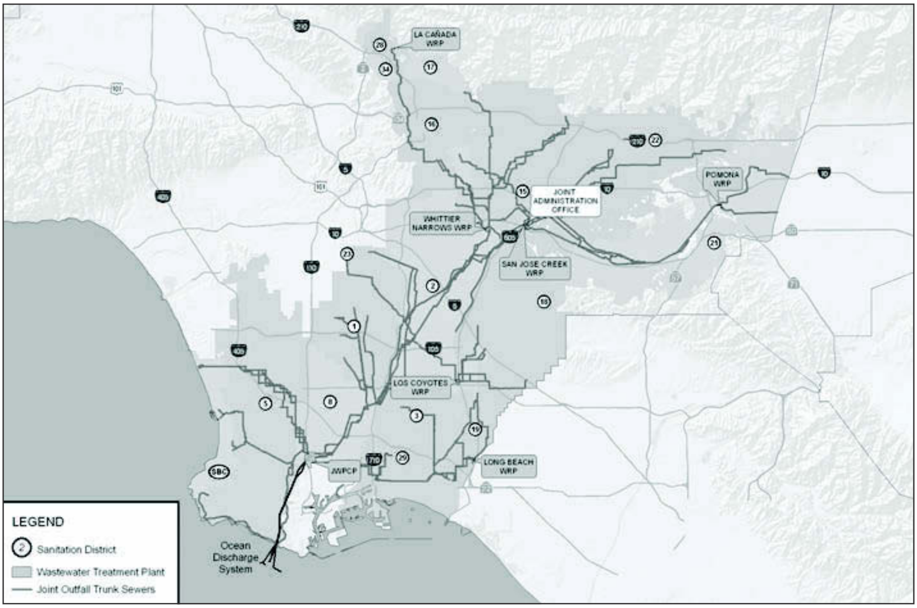

Figure 1. Sanitation Districts joint outfall system service area

Environmental Impact Report/Environmental Impact Statement (EIR/EIS) documentation to guide the management and development of the JOS through the year 2050. One objective of the Clearwater Program is to provide overall system reliability by allowing for the inspection, maintenance, repair and replacement of aging infrastructure. The 8-foot tunnel was constructed in 1937 and the 12-foot tunnel in 1958. Because both tunnels are always in service and flow full every day, neither has been inspected for over 50 years. Another objective of the Clearwater Program is to ensure there is sufficient capacity within the JOS to meet the needs of future population growth. In January 1995, the JOS service area was inundated by two major back-to-back storm events. The resulting peak wastewater flows in the sewerage system from these storm events nearly exceeded the capacity of the JWPCP tunnel and ocean outfall system.

A multi-step, program-wide screening process was conducted and then followed by a project-specific alternatives analysis, which evaluated over 50 potential alternatives and determined four highest ranked feasible alternatives. These four viable alternatives were then carried forward for detailed environmental analysis in the EIR. The EIR was released for public review in January 2012. Based on the detailed review of the four feasible alternatives, the recommended project is to construct a new 18-foot diameter, approximately 7-mile long, on-shore tunnel from the JWPCP to the existing White Point manifold structure at Royal Palms Beach. The proposed tunnel alignment is shown in Figure 3. The new tunnel, when connected with the existing ocean outfalls, would have adequate capacity to accommodate the peak wastewater flows projected for the year 2050 and would enable the Sanitation Districts to inspect and repair the existing tunnels, if necessary.

TUNNEL DIAMETER DETERMINATION

In the MFP, the average flow at the JWPCP is projected to be 400 MGD in the year 2050 and the associated wet weather flow is 927 MGD. The current JWPCP tunnel and ocean outfall system has a maximum capacity of approximately 675 million gallons per

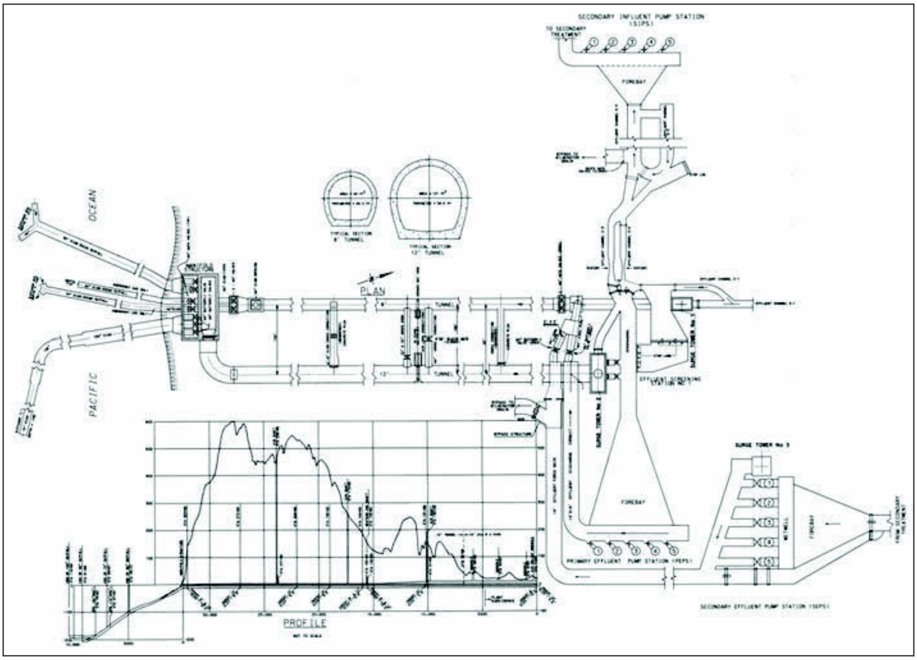

Figure 2. Schematic of existing tunnel and outfall system

day (MGD). The system capacity is limited by the maximum internal pressure the tunnels can handle at the JWPCP. The maximum internal pressure the outfalls can handle is greater than that of the existing tunnels and the combined maximum capacity of all the outfalls together is greater than the 927 MGD projected storm flow. A condition assessment of the existing outfalls was conducted during Clearwater Program analysis and found the pipes to be in excellent condition. If necessary, rehabilitation of the existing outfalls could be performed in the future to extend their remaining service life well beyond the 2050 planning horizon.

In selecting the diameter of the new tunnel, variables such as constructability, the present day and future hydraulic performance of the system, and operational and construction costs were analyzed and balanced against each other. For example, a smaller tunnel would perform better with present day flows and cost less to construct, but it would require more pumping to handle future flows and would increase operational cost. Conversely, a larger tunnel would convey the present day and future flows with less pumping, but the construction cost would be significantly greater. Diameters ranging from 14- to 22-feet were analyzed. Ultimately, it was determined that an 18-foot internal diameter tunnel gave the best balance between present day and future hydraulic performance while being the most cost effective.

TUNNEL BORING MACHINE (TBM) SELECTION

The 18-foot internal diameter tunnel will be built using either an earth pressure balance (EPB), slurry pressure balance (Slurry), or a hybrid TBM. The outside diameter is expected to be approximately 21-feet in diameter, but will be refined after the precast concrete segmented liner design is complete. These machines are considered the only suitable means for underground excavation given the expected ground conditions along the alignment and for meeting other project requirements. The choice between

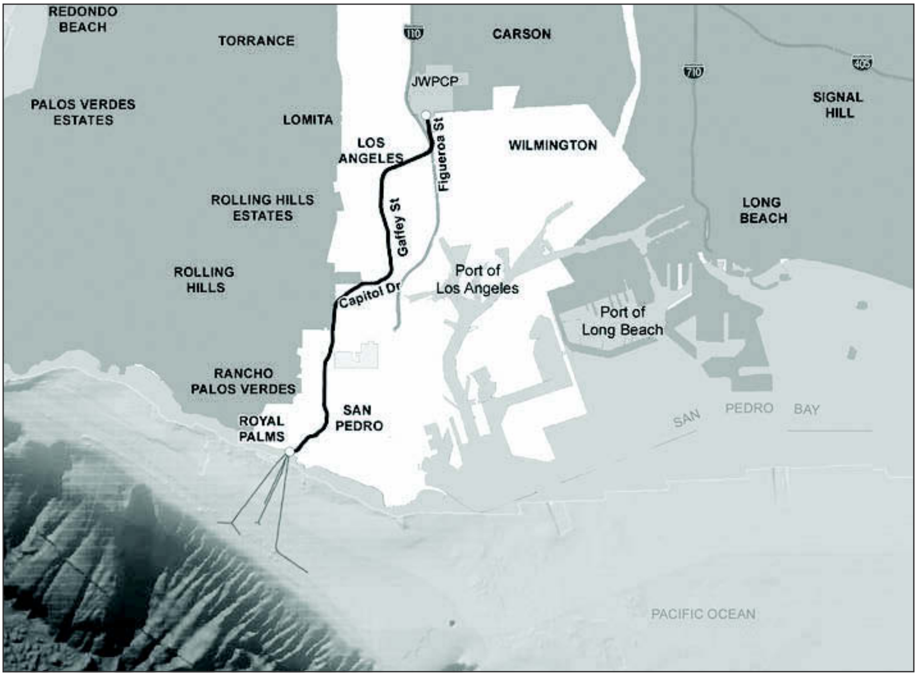

Figure 3. Recommended tunnel alignment

the three types of TBM's is influenced by several factors, including grain size distribution; soil and rock strength; hazardous gases; and the feasibility of soil separation and muck disposal.

The geological profile along the proposed tunnel alignment is shown in Figure 4. Along the alignment there are two distinct geological types of material the TBM will encounter, soil and rock. The northern part of the alignment will be located within Quaternary-aged deposits that include Holocene sediments consisting of fill, alluvium, and terrace deposits. These are underlain by Pleistocene sediments which include the Lakewood Formation and the San Pedro Formation. Both formations are primarily consolidated sediments and include aquifers which will have an impact on the selection of the machine type. The southern part of the alignment within the PV Hill will be located in rock-like material that includes Miocene-age Malaga Mudstone, Altamira Shale, and possible Miocene Volcanic rocks, San Onofre Breccia and possibly the Catalina Schist. The materials are anticipated to exhibit a range of ground behaviors, from soil-like or weak rock-like to raveling or squeezing ground conditions. Also, interbedded volcanic intrusive and extrusive beds as well as dolomite beds are expected which can exhibit strong rock properties. Hydrocarbons and hydrogen sulfide may also be encountered. With a single heading proposed, selecting a TBM that can accommodate both soil and rock will require additional geotechnical investigations to better define underground conditions.

Cutterhead Selection

Regardless of which type of TBM is used, a bi-rotational cutterhead equipped with cutting tools to remove the ground will be utilized. A mixed ground cutter head will most likely be designed given the mixture of soft ground and weak rock. Back loading saddles or cutter boxes that allow the use of either disk cutters or rippers will most likely

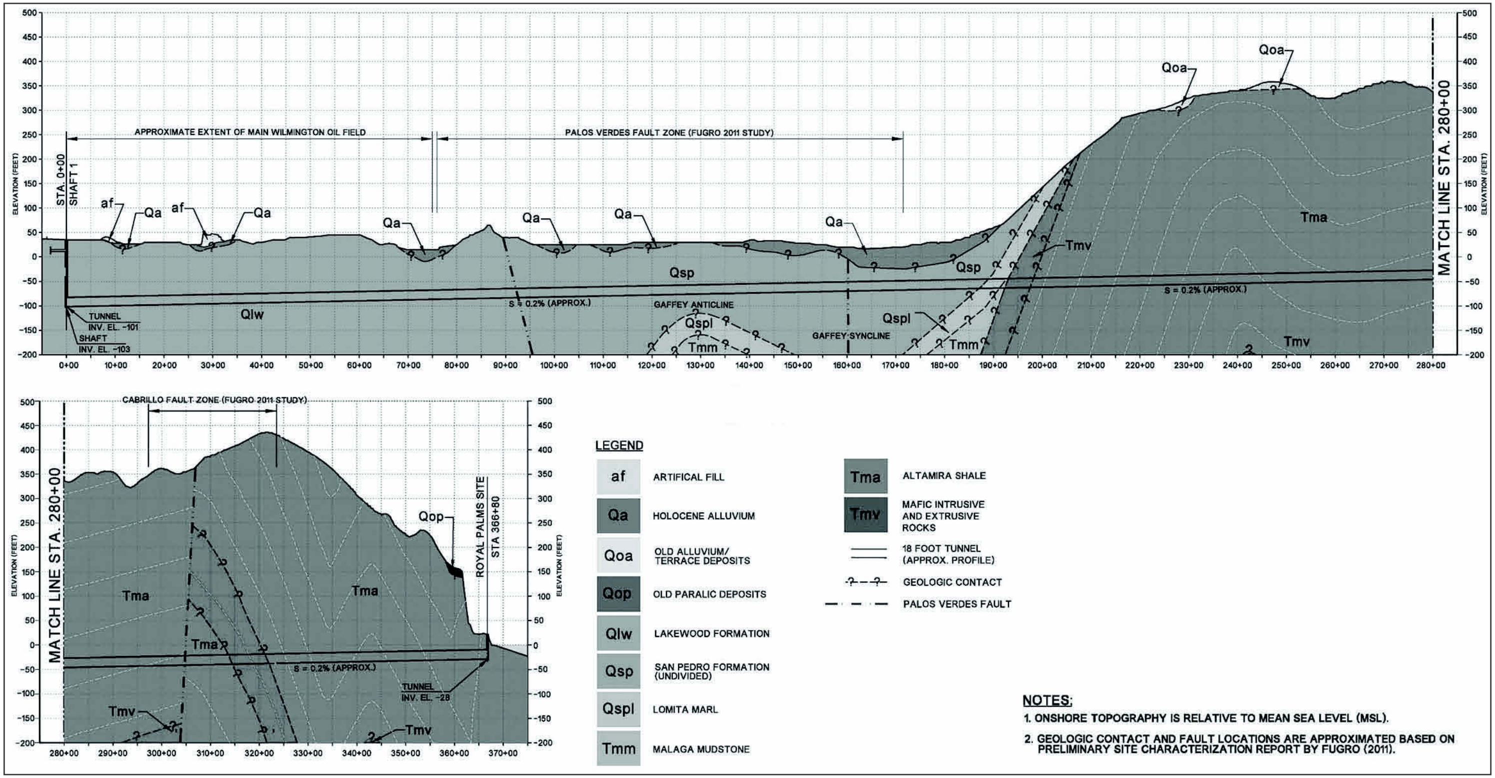

Figure 4. Geological profile along the tunnel alignment

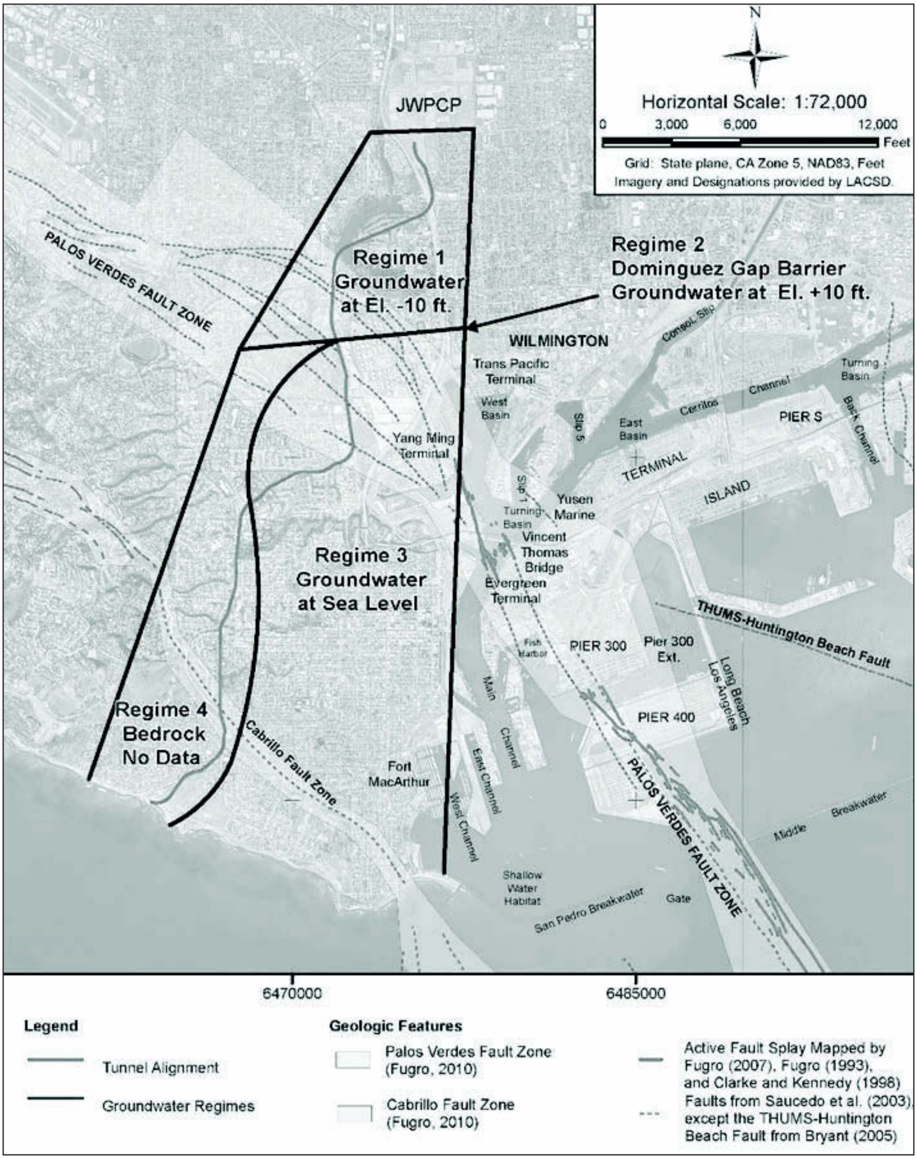

Figure 5. Groundwater regimes along the tunnel alignment

be incorporated into the cutterhead design. Based on preliminary data collected on the sedimentary rock along the alignment, the typical unconfined compressive strengths should be less than 150 lb/in^2, however, the lenses of higher strength material could be 5,000 lb/in^2 or greater.

Groundwater Conditions

Along the alignment, four hydrogeological regimes were identified as shown in Figure 5. In the area surrounding the JWPCP (Regime 1), the groundwater level was measured at an elevation of approximately 15 feet below Mean Sea Level (MSL). The second regime is a band of injection wells called the Dominguez Gap Barrier. Fresh water is injected into the ground to prevent saltwater from intruding into the Los Angeles Basin aquifers. The water level within the wells is usually kept at an elevation of +10 ft MSL. Regime 3 is located south of the Dominguez Gap Barrier and reliable water level data is absent along the alignment. Given the proximity to the Pacific Ocean, the current assumption is that groundwater levels have generally equilibrated to sea level. The presence of groundwater in the bedrock formations (Regime 4) can be highly variable and vary greatly over short distances. Due to the limited data and the variability, no groundwater level is specified at this time for Regime 4. It is expected that the groundwater head along the alignment in the alluvium material will be less than 3.5 bars, while in the rock-like material of the PV Hill, should a zone of highly fractured rock filled with ground water extend from the ground surface to the tunnel there is a possibility the hydrostatic pressure could reach as high as 11 or 12 bars.

Interventions

Throughout the entire tunnel length, TBM cutterhead interventions will be necessary. Ideally the interventions will be performed under free air, but access to the cutterhead while in soft ground or highly fractured rock beneath the water table may require the use of compressed air or possibly a mixed-gas environment. If longer interventions are required to perform repair work or change multiple cutters, the use of mixed gas under saturation conditions may be necessary. Working under saturation conditions will require the use of a saturation diving shuttle to transport the workers from the hyperbaric living quarters on the surface to the airlock on the TBM. While an 18-foot internal diameter tunnel will have enough space for ventilation, switches and pumps, there may not be enough space within the TBM for the shuttle to be connected directly to the bulkhead. To connect the shuttle directly to the bulkhead airlock, equipment will need to be removed to create a large enough space for the shuttle. Connecting the shuttle to the TBM bulkhead airlock by a transfer tube appears to be more appropriate for the proposed tunnel. As the geotechnical conditions along the alignment are better defined, it might be possible to avoid saturation diving conditions by the use of ports in the TBM to provide for grouting, or ground freezing, to create a conditioned environment that has a reduced pressure at the cutter face.

Muck Handling

A major area of analysis in the Clearwater Program EIR was the effect the tunneling operations will have on the air pollution and greenhouse gases in the surrounding environment while using diesel locomotives. The baseline for the analysis was an EPB TBM because larger horsepower locomotives will be required to transport the loaded muck cars. To mitigate the impact, different types of locomotives such as electric or natural gas were investigated. The analysis determined the batteries on electric locomotives would drain rapidly when the alignment becomes significantly long. The amount of time necessary to recharge the batteries was determined to have a significant impact on the production rate of the TBM. Either slower advance rates would be realized, or additional locomotives would be required to maintain an adequate amount of trains entering the tunnel with supplies. Neither option was deemed an acceptable alternative. The use of natural gas locomotives inside the confined space of the tunnel was determined impracticable due to safety concerns. Diesel locomotives were deemed the only power source capable of handing the transportation of the muck and supply cars over the entire length of the tunnel. Utilizing conveyor belts for muck disposal would reduce the amount of emissions in the tunnel by allowing small horsepower locomotives to be used. The reduction in emissions was not analyzed in the EIR because it was assumed the locomotives used with a Slurry TBM would be the same horsepower

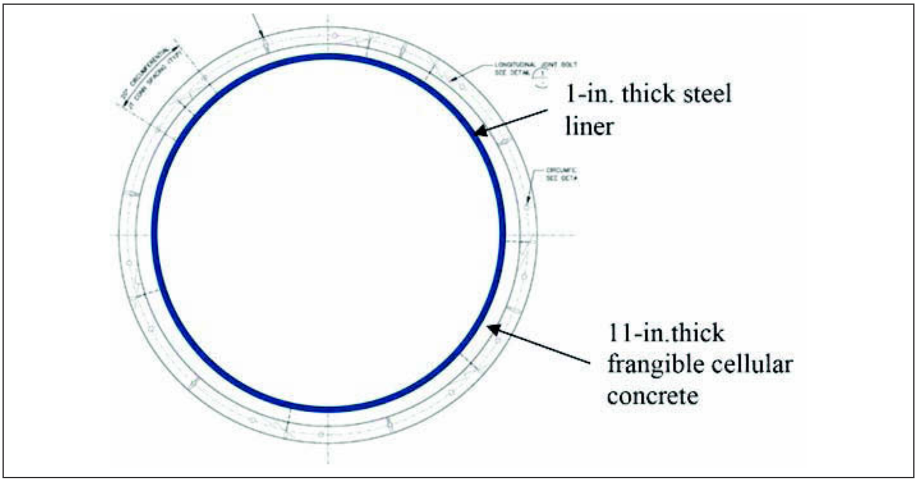

Figure 6. Typical cross section of 16-ft ID liner at PV fault crossing

with an EPB TBM using a conveyor belt as the muck disposal method. To reduce the amount of diesel particulates entering the tunnel environment and being exhausted into the surrounding community, a Tier 4 engine on the locomotives was mandated as part of the mitigation measures.

PRECAST CONCRETE SEGMENTED TUNNEL LINING

Segmental precast concrete lining systems are typically used for tunnel excavation using EPB or Slurry TBM's. For the project, both one-pass and two-pass systems were considered. Although a one-pass lining system is more suitable, some type of special lining may be required to contend with the internal operating pressure to prevent leakage and deal with any offset created by movement along the PV Fault. For the projected wet weather flow of 927 MGD, the head on the system at the JWPCP could be approximately 100 to 115 feet. Although the effluent is treated to secondary levels, any leakage into the surrounding groundwater table is not acceptable. A possible design to handle the high internal pressure includes installation of tension reinforcement in the hoop direction with special connections carrying the tension across the radial and shear joints at the circumferential joints. To limit cracking and leakage, the quantity of reinforcement steel may be large resulting in a "waffle slab" segment where the segment is thicker around the joints and thinner in the middle. The design will allow adequate space for bolt connections while minimizing the volume of high-strength concrete. After installation, the waffle pockets could be filled in with a low-strength concrete to provide a smooth pipe-like finish. Another design to handle the high internal pressures includes installation of a post-tensioning strand inserted in a continuous circumferential duct embedded in the segments.

Within the PV Fault Zone, a two-pass system incorporating a 16-foot ID steel pipe inside the 18-foot ID precast concrete segmented liner is proposed to contend with any displacement of the fault and prevent the effluent from leaking out of the tunnel. The fault zone is shown in Figure 5. In advance of comprehensive geotechnical explorations, the assumption was made that the lining would extend the whole fault width of approximately 5,000 feet. An illustration of the crossing is shown in Figure 6. The annular space would be backfilled with low strength grout or crushable concrete. After the PV Fault characteristics are better defined, the length of two-pass lining may be reduced, or possibly eliminated entirely.

SEISMIC SETTING

Preliminary seismic design criteria for the project was developed from seismic design criteria used for similar projects, the service life of the project, and the geologic conditions in the project area. Three levels of seismic exposure were considered for the project which corresponded to a 475-, 975-, and 2475-year average return period. Evaluations were performed to identify potential geotechnical and seismic issues that could pose hazards to the structural integrity of the tunnel. The principal hazards were determined to result from wave propagation (WP) and permanent ground deformation (PGD). Seismically-induced WP will stress the tunnel liner and result in strains. WP may also produce hydrodynamic forces, which could produce a water hammer within the pipeline. In loose to soft-to-medium dense soils, PGD consists of liquefaction-induced settlement. Fault offsets of different magnitudes may result from PGD, causing shear, tension, and/or compression that may lead to failure or collapse of the tunnel. A return period of 975-yr was selected for the design of the tunnel, which corresponds to a displacement on the PV Fault of 1.0 to 1.3 feet resulting from a magnitude 7.3 earthquake.

CONCLUSION

Once the geotechnical investigation begins for final design, several of the issues related to completing the design of the tunnel and associated structures will be determined. Final design, which is currently underway, is expected to take approximately 2.5 to 3 years to complete. For the geotechnical program, 54 borings totaling approximately 14,000 feet are anticipated to be drilled. Construction of the tunnel and associated structures is envisioned to be packaged under a single contract. Advertising and bidding is tentatively set for late 2015 or early 2016. The construction duration is estimated to take approximately 7.5 years after notice to proceed is given. The 18-ft Tunnel, once built, will insure the needs of the JOS are fulfilled to the year 2050 and beyond.

ACKNOWLEDGMENTS

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UPDATE ON THE "PIPELINE/TUNNEL OPTION" FOR THE DELTA HABITAT CONSERVATION AND CONVEYANCE PROGRAM

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Howard Lum ▪ Metropolitan Water District of Southern California

ABSTRACT

In order to improve the environmental sustainability of California's Sacramento-San Joaquin Delta region, and ensure the reliability of water deliveries throughout the state, the California Department of Water Resources has proposed multiple alternatives for water conveyance under the Delta Habitat Conservation and Conveyance Program (DHCCP). Each of the proposed alternatives will convey water from the Sacramento River north of the Delta to existing state and federal pumping plants south of the Delta. This paper describes recent developments to the conceptual engineering analysis of the "Pipeline/Tunnel Option," one of the proposed DHCCP options. The update to this analysis addresses revised anticipated system hydraulics, operational criteria, site conditions, and state-of-the-practice tunneling methodologies. The main features of the "Pipeline/Tunnel Option" consists of river intake structures, sedimentation basins, intake pump plants, forebays, and tunnels capable of diverting gravity flows up to 9,000 cfs. The main twin-bore tunnels will span approximately 35 miles consisting of 40-foot diameter tunnels with a precast concrete segmental liner constructed at depths up to approximately 150 feet below grade.

INTRODUCTION

Water drawn from the southern portion of the Sacramento-San Joaquin Delta provides water supply to 66 percent of California population and supports the State's agriculture. This existing through-Delta water system is outdated and unreliable with environmental risk to some fish and wildlife species. The Bay-Delta Conservation Plan (BDCP) has been established to environmentally retrofit and modernize California's water delivery system through the Delta by restoring habitats, constructing new diversion points in the north Delta, and providing a means to transport water supplies under the Delta, rather than through sensitive natural channels.

Under BDCP, the Delta Habitat Conservation and Conveyance Program (DHCCP) has developed several alternatives to convey water from the Sacramento River in the north to the existing pumping facilities in the south Delta through an isolated conveyance system. The new conveyance system would become an integral part of the State Water Project (SWP) and the federal Central Valley Project (CVP) by transporting water to the export pumping plants for each of these projects. The DHCCP is managed by California Department of Water Resources (DWR), while Metropolitan Water District of Southern California (MWD) provides support to the program on a proposed pipeline/tunnel conveyance option.

The initial conceptual study efforts on the overall program commenced in 2007 and examined various options for the proposed conveyance system. Three conveyance alternatives were analyzed at that time. Work efforts on one of the options, known as the "Pipeline/Tunnel option," have focused on preliminary system configuration, tunnel sizing and constructability issues, and the result of these studies is the focus of this paper. The conceptual study efforts conducted to date provide needed information for the environmental impact report (EIR) and environmental impact statement (EIS) processes that are currently underway. The final configuration of the approved conveyance option will be determined upon finalization of the EIR with anticipated preliminary engineering to begin in late 2013. The conceptual design presented here is subject to change upon review and approval of the final EIR. Figure 1 shows the conceptual alignment of the isolated pipeline/tunnel conveyance option.

SYSTEM CONFIGURATION

Under the current operations of SWP and CVP, water is conveyed through the Delta and the rivers are used as conveyance channels. Over time, the levees in the Delta have deteriorated, and environmental concerns of the fish and wildlife habitats become evident in the region. In order to restore the Delta habitats and improve water supply reliability, an isolated conveyance system is being developed as an alternative to divert water from Sacramento River through screened intakes via pumping and convey it around or under the Delta.

The DWR study conducted from 2009 to 2010 proposed a pipeline/tunnel system to convey up to 15,000 cubic feet per second (cfs) with these components: five intake facilities with fish screens, sedimentation basins, intake pumping plants, pipelines, an Intermediate Forebay, an Intermediate Pumping Plant, twin 33-foot inside diameter (ID) tunnels and a new Bryon Tract Forebay (BTF).

Subsequent studies further developed the above configuration with a modified Pipeline/Tunnel option (PTO). Based on a slightly refined alignment, the revised PTO configuration eliminated the Intermediate Pumping Plant and utilizes gravity-fed Main Tunnels. The total intake capacity was reduced to 9,000 cfs. Water will be conveyed under the Delta from three river intake facilities (fish screens, sedimentation basins and pumping plants) through intake tunnels (North Tunnels), an Intermediate Forebay (IF), and gravity-fed twin 40-foot ID conveyance Main Tunnels to BTF. Figure 2 shows a schematic of the proposed conveyance system.

CONCEPTUAL STUDY

Following the preliminary concept developed in 2010, the Project Team performed additional studies to evaluate the feasibilities of the DHCCP Pipeline/Tunnel option. An optimization study was conducted with the following objectives:

- Determine if an all-gravity system between IF and BTF is feasible
- Evaluate alternate intake capacities (variations from the original 15,000 cfs capacity)
- Optimize the tunnel diameter and liner configuration between IF and BTF
- Optimize the precast concrete segmental liner system for both internal and external pressures
- Study the potential leakage effects through the tunnel liner
- Study the historic seismic performance of tunnels
- Study the effects of tunneling-induced settlement and ground vibration
- Compare the Delta tunnel configuration with recent tunneling projects to evaluate constructability

The Project Team evaluated alternative system intake configurations ranging from 3,000 to 15,000 cfs to meet different demand and environmental scenarios. Concurrently, the feasibility of an all-gravity flow system between IF and BTF was

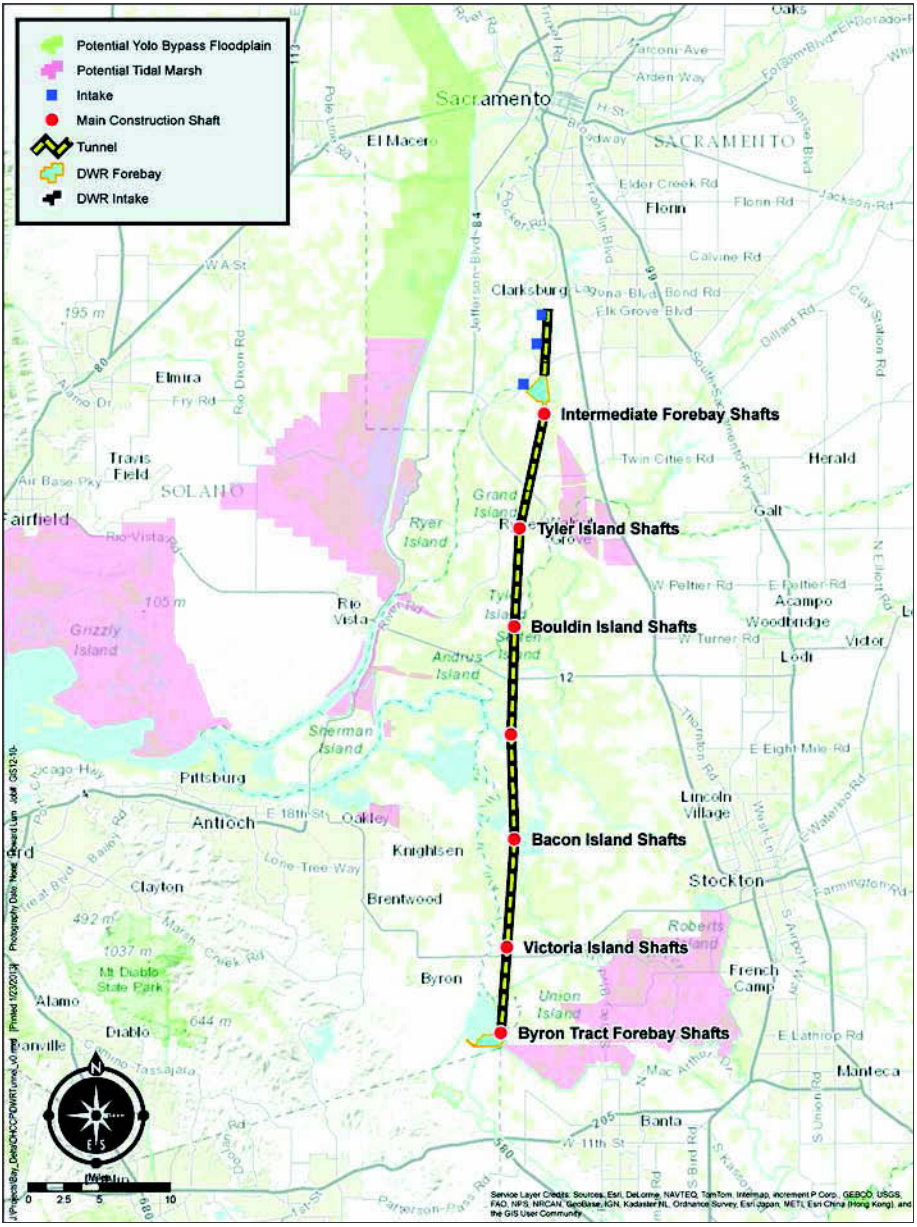

Figure 1. Conceptual plan of DHCCP pipeline/tunnel option

investigated. At the commencement of this study, the apparent advantages of eliminating the Intermediate Pumping Plant and utilizing an all-gravity system were identified as follows:

- Reduce electrical demands of the overall system. The reduction in electrical power consumption will better conform to future California energy use policy.
- Minimize long-term operating cost associated with power consumption.

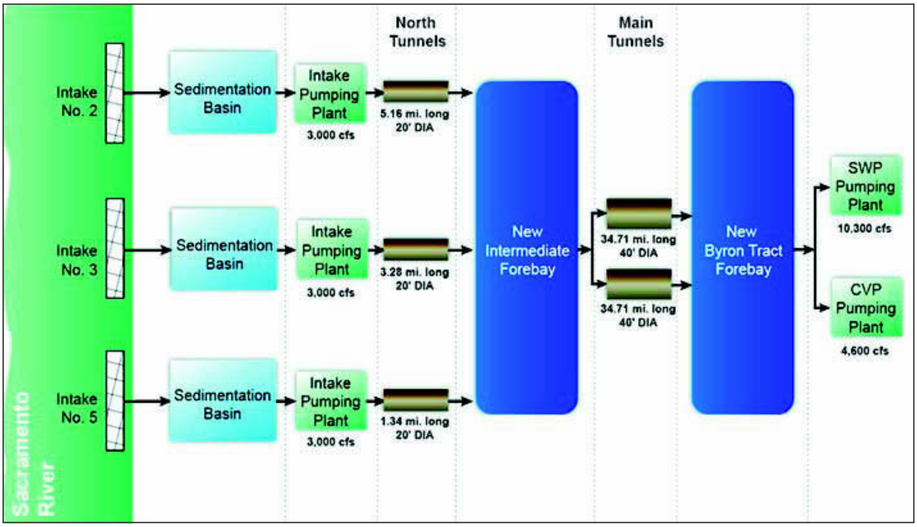

Figure 2. System configuration of DHCCP pipeline/tunnel option

- Reduce capital (construction) cost by eliminating the Intermediate pumping plant.
- Eliminate maintenance costs associated with pump maintenance and future upgrade.
- Improve and simplify system operation.

The Project Team utilized the basic assumptions in the original conceptual study, incorporated latest inputs from stakeholders, researched current regulatory requirements, further developed tunnel engineering concepts and evaluated constructability based on latest underground construction technologies. The resulting configuration is a conveyance system that consists of the following components designed to the respective criteria.

River Intakes

Each intake facility will consist of on-bank screened intake structures, gravity-fed intake pipelines between intakes and sedimentation basin, sedimentation basins, and an intake pumping plant with power substation/transformers. Given the historic flow data of Sacramento River and previous operation experience of DWR intake facilities, the maximum intake capacity of each facility is assumed to be 3,000 cfs. The final size and configuration of each intake structure will be determined by the maximum allowable intake/entrance velocity through the screen for different fish species in the River. The most critical intake flow velocity will be governed by the Delta smelt with a maximum allowable velocity of 0.2 ft/sec at the fish screen.

Based on demand/supply input by DWR and other stakeholders, it is assumed that the lower bound of the maximum river inflow is 9,000 cfs, while the upper bound is 15,000 cfs. For 9,000 cfs intake capacity, three 3,000 cfs river intakes will be required along the Sacramento River to supply water to the IF.

North Tunnels

A 20-foot ID tunnel will be required to connect each intake facility to the IF. Depending on the location of the intakes, the length of the tunnels will vary from 1.3 to 5.2 miles.

Tunneling will be performed using closed faced tunnel boring machines (TBM). A bolted-gasket precast concrete segmental liner will serve as a one-pass liner system for the tunnels.

Intermediate Forebay (IF)

Water from the three river intakes will be pumped to the Intermediate Forebay which provides an atmospheric separation between intake tunnels and the main twin-bored tunnels. The Forebay will regulate outflows by providing a relatively constant water elevation and will improve operational stability. This hydraulic break in the Forebay will allow independent operation of each river intake and the two main outlet tunnels. The planned embankment elevation is estimated to be at +32 feet with forebay invert at 0.0 feet. Operating range of water elevation is between +10 and +20 feet for 9,000 cfs inflow. The volume and surface area of IF is being optimized considering operation flexibility, environmental impact and construction cost.

Main Tunnels

Two parallel, identically sized tunnels were considered for the purposes of overall tunnel sizing. Twin tunnels are provided to ensure system reliability and to maximize operational flexibility. Maintenance and inspection of each tunnel can be performed by isolating one tunnel at a time. As the tunnels convey water from IF to BTF, the tunnel invert varies from elevations 145 feet to 163 feet below ground/mean sea level. The tunnel depths were estimated based on regulatory requirements to cross under a key river channel located near the mid-point of the tunnel alignment, and to provide sufficient clearance from any potentially liquefiable soils at upper ground elevations.

The total distance for the tunnel drives between IF and BTF is estimated to be approximately 35 miles. Final tunnel alignment and grade are subject to change upon completion of the Environmental Impact Report (EIR) and further geotechnical exploration. The preliminary alignment consists of twin-tunnels with five reaches and six working shafts as shown in Figure 1. To convey the required flow at an acceptable pressure and velocity for concrete liner, two identically sized parallel tunnels of 40-foot ID will be required. A bolted-gasket precast concrete segmental liner will serve as a one-pass liner system for the tunnels.

Given the high ground water and alluvial ground conditions, tunneling using an Earth Pressure Balance (EPB) or Slurry Pressure Balance (SPB) TBM is being considered. The TBM is advanced with tunnel muck admitted into the machine via a screw auger/conveyor arrangement while maintaining soil or slurry pressure at the tunnel face to remain balanced.

Shafts

In order to meet scheduled on-line dates and to maintain a tunnel drive distance consistent with current tunneling practice, the tunnels are divided into five reaches with an average distance of 7 miles. It is anticipated that each tunnel reach would be constructed under a separate construction contract. Launching and receiving shafts are required to connect between tunnel reaches. Given the tunnel diameter, the preliminary size for the launching/receiving shafts is estimated to be 110 feet ID, and the preliminary size for receiving-only shafts is 85 feet ID.

Due to the relatively long distance of each tunnel drive, hyperbaric interventions, intermediate intervention shafts or localized grouting from the surface may be required at locations between the shafts to service the TBM or mitigate adverse tunneling conditions. Interventions will be required in case of excessive face wear or elements failure that cannot be serviced from inside the TBM. Hyperbaric intervention allows repairs performed under pressurized condition, but progress can be slow because of limited work schedule allowed under high pressure environment. Intermediate intervention shafts will provide an atmospheric environment to service and maintain the TBM, but a new shaft is required at each service location. The size of the intervention shaft will vary depending on the type of work required. Given the difficulty of performing pressurized interventions and their schedule impact, TBM should be designed for high durability, advanced monitoring system and better internal access for maintenance so as to minimize such interventions.

Byron Tract Forebay (BTF)

The main tunnels will terminate at a new forebay adjacent to the existing Clifton Court Forebay (CCF). The current study proposes construction of a new forebay known as Byron Tract Forebay at the southeast side of CCF, while other alternative locations are being considered. The storage requirement of the forebay is estimated to be 4,400 to 5,900 acre-feet with a surface area of approximately 620 acres. The top of embankment elevation is estimated to be at +25.0 feet with forebay invert at –10.0 feet below mean sea level.

GEOLOGY OF PROJECT SITE

The Sacramento-San Joaquin Delta represents the extension of the San Francisco Bay estuary that blends into the Central Valley geomorphic province of California. Geologic units exposed within the project area consist predominantly of Holocene deposits of alluvial and tidal environments. These Delta deposits are underlain by alluvial fan and eolian deposits of Holocene and Pleistocene age derived from the drainage basins in the Sierran and Coastal ranges to the east and west.

Between 2009 and 2012, approximately 190 borings and cone penetration test (CPT) sounding have been advanced at the river intake sites, forebays and along various conveyance alignments. The subsurface exploration depths varied from 37 feet to 320 feet below existing ground surface, with the majority of explorations conducted between 100 and 200 feet. In addition, a seismic stability evaluation of the Delta levees was performed that required using accelerometers to be installed in deep borings and down-hole suspension of shear wave velocity logs.

In general, the Holocene deposits of soft/organic soils and peat were encountered to a maximum depth of 60 feet below ground surface. The Holocene materials are characterized as peat or very soft to medium stiff clay with shear wave velocity between 300 and 900 fps, and medium dense silty sand with shear wave velocity of 400 to 1,000 fps.

The deeper alluvium of probable Late Pleistocene age are characterized by dense to very dense silty sand and stiff to hard silty clay with shear wave velocity between 700 and 1,150 fps. As depth increases towards the tunnel invert, the Pleistocene sandy soil varies from dense to very dense with shear wave velocity from 1,200 to 1,850 fps. Because of the alluvial nature of the depositional environmental at the proposed tunnel grade, lateral and vertical changes from silty clay to clayey silt to silty sand, and fine to coarse grained sand should be anticipated over short distances.

MAIN TUNNELS DESIGN CONCEPT

The proposed tunnels consist of approximately 35 miles of twin 40-foot ID conveyance tunnels at invert elevations of 145 to 163 feet below mean sea level. Early study on parallel tunnels behavior in soft ground (Ghaboussi and Ranken; Gercek) concluded that deformation interactions between two parallel tunnels would be negligible if the clear spacing (pillar width) between the tunnels was greater than two tunnel diameters. For seismic response of twin tunnels in stiff ground, research suggests that if the center spacing between the two tunnels is more than three times the tunnel diameter,

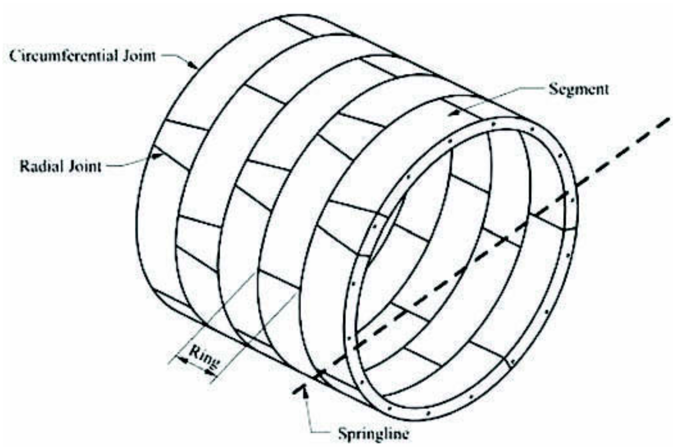

Figure 3. Schematic view of precast concrete segmental liner (PCTL)

the cross-tunnel seismic interference effects should be negligible (Anitha Kumari et al). Using an excavated diameter of 45 feet for the Delta tunnels, the center to center spacing of the two tunnels is then assumed to be 150 feet for EIR planning purpose. Spacing of the two tunnels will be refined and determined during preliminary design after site specific geotechnical data are available for analysis.

Preliminary geotechnical data of the Delta indicates alluvial ground conditions and high ground water table. For the large tunnel diameter required, a one-pass, bolted and gasketed precast concrete tunnel liner (PCTL) system utilizing EPB or SPB tunneling technology is proposed. A schematic sketch of PCTL is shown in Figure 3. PCTL will be installed continuously following the advancement of the TBM. The lining consists of eight precast concrete segments plus a key segment, inter-connected to maintain alignment and structural stability during construction. Reinforced concrete segments are precast to comply with strict quality control. High performance gaskets (single or double sets) maintain water tightness at the concrete joints, while allowing each joint to rotate and accommodate movements during earthquakes (Dean et al). EPB and SPB tunneling technologies have improved substantially over the years, and many large diameter tunneling projects utilizing PCTL and EPB/SPB have been completed successfully around the world. Recently completed large diameter tunnels using PCTL are listed in Table 1.

Excavation of the largest PCTL/EPB tunnel at 54-foot excavated diameter will commence in 2013 for the SR-99 Alaskan Way project in Seattle. Based on the above data showing current tunneling technologies and recent case histories, constructing a 40-foot ID PCTL for the Delta project that utilizes pressure balance TBM is considered feasible.

The Project Team conducted conceptual analysis of the 40-foot ID PCTL to provide support information for the EIR, identify potential design issues and to develop an engineering management plan for future preliminary and final design scope. The following issues for the main tunnels were studied during the conceptual phase:

- Tension design of PCTL resulted from net internal pressure
- Leakage estimate through the pressurized tunnel
- Earthquake performance of PCTL
- Seismic design approach for tunnels and shafts

Table 1. Recently completed large diameter PCTL tunnels utilizing pressure balance TBM

Project	Location	Machine Type	Machine Diameter (feet)	No. of Tunnels	Geology
Hubertus Tunnel	The Hague, Netherlands	Slurry Mixshield	34.54	2	Fine Dense Dune Sands, Soft Clays and Silts. High groundwater table
Heathrow Airside Road Tunnel	London, England	Dual Mode–EPB and Compressed Air	30.11	2	Gravels over stiff, competent London Clay. High groundwater table
SMART	Kuala Lumpur, Malaysia	Slurry Mixshield	43.33	1	Mix: Karstic limestone, quarternary alluvium. High groundwater table
Shanghai Yangtze River Tunnel	Shanghai, China	Slurry Mixshield	50.61	2	Soft clay and silty clay, liquefied soil and quick-sand. High groundwater table.
M30, North and South Bypass Tunnels	Madrid, Spain	EPB	49.53 49.86	2	Alluvial deposits fissured hard clay with gypsum layers. High groundwater table
Barcelona Metro Line 9	Barcelona, Spain	EPB	39.56	1	Soft soils, stiff over-consolidated clay
		Dual Mode–EPB and open mode	39.2		Sands, clay, and silts overlying gravels with sands
					Heterogeneous mixed face with soft soils, weak to hard rock
		EPB	30.8		Submerged fine silty sands and sandy silts
Sparvo Tunnel	Bologna, Italy	EPB	51.2	2	Clay, claystone, sand-stone, limestone and alluvium

- Probable ground vibration effects from tunneling and potential mitigation measures
- Tunneling-induced ground settlement on levees, above-ground structures and buried utilities

Tension Design and Leakage Estimate

The Delta tunnels will be operated as pressure conduits, and the PCTL will be designed to withstand the net internal pressure that results from the difference between internal and external hydrostatic heads. The maximum net internal pressure for the Delta Tunnels is 50 feet of hydrostatic head (22 psi). Hoop tension from the internal pressure will induce tension at the segment joint and circumferential tensile strain on the concrete segment body. Two important design considerations were evaluated: tension force transfer at segment joints and potential water leakage (exfiltration) from the tunnel.

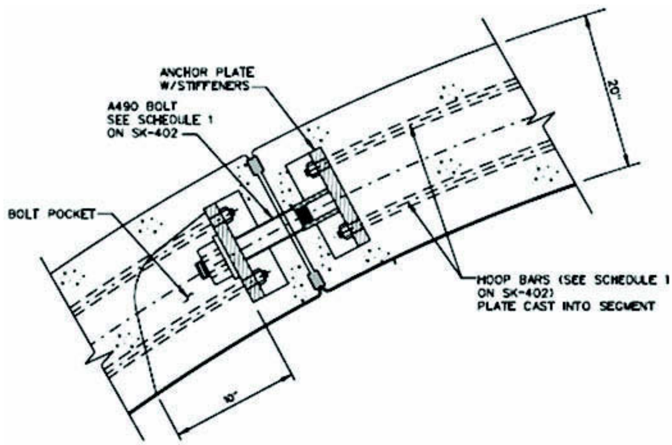

Figure 4. Sketch of PCTL segment joint for tension load

The conceptual design estimates that a 20-inch thick precast concrete segmental liner (8,000 psi minimum) with high performance gasket would be required to handle the ground and hydrostatic loads. Steel reinforcement will be designed in the concrete segment to resist the tensile stress. To resist the hoop tension force, several alternatives were considered in the study, including tension bolts through radial joints, embedded steel plate in the segment, post-tensioned prestressed circumferential tendons and shear cones. The proposed design suggests that hoop tension reinforcement and bolt connection at the radial joints should provide adequate strength. A sketch of the concrete segment joint detail is shown in Figure 4.

Both the segment joints (radial and circumferential) and the concrete segment body will be designed to minimize water leakage out to surrounding ground. Controlling water exfiltration will prevent erosion of tunnel support and minimize economic loss of transported water.

For flows through segment joint, the gasket(s) will be designed to seal against any potential leakage from the tunnel. Researching recent tunnel projects indicate that modern gasket technology is capable of handling pressures up to 390 psi with sufficient factor of safety (Lum et al). Leakage through the segment joint can be controlled and sealed with proper gasket design and construction, so exfiltration through the joint should be negligible.

Tunnel leakage through the segment body is anticipated, and the exact quantity is dependent on the liner permeability and the ground permeability surrounding the tunnel. Predictive models (Fernandez) have been established to estimate tunnel leakage based on concrete liner crack width/spacing and ground characteristics. Using preliminary ground permeability parameters and ignoring ground overburden (which reduces seepage), an evaluation of tunnel stability estimates that piping or erosion can be controlled within acceptable limit of water exfiltration by utilizing ground treatment, balanced reinforcement ratio and appropriate concrete mix design.

Earthquake Performance

Based on historical records of structures subject to seismic events, underground structures suffered appreciably less damage than surface structures. Reported damage decreases with increasing over-burden depth, and deep tunnels appear to be safer and less vulnerable to earthquake shaking than are shallow tunnels because of ground attenuation effect. Studies indicate that lined and grouted tunnels (e.g., PCTL) perform better than unlined tunnels, and damage can be further reduced by improving the contact between the liner and the ground (grouting of annular space between the liner and the surrounding soil). Earthquake effects on underground structures can be grouped into three categories:

1. Ground shaking that induces strains through axial deformation, longitudinal bending and ovaling
2. Ground failure such as liquefaction and fault displacement
3. Displacement incompatibility between elements, such as shaft to tunnel connection

Since the 1980s, PCTL systems have been used extensively in seismically active areas such as Japan, Puerto Rico, Taiwan, Turkey, Italy, Greece and United States. Responses of PCTL to recent earthquakes were studied and the results indicated little or no damage to modern PCTL under moderate to strong ground motions (Dean et al., 2006).

Preliminary modeling of active and potentially active faults was developed and evaluated as part of the Delta Risk Management Strategy (DRMS) study. Preliminary results indicate that the proposed Delta tunnel alignment does not cross any major fault rupture or creep zones. Liquefaction was also investigated during the study through soil sampling with vertical boreholes and cone penetration tests. Ground motion from a 500-year return period earthquake was used to analyze the liquefaction potential of the tunnels. The results indicate that the upper 40 to 60 feet exhibited liquefaction potential given the presence of soft and loose soils. Tunnel shaft design would account for any liquefaction potential. Currently, the assumed depth of ground cover above the Delta tunnels is more than 100 feet, which should eliminate or minimize the liquefaction risk to the tunnels.

Tunnel-Induced Ground Vibration and Settlement

The Delta tunneling work will not be conducted in highly urbanized areas, however tunneling will be conducted in the vicinity of rural development for preliminary design purposes and the EIR; urban ground vibration limits were reviewed to provide an upper bound for comparison. The governing ground vibration criteria will be evaluated based on effects to:

- Existing above-ground structures such as buildings, bridges or levees
- Buried structures such as pipelines or other utilities
- Ground subsidence
- Architectural or historical elements
- Contents inside a structure
- Human perception during different times of the day

For the purposes of the Delta project study, two approaches were used to evaluate ground vibration effects resulted from TBM tunneling in alluvial soils. The first approach involved researching and analyzing existing TBM induced ground vibration data from recently completed tunneling projects. This research provided information on actual field measurements and observations of TBM ground vibration effects to surface facilities. The second approach calculated anticipated surface/near-surface ground vibrations using preliminary geotechnical data from the Delta area.

The proposed Delta tunnels will be constructed with a minimum of 100 feet of soil (alluvium) cover. Using peak particle velocity (PPV) data collected from other urban tunneling projects, it appears that surface PPV vibrations are beyond human perception for tunnel depths over 100 feet. Preliminary analysis using site-specific data also concludes that the deep soil cover over the Delta will likely dampen and absorb any TBM propagated energy. Consequently, the proposed Delta tunnels will be at depth sufficient to render minimum induced vibration to structures and not likely to be perceptible on the surface.

Tunneling activities have the potential to cause excessive surface settlement if appropriate measures are not undertaken. Settlements occur above tunnels during construction in response to the removal of earth materials at the tunnel face, convergence of voids created around the tunnel excavation, and stress redistribution around the excavated tunnel. Recent advancement in analytical prediction of settlement, sophisticated TBM control system, improved structure protection methodologies, and advanced settlement monitoring systems have significantly mitigated the risk of settlement effects and structural damage.

With the advancement of pressurized face tunneling, it is possible to minimize ground loss through careful TBM control and monitoring during tunneling. Additionally, pre-excavation grouting can be performed in front of the TBM (through the cutter head) to fill voids and stabilize ground prior to mining. Grouting from the surface, to densify and improve ground conditions at tunnel depth, can supplement pre-excavation grouting for settlement control. If required, a special belt measuring system with radar can be installed in the TBM to provide accurate real-time face loss data during tunneling.

Settlement can be estimated by considering the effects of tunnel face ground deformation, TBM shield/lining configuration and workmanship (Lee et al). For the Delta project, both differential and total settlement effects will be closely evaluated as the alignment crosses the following features:

- Above ground structures such as buildings, bridges, railroads and highways
- Existing buried pipelines and utilities
- Existing levees

Once the ground conditions for the Delta tunnels are determined through detailed geotechnical exploration, probable ground settlements and settlement profiles resulting from tunneling can be calculated using established principles. Pre-construction surveys will be performed for critical and settlement sensitive facilities, utilities and surface features to establish elevation baselines. Project designers will consult with owners of individual facility, structure, pipeline and utility to develop acceptable tunneling protocols and permissible settlement criteria to ensure there are no negative impacts to surface or buried facilities from tunneling. Potential tunneling-induced settlement effects on existing levees, structures, bridges, railroads, and buried pipelines and utilities will be analyzed during design phase. Settlement mitigation measures, if necessary, will be appropriately designed and implemented during construction as required by affected levees, structures, facilities, pipelines and utilities.

During construction, tunnel construction practices will be performed under contract specified conditions to limit and control water table, settlement and vibration within stated permissible limits. Structures and other features sensitive to settlement or vibration may have to be protected prior to tunneling. As required, a settlement monitoring program will be implemented on sensitive features to ensure tunneling-induced settlements are controlled within acceptable limits. Additionally, a ground vibration monitoring program using seismographs and other high-precision equipment will be implemented as necessary to ensure surface vibration is within contract limits.

PROJECT SCHEDULE

A draft EIR is being prepared for the DHCCP program with public release anticipated in 2013. Preliminary and final design will follow on separate construction contracts including but not limited to intake facilities, north tunnels, Intermediate Forebay, main tunnels and Bryon Tract Forebay. The main tunnel drives will be divided into 5 or 6 contracts to distribute sufficient industry bonding capacity, meet TBM manufacturing schedules and promote competitive bidding. A 6-month lapse is currently planned between the award phase of one contract and the bidding phase of the next contract, to allow contractors to assess, regroup and submit subsequent bids. In addition, alternative project delivery methods will be explored to expedite program schedule and to balance construction risks. The planned completion date for the overall program is currently 2026.

CONCLUSIONS

The overall system configuration of DHCCP all-gravity PTO option is not fully defined at this stage, and the information described herein is based on conceptual engineering and only provides baseline parameters for the development of the Draft EIR. Upon issuance of the Draft EIR, it is anticipated that subsequent refinements to the program design will be required based on public comments, additional geotechnical exploration and engineering refinements. Besides fulfilling all the EIR requirements, further engineering study of the following areas will be required to optimize the underground work of DHCCP program:

- Finalize tunnels alignment (horizontal and vertical)
- Explore alternative contract delivery methods to optimize project schedule, cost and risk
- Secure temporary construction power for the tunnel drives
- Coordinate with affected agencies on construction access routes
- Estimate ground abrasiveness for tunneling and establish requirements for tunnel drive interventions
- Provide tunnel design to resist tension load and minimize water leakage through liner
- Establish settlement criteria and design for existing levees, roads, bridges and underground utilities
- Set up geotechnical exploration program to obtain needed data for design and analysis
- Secure required tunnel-related permits

The anticipated tunneling work associated with the DHCCP program will require creative management, advanced engineering and innovative construction approaches. Given the urgency to secure California's water supply while restoring the Delta's nature ecosystem, engineers and constructors will take on the challenge to meet the program goals with a balance of environment, cost, operations and engineering.

ACKNOWLEDGMENTS

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PART 6

Geotechnical Considerations

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INITIAL GROUND SUPPORT FOR TUNNELING IN CENTRAL TEXAS, FROM DESIGNER'S AND OWNER'S PERSPECTIVES

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Peter Chou ▪ Parsons Corporation

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Joe Hoepken ▪ City of Austin

ABSTRACT

Geologically, central Texas inherits one of the best tunneling media, "Austin Chalk" alongside some troublesome, weak sedimentary rock or pyroclastic materials, namely Grayson Marl and calcareous claystone or shale. The weak inhomogeneous rock mass usually exhibits relatively short stand-up time, which would require quick installation of closely spaced initial ground support, while Austin Chalk can have stand-up times exceeding several months and no more than spot bolting for stabilization. From a recently completed tunneling project in the Austin Metropolitan area and our literature review of past tunneling projects furnished in central Texas, structural and economical comparisons are made between each routinely used tunnel support system in terms of capacity, deformability, design and construction considerations, cost, and risks associated with unexpected fault zones, slaking, groundwater inflow, and block/rock fall, based on our design experiences. Finally, differences between the designer's and contractor's value engineered ground support systems are discussed from the owner's perspectives.

INTRODUCTION

The market for smaller size water and wastewater tunnels has rapidly grown in past decades in central Texas, thanks to the utility improvements required to meet the demands from significant population growth along with undersized and aging infrastructures. From the authors' recent tunnel experiences for the City of Austin's South I-35 Water/Wastewater Improvements Program, distinct differences of the design approaches for initial ground supports between the designer's conservatism and the contractor's optimism are noted; and therefore, additional literature of four other Austin tunnel projects are reviewed to find a consent initial ground support design approach for tunneling through central Texas' competent soft rock stratum when several noticeable ground risks are presented. The preferred approach considers the most commonly used initial tunnel support systems and their applications in the specific central Texas geology. Most importantly, these ground support systems were also discussed from the owner's perspectives in this paper.

RECENT TUNNELING EXPERIENCES IN CENTRAL TEXAS

To accommodate the rapid population growth in the Southern Austin area, the City of Austin initiated the South I-35 Water/Wastewater Improvements Program and broke the wastewater component of the project into 3 separate tunnel contracts: the Onion

Creek Rinard to Slaughter (OCRS) Interceptor, the Golf Course (OCGC) Interceptor, and the tie-in Interceptor. The OCRS and OCGS tunneling operations were completed in 2012 and 2011, respectively, and their tunnel initial supports were reviewed in this paper. The OCRS Interceptor consists of approximately 1 km of 2.2 m excavated diameter bored tunnel and adits with finished 0.9-m and 1.4-m-diameter wastewater pipes that intercept flows from the Zachary Scott Wastewater Interceptor and the OCGC Interceptor for conveyance to the downstream connection with the existing Slaughter Creek Wastewater Tunnel. The OCRS segment intersects the Onion Creek and Slaughter Creek three times, with ground covers as shallow as approximately 2.7 m including up to 1.8 m thick pyroclastic materials. The OCRS tunnel was excavated by a TBM except for a small 15 m long adit tunnel, dug by a road header, connecting the OCGC tunnel. Another short adit tunnel was excavated by a smaller diameter TBM.

The OCGC Interceptor, excavated also by a TBM, consists of approximately 2.3 km of 2.2 m diameter tunnel from Onion Creek Wastewater Treatment Plant under Onion Creek Golf Course, to upstream location immediately west of I-35 along Onion Creek. During OCGC construction, the tunneling operation set a record advance rate of approximate 50 m in a 10-hour-shift with its TBM.

Prior to the construction of the most recently completed I-35 Onion Creek tunnels, four other nearby existing tunnel constructions were also studied through the available literatures to assist the design of I-35 Onion Creek tunnels; they are Austin Crosstown Wastewater (ACW) Interceptors 5029-1 and 5029-3 located north of Downtown Austin, and Onion Creek Wastewater (OCW) Interceptor Sections II and IV, located between Downtown Austin and I-35 Onion Creek tunnels. Tunneling for the ACW Interceptor was completed between 1973 and 1974, and the OCW Interceptor was completed between 1984 and 1986. The approximate locations and overview of the studied tunnels are shown in Figure 1 and Table 1, respectively.

CENTRAL TEXAS GEOLOGY AND GROUND RISKS FOR INITIAL TUNNEL SUPPORTS

The central Texas is located within the Balcones Fault Zone (BFZ), which is a belt of inactive normal faults that trends southwest to northeast through central Texas and separates the Edwards Plateau to the west and the Coastal Plains to the east and southeast. In the City of Austin (eastern part of BFZ), below the alluvium deposits (along the creeks/rivers), the underlying bedrock is primarily Cretaceous-aged limestone, chalk and shale of the Austin Group, and shales of the Eagle Ford Group. The Glen Rose and Walnut Member dolomitic limestone is generally encountered within the western portion of the central Texas. The depositional environment in central Texas during the Cretaceous Period was primarily marine, influenced by the northwest encroachment of the Gulf of Mexico syncline located to the southeast.

Majority of the studied tunnel alignments were bored within the upper bedrock stratum of their locations, at depths ranging from 6 to 25 m. The central Texas bedrock units are briefly summarized in Table 2.Three studied tunnels, located east of or through the Balcones Fault Zone, encountered the Taylor Group (Claystone and Marl, about 210 m thick). All studied tunnels but one (OCW-IV) encountered the Austin Group (generally 100 to 130 m thick). One studied tunnel (ACW-5029-3), located west of the Balcones Fault Zone, encountered the Glen Rose and Walnut Formation (Dolomitic Limestone and Marl, 210 to 230 m thick), which is friable, porous, and sensitive to abrasion and changes in water content.

The studied tunnels did not encounter the Eagle Ford Formation, which was commonly seen in other tunnel projects in central Texas. The contact between the Lake Waco shale and South Bosque Shale is gradational, but typically considered as the uppermost bench-forming limestone bed. Weathering of the South Bosque Shale and

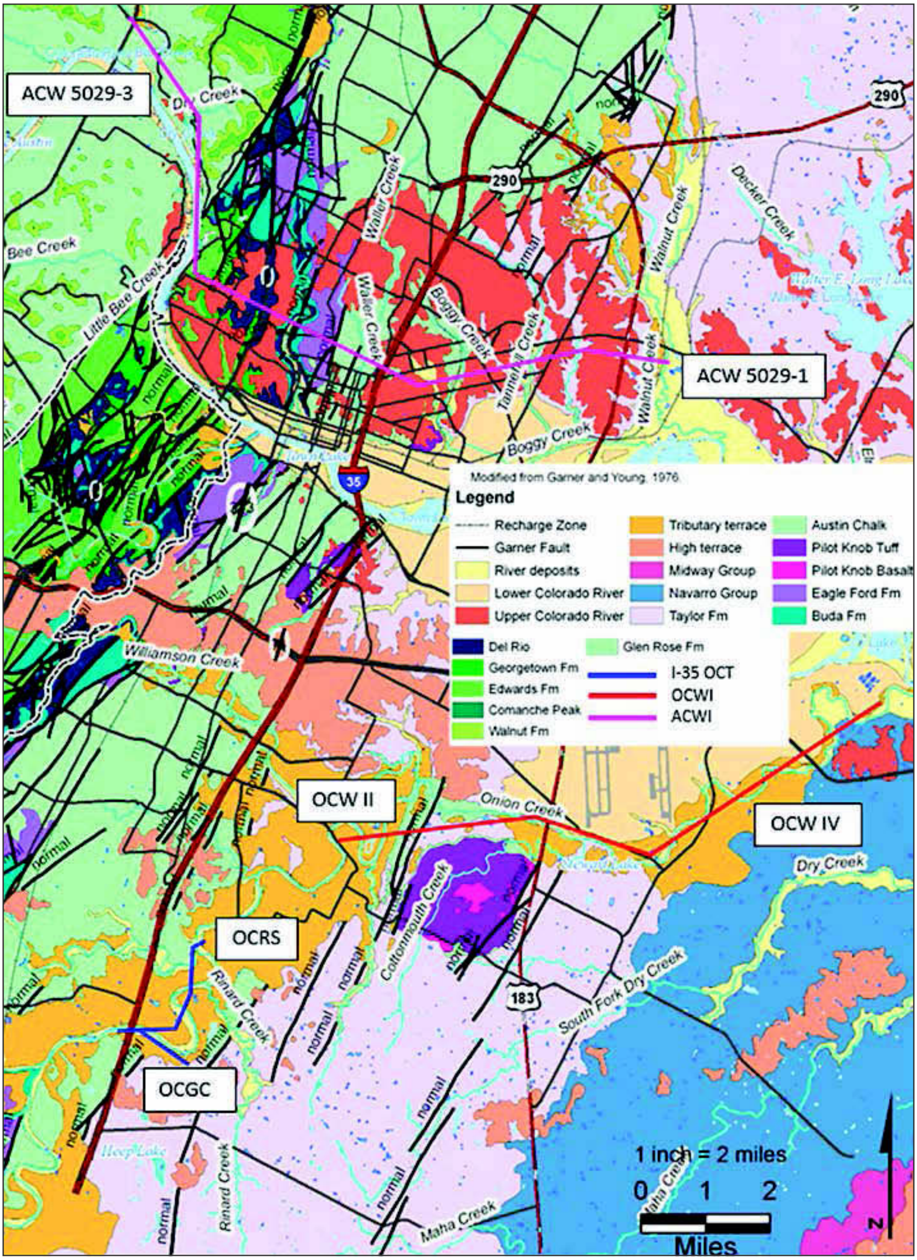

Figure 1. Geological map of the studied tunnel alignments in central Texas

the Lake Waco Formation produces highly plastic clay soils (fat clay). The summary of available bedrock engineering testing properties from the studied tunnels is listed in Table 3. To better compare the available testing data, the lab testing from another nearby tunnel project in the Eagle Ford Formation was also listed. From Table 3, it can

Table 1. Overview of the studied tunnels

Contracts	Tunnel Size (m)	Tunnel Length (km)	Contractors	TBM	Bedrock Types	Tunnel Supports	Initial Tunnel Support Ground Risks
I-35 OCRS	1.6–2.2	1.6	Southland	SCI & Road Header	Austin	Rock Bolts, Plates	Shallow rock cover, sloughing, overbreak, water infiltration
I-35 OCGC	2.2	2.3	S.J. Louis	Robbins	Austin	Rock Bolts	Shallow rock cover, Sloughing, overbreak
ACW-5029-1	3.2	9.2	Peter Kiewit Sons	Calweld	Austin, Taylor	35% unsupport, 45% Ribs/lagging, Rock Bolts, 20% Shotcrete/sealer	Shallow rock cover, sloughing, overbreak, water infiltration, slickensides, faults
ACW-5029-3	2.6–2.9	7.9	Granite	Robbins	Austin Glen Rose	Not installed	Sloughing, overbreak, faults
OCW-II	2.8	6.3	Seven K	Lovat	Austin, Taylor	Ribs/Lagging	Sloughing, overbreak, water infiltration, faults, slickensides
OCW-IV	2.8–3.0	9.0	Mole/S&M (JV)	Lovat	Taylor	Ribs/Lagging, Plywood, Channels	Water infiltration, overbreak, faults, slickensides

Table 2. Geological setting of central Texas (Austin)

<table>
<tr><th>Period</th><th colspan="2">Unit</th><th colspan="2">Description</th></tr>
<tr><td>Quaternary</td><td colspan="2">Alluvium</td><td colspan="2">Gravel, sand, silt, clay</td></tr>
<tr><td rowspan="5">Upper Cretaceous (Gulf)</td><td colspan="2">Taylor Group (ACW5029-1, OCW II & IV)</td><td colspan="2">Thick massive, calcareous claystone. Unweathered Taylor rock is typically massive with little trace of bedding.</td></tr>
<tr><td colspan="2">Austin Group (Chalk)* (OCRS, OCGC, ACW, OCW II)</td><td colspan="2">Alternating beds of chalky limestone and argillaceous limestone, referred to as volcanic deposits and marl.</td></tr>
<tr><td rowspan="2">Eagle Ford Group†</td><td>South Bosque</td><td colspan="2">Laminated shale, lacks abundant limestone</td></tr>
<tr><td>Lake Waco</td><td colspan="2">Shale with thinly bedded limestone and bentonite layers</td></tr>
<tr><td colspan="2">Pepper Shale</td><td>Buda Limestone</td><td>Grayson Marl (Del Rio Clay)‡</td></tr>
<tr><td rowspan="2">Cretaceous (Comanche)</td><td colspan="2">Georgetown Limestone</td><td colspan="2">Edward Limestone</td></tr>
<tr><td colspan="2">Glen Rose Form. (ACW 5029-3)</td><td colspan="2">Dolomitic Limestone and Marl</td></tr>
</table>

* Volcanic activity associated with the Pilot Knob volcanic complex introduced pyroclastic sediments into the predominantly marine environment, and settled to the bottom as clay, referred to as the Pilot Knob Tuff.

† The unit is described as dark calcareous clay shale that contains sandy and silty limestone in the middle and a bentonite bed at the base. The Eagle Ford Formation is generally 7 to 20 m thick.

‡ It is composed of soft calcareous clay and thin siltstones. Calcareous Clay (Marl) is described as low dark gray, low hardness to moderately hard with calcite veins.

be seen that the material strength of Taylor Group claystone and Eagle Ford South Bosque shale was generally low and highly variable, including the average values.

Typical Ground Risks for Tunnels in Central Texas

Both the OCRS and OCGC tunnels encountered Austin Chalk with unconfined compressive strengths averaging at approximate 13 to 20 MPa and localized bentonite and pyroclastic zones. The unweathered Austin Chalk is generally uniform, with medium high durability, and RQD values indicate good to excellent quality rock. For small tunnel diameters, primary support was not required in Austin Chalk and the rock mass standup time could be more than several months. Spot bolting was generally sufficient for the majority of tunnel alignments (see Figure 2). The Austin Group was satisfactory for tunnel advancement, except in bentonite, pyroclastic/claystone, and within fault zones, where liner plates (or steel ribs) were used (see Figure 2). Additional rock bolts were installed in the vicinity of the pyroclastic zones, and some of the rock bolts experienced pull-out failures (see Figure 2). In some areas, soft zones collapsed (overbreak) prior to installation of liner plates. The OCRS and OCGC tunnels also bored through several creek crossings with less than 3 m of ground cover. Probing and pre-excavation grouting were not performed on these jobs despite the considerable history of high inflows under creek crossings on previous jobs in the area. Fortunately, significant inflows were not encountered during OCRS and OCGC tunneling.

The ACW 5029-1 tunnel began in the Taylor Group rock at the east, and continued through the Austin Group into the fault zone. The contractor encountered softened claystone and alluvium with water inflow within the Taylor Group. Wet and sloughing rock and overbreak could not be stabilized with rock bolts and shotcrete. In some places, standup time was as low as five minutes. Rock bolts and mining pan supports often held for a few days, but delayed support failure produced extensive overbreak. Later,

Table 3. Summary of RQD and available bedrock testing results

Bedrock	Tunnel	Unconfined Compressive Strength (MPa)†	Tensile Strength (MPa)†	RQD*,† (%)	Slake Durability Index (%) †
Taylor Group (Claystone and Marl)	OCRS	2.1–10.1 (6.5)	—	50–75	—
	ACW	0.5–11.7 (3.1)	—	—	—
	OCW-II	1.9–9.7 (5.9)	—	90–100	44
	OCW-IV	0.2–5.6 (1.9)	—	90–100	0–72 (29)
Austin Chalk	OCRS	1.3–40.5 (12.9)	1.9–4.0 (2.7)	90–100	83–97 (93)
	ACW	7.4–18.5 (13)	—	—	—
	OCW-II	14.4–26.5 (18.9)	—	50–75	73
Pyroclastic/ Tuffaceous Claystone	OCRS	2.5–3.0 (2.7)	0.9–2.0 (1.5)	50–75	17–32 (22)
	OCW-II	7.9–11.9 (9.8)	—	50–75	—
Eagle Ford Formation South Bosque Lake Waco	OCRS	6.6–14.4 (9.3)	—	25–50	—
	BCT‡	0.2–5.0 (2.3)	0–2.8 (1.6)	< 25	1–75
		2.0–10.4 (7.0)	1.0–1.9 (1.4)	75–90	76–87
Glen Rose/Walnut	ACWI	6.2–29.6 (16.5)	—	—	—

* Only the weighted average RQD is shown.
† Not available; (average value).
‡ Testing results from another tunnel near the Bird Creek was used as references for Eagle Ford Formation.

Figure 2. (Clockwise from top left) Spot bolting in Austin Chalk; semi-circular liner plates supporting bentonite zones; rock bolt failure in pyroclastic material; liner plates; steel arches supported pyroclastic materials

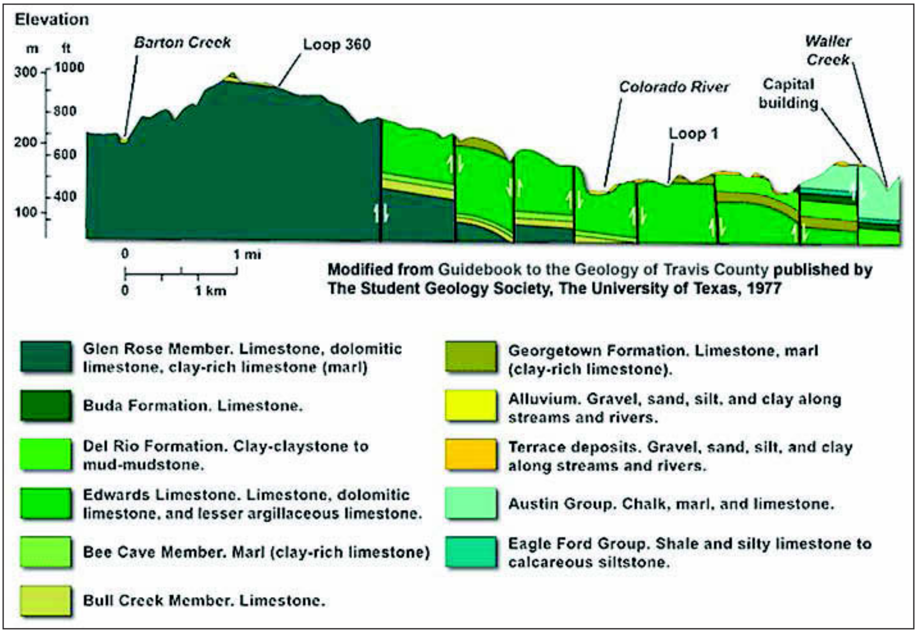

Figure 3. Geological profile through Austin, Texas

a tail shield and a rib and lagging support and thrust system were utilized. In the Taylor rock where the slickensided zones are presented, the capability for quick installation of support was required to prevent falling. For most of the tunnel in Austin Chalk outside of the poor ground areas, primary support was installed only at one location where a fault brought shale between the tunnel crown and springline for a short tunnel stretch, and steel sets, rock bolts and mining pans were installed.

The ACW 5029-3 contract began in the fault zone and continued west into the Glen Rose Formation. No primary support was installed. The only ground problem noted was associated with the occasional presence of about 1-m-thick marly layers, possibly a thin layer of Grayson Marl. A general geological profile through Downtown Austin along ACW tunnel is shown in Figure 3.

Similar to ACW 5029-1, the OCW Section II was excavated in the Taylor and Austin groups, including pyroclastic deposits generated from Pilot Knob, and one locally-occurring limestone unit (the McKown Formation). Several creek crossings on the order of about 3-m-thick ground cover were planned and some crossings had even less than 1-m-thick bedrock cover. Due to the presence of low strength, and low RQD pyroclastics/claystone encountered in the vicinity of Pilot Knob, the selected excavation support scheme consisted of rib and lagging support regardless of ground conditions. Due to the use of conservative initial ground support design, no major tunnel support problem was reported during construction.

The OCW Section IV tunnel was excavated in the Taylor Group, similar to ACW 5029-3, proceeding from the location of a sewage treatment plant near the Colorado River, to the southwest to connect the OCW Section II tunnel near Highway 183. The section also included several Onion Creek crossings. The contractor elected to use stagger double-layered and butt jointed 10-cm-thick plywood sheets as primary support for the upper 270 degrees of tunnel with a poured concrete invert to ensure track stability and prevent rock deterioration. For cases of slickensided, blocky ground, light steel channel sections were on hand to reinforce the plywood. The softened claystone, groundwater, and alluvium were occasionally encountered at the face, sometimes with catastrophic results. The contractor eventually constructed an extra shaft, installed a tail shield, and continued excavation with conventional rib and lagging support. With this excavation system, good production rates were achieved.

In addition, the majority of the central Texas could be classified as potentially gassy ground, particularly in the Austin Group and Eagle Ford Group, where hydrocarbon gas does exist. However, this ground risk is not related to initial ground support; therefore, it is not the focus of this paper.

TUNNEL INITIAL SUPPORT DESIGN AND CONSTRUCTION

There are many ways to approach the design of ground support in rock tunnels. For relatively good rock (high RQD), empirical calculations are often sufficient prove the ground has good stand-up time. In this case, the philosophy for temporary support is to address only local instabilities and enhance safety until the final lining can be installed. The ground is therefore relied upon to be self-supporting. In relatively poor rock (low RQD), it is critical to understand the mechanisms that drive excavation instability so that the most effective form of temporary support is chosen.

As described in section III, the bedrock in central TX typically consists of homogeneous rock mass that is interrupted at random by thin layers of pyroclastics or tuffs. Where tunnel alignments cross these interruptions, local ground support must be installed to fully support the zone of problematic material that may loosen after excavation. Several different methods can be used for this type of support, including half or full liner plate, shotcrete, rock dowels or bolts, and steel ribs with timber lagging. Typical implementation details are shown in Figure 4 through Figure 6.

Characterization of the ground through traditional exploration programs may not be sufficient for determining the most appropriate method of temporary ground support, due to the existence of pyroclastic zones. Additionally, analytical tools such as finite element analysis can become prohibitively complex if the designer attempts to model such features. The most efficient way for a designer to undertake a temporary support design in a reasonable amount of time while maintaining the flexibility appreciated by tunneling contractors is to present a set of acceptable options in the final design documents. This approach allows the contractor to select whichever method he finds appropriate based on the conditions encountered during tunneling. The designer may also wish to designate certain parts of the alignment where a specific support is mandated to the contractor. This is done in areas of heightened uncertainty concerning ground conditions, beneath creeks for example, so that the owner's exposure to a differing site condition claim is minimized and the contractor's safety is enhanced. An overview of the recommended practices for design of several temporary support systems is presented in the following paragraphs in the context of the typical central Texas geology.

Rock dowels are not extensively used in small diameter tunnels in central Texas, unless the excavation is performed in Austin Chalk or Glen Rose Formation where conditions are excessively blocky, because rock dowels are difficult to install in a tight space. However, shaft and turn-under support can be efficiently done using rock dowels and shotcrete lagging with mesh since there is more working space. In larger diameter tunnels, rock dowels are feasible and designed in the typical manner.

For protecting pyroclastic material in particular, a more practical alternative to rock dowels is to install liner plate to fully cover the exposed pyroclastics. To mitigate pyroclastics found only in the crown, it is relatively quick to install a 180 degree strip of liner plate in the crown and secure it at its base using hand-installed dowels. If required, such as excavation in completely weathered Eagle Ford South Bosque shale, full liner plate coverage can also be installed by jacking and bolting. The plate can also be used to drain unexpected groundwater inflows in the temporary condition or even be

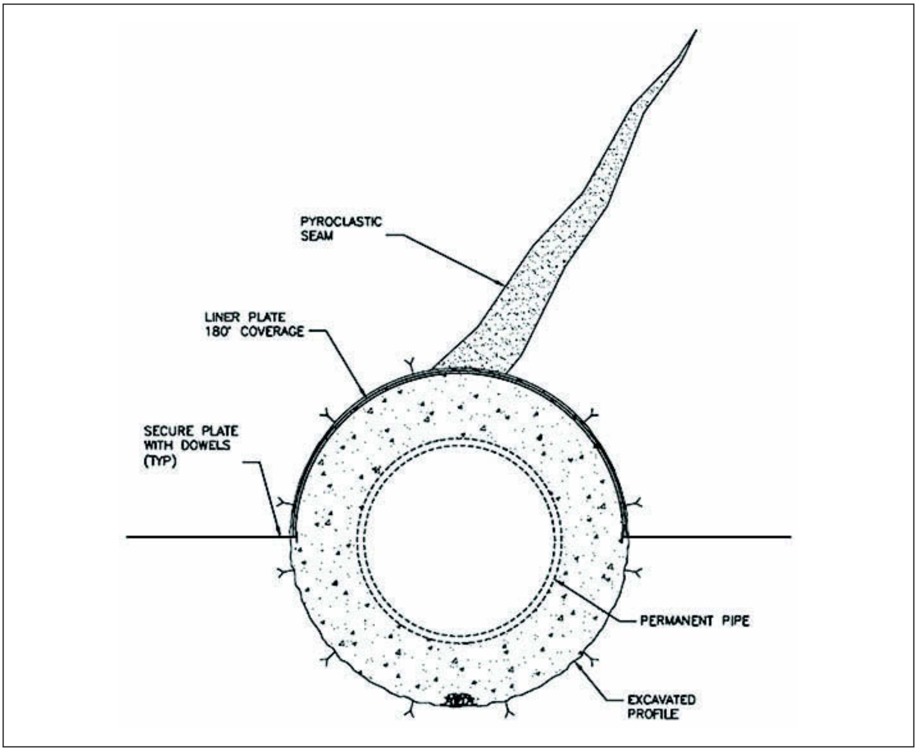

Figure 4. Typical implementation of half liner plate below pyroclastic seam

incorporated into a permanent drainage system, if required. Liner plate may be inappropriate in some situations in TBM tunneling where the rock has insufficient competency to resist thrusting forces during the TBM drive. In these cases, liner plate installed for pyroclastic support would be ineffective in providing a thrusting point for the TBM.

In ground that has more extensive pyroclastic zones, steel rib and lagging support would provide the same overhead protection as liner plate with the added ability to serve as thrust support for the TBM. More importantly, it is prudent for the designer to mandate steel rib support in regions where the ground is known to be compromised, near a tunnel-shaft intersection for example, so the thrusting forces from the TBM have little risk of loosening material.

The designer is cautioned against relying heavily on finite element analysis for determination of support requirements in central Texas geology. Pyroclastic materials can have significantly different elastic and strength properties than the bulk of the rock mass expected, therefore any accurate model would have to make assumptions about the locations and sizes of pyroclastic zones. If the designer considers these zones as horizontally bedded strata, finite element analysis might predict more deformation than actually occurs since these layers are thin, in reality, and more akin to fault zones in the context of numerical modeling. However, since neither fault zone modeling nor standard modeling of horizontally bedded strata can fully represent the behavior of this material, it is recommended that the designer ignore this material when estimating tunnel convergence in a finite element model. These models also rely on the accuracy of joint sets and discontinuities information.

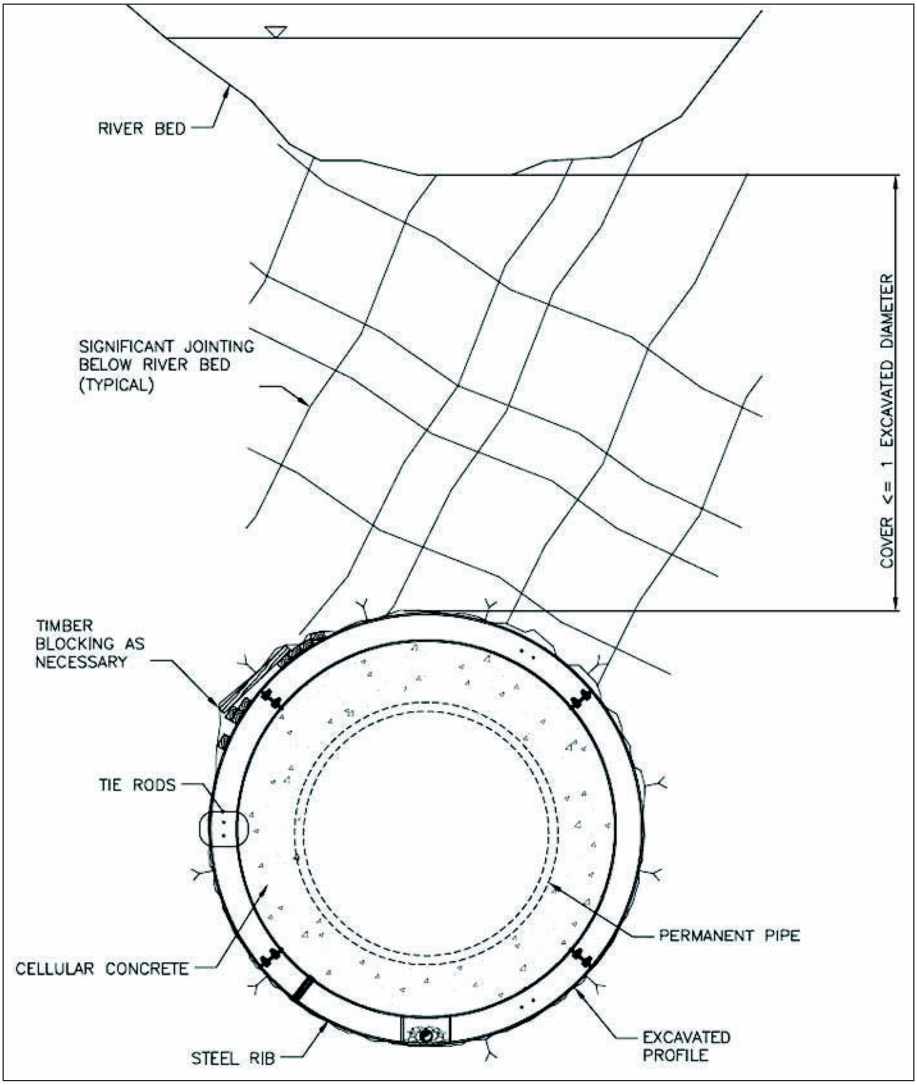

Figure 5. Typical implementation of steel ribs and lagging below a creek crossing

Another consideration when constructing a numerical model in this type of ground is that pyroclastic is often characterized to be quite soft and thus could lead to overestimation of tunnel convergence. In reality, pyroclastic material is more likely to dry after being exposed and exhibit crumbling (overbreak) as opposed to deforming excessively during relaxation. As a result, observed relaxation will be more similar to models with proper jointing and a homogenous rock mass than to models that attempt to implement a pyroclastic seam. Overestimation of tunnel convergence can lead the designer down a problematic path where unrealistically heavy steel ribs are required to maintain excavation stability, prompting the contractor to reassess ground support needs and bringing into question the designers true understanding of the ground. Previous experience in the central Texas geology show that heavy support systems are seldom required except near junctions, creek crossings, and identified local problem areas, such as in

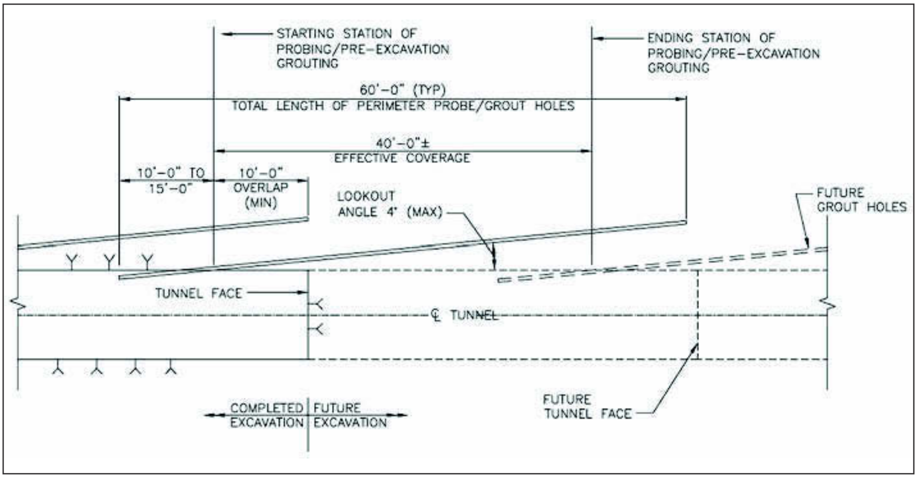

Figure 6. Typical probing and pre-excavation grouting plan

Taylor Claystone. A number of tunnels have been excavated in the Austin Area within the Austin Group without significant ground supports.

Five types of commonly used initial ground support systems inside tunnels up to 4 m in diameter are shown in Figure 4 through Figure 6, and their specific engineering, labor, and logistical features are compared in Table 4. The summary also includes the discussion of a commonly used additional measure (probing and pre-excavation grouting) for ground support and groundwater control. Table 4 serves to consolidate the authors' experiences with initial ground supports in central Texas geology into a single reference to provide holistic guidance in future tunnel project planning and initial ground support design to best serve the Owner, Engineer, and Contractor.

OWNER'S PERSPECTIVES OF INITIAL TUNNEL SUPPORTS AND RISK MITIGATION

During the OCRS and OCGC design phase, the Owner's main risk concerns for the tunneling program were related to construction in the floodplain, potential disruption to surrounding neighborhoods, tunneling under creeks with shallow cover, and construction cost. The risk of local ground support failure in Austin Chalk was not a major concern during the design stage. The Owner provided very limited input on initial ground support but did have some concerns during the bid phase that the original ground support design was excessively conservative, based on the feedback received from potential bidders. The Owner is a strong believer in having a Geotechnical Baseline Report on all tunnel projects in competent ground and allowing contractors to determine their own temporary ground support based on the GBR along with nominal design guidance provided by the Engineer. If the Contractor wishes to proceed with tunneling based on applying their own designs in an "observational" approach, the contractor could proceed with the construction provided that there is no major safety concern involved.

One major exception seen by the Owner as a condition too risky for freedom of contractor means and methods was tunneling under creek crossings and in shallow cover zones. The Owner's past experience with these conditions precipitated a requirement that the contractor use a TBM capable of providing probing and pre-excavation grouting. This requirement was not relaxed during the bidding and construction phases because a failure beneath a creek crossing could be catastrophic. Therefore, it is especially important when the Owner grants this much freedom to the contractor for initial

Table 4. Overview of commonly used initial support systems in central Texas

Des. No.	Support Description	Ground (Rock) Type	Mitigated Ground Risk	Construction Logistical Limitation	Approx. Installation Time
I	Steel Rib & Lagging	Squeezing (Taylor Group Claystone, Bentonite and Marl)	Excessive Overbreak, Excessive Convergence, Rockfalls	Internal space limitations, which affects productivity particularly in smaller tunnels. Difficulties in surveying (such as capturing curves) and water handling if pumps are necessary. Material mobilization adds affects production rate. Benefit to support: No grouting necessary.	20–30 min/Set
II	Liner Plate	Squeezing or low strength highly weathered (Eagle Ford South Bosque Shale)	Light Groundwater Inflows, Excessive Convergence	Relatively easy to install. Grouting is necessary because of the potential for tunnel ovality. Slower installation. Benefit to support: Does not significantly decrease the amount of internal space within tunnel. Material mobilization within tunnel is simple.	5–10 min/plate 10 min for grouting
III	Semi-circle/Half-moon Plate	Competent Rock w/ Pyroclastic materials (Austin Group, Eagle Ford Lake Waco Shale)	Pyroclastics, Rockfalls, Light Groundwater Inflows	Quick installation. No loss of space in tunnel. No grouting required. Material mobilization within tunnel is simple.	15 min/semi-circle
IV	Patterned Bolts	Blocky (Austin Group, Eagle Ford Lake Waco Shale)	Rockfalls	Difficulty mobilizing bolt and drill within smaller tunnels. Drill space is limited. Patterns not always sufficient for ground conditions.	15 min/each
V	Spot-Bolting	Jointed (Austin Chalk, Glen Rose Dolomitic Limestone)	Rockfalls	Similar Limitations as Patterned Bolts, with less bolting.	15 min/each
Add.	Probing and Grouting	Blocky, jointed near Creek crossing, Shallow Bedrock Cover, Fault Zone	Water Inflow, Crown Collapse, Face Instability	Entirely dependent on the ground condition and its susceptibility to groundwater. Water is the deciding factor. Dewatering is involved and substantially reduces production.	Varies based on amount of groundwater encountered.

Notes: All installation times are approximate depending on the site specific ground condition.

support design that the designer identify any and all of the Owner's concerns and clearly state them in the specifications. This way, the contract documents are written so that the Owner only exercises control where risks are believed to be high and the contractor is free to innovate elsewhere.

SUMMARY AND CONCLUSION

Small diameter tunneling is gaining popularity in metropolitan areas, such as Austin, TX, that are experiencing steady population growth and have an abundance of favorable geology for TBM tunneling. Owners are benefitting by providing durable infrastructure that is invisible to the general public and building solid framework for residential and commercial development while leaving the surface clear for more parks, landscaping, and green spaces. However, the Owner's local knowledge and the Contractor's local experience play a large role in making these medium to small budget underground projects financially successful. Contract documents must be developed in such a way that this local knowledge is not stifled in deference to unnecessary risk mitigation by the designer. Another good example of an ongoing City of Austin tunnel project has also proven this point; the 10.5 km, approximate 3-m-excavated diameter Jollyville Transmission Main tunnel in Glen Rose Formation has almost no ground support needed, and only spot bolting and wire mesh used so far for ground supports in karst features.

The cornerstones of such an approach include thorough investigation of previous tunneling experience in the local geology, a complete understanding of the Owner's risk tolerance, and development of contract documents that leave tunneling in good ground up to the Contractor and tunneling in risky ground appropriately constrained by the designer. Finally, the contract documents must require sufficient measures from the contractor so that unexpected changes in ground condition can be identified early and planned for. Probing and pre-excavation ground capability is one example of a contract requirement used in central Texas that gives the contractor a means to identify unexpected ground conditions early and avoid dangerous repairs that can incur devastating costs to the owner. Similar measures can best be identified for small diameter tunnels in any other locale with good ground through extensive historical research.

It should be noted that small diameter tunnels are most efficiently designed and constructed in different ways from large diameter tunnels. During the design phase, large diameter tunnels reap great benefits for the time spent on finite element analysis, where small diameter tunnels often gain little in cost savings through the investment in finite element modeling, as experience has shown in central Texas. In fact, these analytical methods can even be counter productive in situations where the Owner and contractor have extensive tunneling experience in the local ground. In these cases, the designer is urged to consult previous tunnel jobs and build on the library of local tunneling knowledge already in place.

ACKNOWLEDGMENTS

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LESSONS LEARNED FROM 130 YEARS OF TUNNELING IN SEATTLE'S COMPLEX SOILS

Robert A. Robinson ▪ Shannon & Wilson, Inc.

ABSTRACT

Seattle has experienced 130 years of increasingly challenging tunneling with more than 150 tunnels totaling over 120 km beneath hilly topography and through complexly interbedded glacial and inter-glacial soils. Local geotechnical challenges include: multiple perched groundwater levels, abrasive granular soils, sticky clogging clays, high strength boulders, "mixed-face" conditions, isolated methane inflows, and man-made obstructions and various contaminants. To handle these conditions local tunneling has evolved through at least four phases defined by: (1) hand-mining and timber support augmented with compressed air ground stabilization; (2) excavation with pneumatic spaders, electric and diesel powered equipment, chemical grouting, and mechanized concrete shuttle forms; (3) digger shields with steel and concrete segment support, deep well and eductor/ejector dewatering, waterproof membranes, jet and compaction grouting, and risk sharing with baseline reports and dispute review boards; and (4) today's closed-face tunnel boring machines (TBMs) with on-line monitoring and single pass gasketed pre-cast segments, all capable of handling over 60 m of groundwater head. Current and recent projects include TBMs with excavation diameters of 1.5 m to 17.5 m and capable of working at groundwater pressure of over 6 bars.

INTRODUCTION

By 1900, the population of the Seattle area had grown to over 110,000. About 60 regrade projects (Morse 1989) involving the movement of over 50 million tonnes of soil from the tops of steep hills down to fill in shallow areas at the waterfront, provided new areas for development and prompted a population explosion. However, the several remaining north-trending ridges created a challenge for construction of gravity sewer systems, and consequently most raw sewage outfalls were to lakes and rivers. By the early 1900s a typhoid epidemic and fears of a cholera outbreak, caused by contamination of drinking water sources in nearby lakes and rivers, prompted the construction of over 35 km of new sewers, much of it in tunnels, to transport wastewater by gravity through the ridges to Puget Sound.

Over the last 130 years, more than 150 tunnels totaling over 120 km of utility and transportation routes have been successfully excavated in the Seattle area. These tunnels were constructed through a wide range of geotechnical conditions, primarily glacial and non-glacial soils with multiple perched groundwater tables, utilizing a variety of tunneling methods. While the techniques for excavating and supporting tunnels have undergone dramatic changes, the soil and groundwater conditions have remained much the same. Consequently, past local tunneling experiences are pertinent to any construction approaches that might be envisioned for future tunnels. This paper discusses prior phases of tunneling as presented in Robinson, Cox, and Dirks (2002) but focuses on the current phase of tunneling started in the late 1980s with the introduction of pressurized closed-face tunnel boring machines (TBMs). In the last 10 years, over 35 tunnels totaling about 39 km having been completed, and another 18 km of tunnels are either in design or under contract. Figure 1 presents a map of selected

Figure 1. Selected tunnels constructed or planned in the Seattle area since 1894 (see Table 1 for map number identification)

tunnels constructed and planned in the Seattle area beginning in 1894 up to the present. Table 1 presents a partial listing of these tunnels, along with brief discussions of their notable characteristics.

Tunneling technology in Seattle has reflected the development of new methods for soft-ground tunneling in other parts of the world. Early tunnels were hand-mined with initial timber supports and permanent linings of brick (e.g., Lake Union Sewer Tunnel, 1894). In the early 20th Century, tunnels were often constructed under compressed air (North Trunk Sewer, 1910) to stabilize wet flowing soils. Current tunneling involves much more sophisticated tunnel boring machines, although compressed air is often needed for maintenance access to cutterheads, and starter tunnels and chambers may be excavated by backhoes and supported with steel ribs instead of by shovel and with timber ribs.

Selected significant historic local tunneling innovations that have been implemented are as follows:

- Compressed air stabilization of groundwater and flowing soil (North Trunk sewers, 1905)

Table 1. Selected tunnels constructed or planned in the Seattle area since 1894

Map No.	Tunnel	Date Done	Size	Length	Excavation Method	Support Method
1	Lake Union Sewer	1894	1.8 m I.D.	1,749 m	Hand tools, wheelbarrows	Timber ribs & brick
2	South Bayview St. Sewer	1894	1.2 m by 1.8 m	1,380 m	Hand tools, wheelbarrows	Timber ribs & brick
3	Great Northern RR	1905	11.6 m O.D.	1,570 m	Multiple 10 ft drifts, electric loci	Timber ribs & concrete
4	Ravenna Sewer	1910	2.1 m I.D.	877 m	Compressed air. Tried TBM	brick, relined with steel pipe
5	Fort Lawton sewer	~1911	3 m I.D.	2,963 m	Hand tools, wheelbarrows, rail	Timber ribs, concrete & brick
7	Grenn Lake Sewer	~1911	~ 2.1 m I.D.	~150 m	Hand tools, wheelbarrows, rail	Timber ribs, concrete & brick
6	Dexter and 8th Ave Sever	1912	2.5 m I.D.	2,840 m	Hand tools, wheelbarrows, rail	Timber ribs & brick
8	Washington Park Sewers	1912	2.5 mI.D.	1,235 m	Hand tools, wheelbarrows, rail	Timber ribs & brick
9	Freemont Sewer Siphon	1913	~4.6 m O.D.	152 m	Hand tools, electric lights	Timber ribs, concrete
10	Montlake Sewer Siphon	1912	1.2 mI.D.	611 m	Hand tools, wheelbarrows, rail	Timber ribs & brick
11	Connecticut Ave. Sewer	~1912	0.9 to 1.8 m	2,152 m	Hand tools, wheelbarrows, rail	Concrete pipe on timber piles
12	Pacific St Sewer	~1910	2.7 m I.D.	3,453 m	Hand tools, wheelbarrows, rail	Timber ribs, concrete & brick
13	Wallingford Sewer	~1913	2.7 m I.D.	550 m	Hand tools, wheelbarrows, rail	Timber ribs, concrete & brick
15	Latona Syphon Water Main	1917	~13 m I.D.	~305 m	Hand tools, electric lights	Timber ribs, concrete, steel pipe
18	South Hanford St. Sewer	1930	4 m O.D.	1,846 m	Air spaders, gasoline powered	Timber lining, concrete
19	Charleston Street Sewer	1931	3.5 ft . I.D.	863 m	Air spaders & mine cars	Timber lining, concrete
19	Laurelhurst Trunk Sewer	1936	9 ft I.D.	564 m	Air spaders, mine cars,	Timber sets, brick concrete
20	SR-20 Highway	1941	twin 8.5 m I.D.	405 m	Spaders, steam shovels, mine	timber ribs, concrete
22	Mathews Beach Sewer	1967	3.4 m O.D	5,335 m	Wheel shield & digger shield, dewatering, chem. & cement	220 kPa air, steel ribs, concrete
23	2nd Avenue Sewer	1968	3.8 m O.D.	6,067 m	20 - 124 kPa air, shield, chem	Steel ribs, concrete
24	Kidney Center Ped	1975	3 m horseshoe	37 m	Small frontend Loader, spaders	Steel ribs, concrete
25	Beacon Hill Waterline	1984	3.7 m O.D.	46 m	Loader, pneumatic spaders	Steelliner plates & concrete
27	Mt. Baker Ridge I-90 Tunnel	1986	24 stacked drifts (2.9 m)-	405 m	2 digger shields (several boulders 2 to 8 ft diam)	Expanded concrete segments, mass concrete fill of each drift
26	Downtown Seattle Bus	1988	twin 6.5 m O.D.	4,154 m	Digger shield, dewatering wells	Junk segs. , PVC, concrete
29	West Point/ Ft. Lawton sewer	1990	5.1 m O.D.	2,561 m	Partial EPBM, flood doors, pressure relieving gate, & muck ring - never converted to auger	Steel ribs, filter fabric, sections of gasketed steel liner plate, final concrete
30	L. Washington Ship Canal Siphon	1993	1 m O.D.	463 m	Microtunnel	Steel pipe - welded joints
31	Royal Brougham St. Sewer	1993	3 m O.D.	91 m	Pipe jacking & well point	Reinforced concrete pipe
32	1st Avenue Utilidor	1995	3 m O.D.	248 m	Microtunneling, frozen shafts	Reinforced concrete pipe
32	Lander St. Sewer	1995	3 m O.D.	~91 m	Pipe jacking & well point	Reinforced concrete pipe
33	Eastlake Storm Sewer	1997	1.1 m O.D.	145 m	Microtunneling (laser directed)	Jacked concrete pipe
34	W. Seattle/Alki Sewer	1997	4 m O.D.	3,201 m	Partial EPBM, conditioners	Gasketed concrete segment
35	W. Duwamish R. Crossing	1997	2.4 m O.D.	267 m	Microtunneling	Steel pipe
36	East Duwamish R. Sewer	1997	2.4 m O.D.	213 m	Microtunneling	Steel pipe
37	Denny CSO/Mercer St.	2002	5.1 m O.D.	1,894 m	EPBM w/ conditioners	Precast concrete segment
		2001	1.8 & 2.4 m	252 m	3 Pipejacks	Jacked concrete pipe
		2002	1.8 m I.D.	793 m	3 Microtunnels	Jacked concrete pipe
38	Beacon Hill Transit Tunnels	2008	6.4 m O.D. twin tunnels and up to 12.2 m O.D.. Station tubes.	1,524 m	EPBM w/ conditioners for running tunnels and Sequential Excavation Method for station tunnels. High pH required	Bolted, gasketed concrete segments for running tunnels and shotcrete and lattice girder ribs for SEM
39	Brightwater Sewer Tunnels	2009	1.8 m I.D. pipe	741 m	3 Microtunnels	
		2010 to 2012	4.7 to 5.9 m O.D.	20,351 m	2 EPBM and 2 slurry pressure balance for 3,537 to 6,402 m	1st use in NW of steel fiber reinforced segmets.
41	University Link Transit	2012	6.4 m O.D. twin	5,070 m	2 contracts with 3 EPBMs	Gasketed concrete segments
42	Fremont Sewer Siphon	~ 2014	Twin 1.5 m O.D.	146 m	Microtunnel	Jacked steel pipe
43	Ballard Sewer Siphon	~ 2014	2.6 m O.D.	549 m	EPBM	2.2 m I.D. oncrete slip lining
44	Alaska Way Viaduct Replacement Tunnel	~ 2016	17.5 m O.D.	2,683 m	EPBM	Gasketed concrete segments
45	Northgate Link Transit	~ 2021	6.4 m O.D. twin	6,922 m	EPBM	Concrete segments

- Electric powered locomotive for muck removal (Great Northern Tunnel, 1905)
- Mechanically expandable steel shuttle form for concrete lining (Henderson Sewer Tunnel, 1937)
- Sodium silicate grouting (Ravenna Trunk Sewer Tunnel repair, 1957)
- Compressed air guidelines that were eventually adopted by OSHA (2nd Avenue Sewer Tunnel, 1967)
- Stacked-drift, semi-flexible compression ring liner (Mt. Baker Ridge Highway Tunnel, 1986)
- Waterproofing membranes and jet grouting (Seattle Bus Tunnel, 1987)
- Slurry pressure micro-tunneling (First Avenue Utilidor, 1995)

- Ground freezing for shaft construction (First Avenue Utilidor, 1995)
- Gasketed concrete segmental linings (West Seattle Sewer Tunnel, 1995)
- Full earth pressure balance machine (Denny Way/Lake Union CSO, 2002)
- Cross-hole tomography in horizontal directional drillholes (Henderson/M.L. King CSO, 2000)
- Sequential excavation method for large tunnels (Beacon Hill Light Rail Station, 2008)
- Slurry pressure balance machine (Brightwater Conveyance Tunnels, 2012)

REGIONAL GEOLOGY

Seattle is located in the center of the Puget Sound Lowland, a north-south elongated topographic depression bordered by the Cascade Mountains on the east and the Puget Sound on the west. The Lowland is characterized by low-rolling relief with deeply cut ravines, river valleys, lakes, and north trending ridges ranging from 90 to 150 m above sea level.

The Puget Lowland has been filled by about 1 km of glacial and interglacial sediments during the Pleistocene Epoch (2 million years ago to about 15,000 years ago); however, scattered outcrops of sedimentary and volcanic rock occur south of an east-west line extending from Issaquah on the east, through downtown Seattle and across Puget Sound to Bremerton on the west (Troost et al. 2004). This east-west boundary is now recognized as the active Seattle Fault. Within the last 25 years, several other active faults have been identified in the area (Mace and Keranen, 2012).

Primary Soil Units

Six or more major glaciations have been recognized in the Puget Sound area during the Pleistocene Epoch, during which the glacial ice loading reached thicknesses of up to one kilometer. The Pleistocene stratigraphic record in the Seattle area is a complex sequence of glacially-derived and interglacial sediments. Partial erosion of older deposits, followed by local deposition of more recent sediments, further complicates the geologic setting. The primary glacially overridden soil units include:

Glacial Till

Gravelly, silty to clayey sand with cobbles and scattered boulders with a very soft rock consistency and forming near-vertical bluffs up to 20 m high. Till has been excavated by backhoes with heavy-duty ripper teeth, hoe-rams, roadheaders, soil tunnel boring machines (TBMs), and even light blasting.

Glacial Outwash

Dense to very dense, clean to silty, fine to medium sand with traces and lenses of coarse sand, gravel, and cobbles. Saturated outwash soils may flow viscously into open excavations, but may also yield large amounts of groundwater in water wells.

Glaciolacustrine Clay

A mix of hard to very hard glacially derived silt and clay deposited in a lake or slow river environment in proximity to an advancing or retreating glacier. Contains scattered lenses of gravel, cobbles and boulders.

Glaciomarine Drift

A mix of glacially derived debris consisting of a clay and silt matrix with variable quantities of sand, gravel and boulders deposited in a marine environment. This over-consolidated soil spans the range of characteristics from glaciolacustrine clay to till.

Interglacial Deposits

Alluvial, lacustrine and fluvial clay, silt, sand, peat, and gravel layers are interspersed between the various glacial units. These soils have also been glacially overridden and are hard to very dense.

Geotechnical Issues

The following paragraphs discuss subsurface conditions that have proven to have a significant impact on the selection of tunneling methods and the resulting success of tunnel construction in the Seattle area.

Boulders and Cobbles

Cobbles and boulders are present in most glacial and interglacial soils, and may comprise up to 10 percent of the till and glaciomarine drift and up to 2 percent of other soil units. Unconfined strengths of cobbles and boulders range from 100 MPa to over 300 MPa, making them difficult to breakup and highly abrasive, although TBMs equipped with disc cutters are capable of breaking up and excavating most boulders (Dowden and Robinson, 2001).

Logs

Buried logs have been encountered by several tunnels in the Seattle area beginning with construction for the Great Northern Railroad Tunnel in 1905 that exposed a “buried forest” in interglacial soil deposits. Logs in older Pleistocene deposits are often decayed and can be easily cut up by tunneling equipment but may clog slurry lines, mud tanks and slurry filtering systems. Logs buried in more recent deposits are often relatively fresh and excavate as long strips of wood that tend to clog the cutterhead and slurry lines.

Man-Made Obstacles

Abandoned tiebacks, steel and timber piles, unmapped utility tunnels, buried riprapped shorelines, random fill, well casings, steel cables from logging and industrial facilities, and other man-made debris have been encountered during tunneling. As anticipated, the digger shields for the Bus Tunnels encountered over 500 abandoned steel tiebacks that had to be cut off when encountered beneath Third Avenue.

Soil Contacts

Contacts between soil units tend to be relatively abrupt, undulating and non-horizontal. Tunnel excavations have encountered mixed-face conditions as they pass from relatively hard till or clay into wet sands, resulting in steering control difficulties and increased ground losses and settlements. Undulations in the contact pool groundwater, making complete dewatering of the overlying sand difficult. Sufficient explorations should be performed to define the frequency of contacts and percentages of each soil type and each combination of soil soils types that will occur along the tunnel alignment.

Perched Groundwater Levels

Permeabilities for various soils vary over several orders of magnitude, and interlaying of soil units typically results in multiple perched water levels in borings, shafts, and tunnels. On the Bus Tunnels, Beacon Hill and University Link Light Rail projects, and several other projects, localized dewatering systems and grouting was used to reduce groundwater and soil inflows from multiple perched groundwater levels.

Fractured to Sheared Clay

Scattered joints and shear zones have been encountered in plastic portions of pre-Vashon age clays in several tunnel projects in the Puget Lowlands (Mt. Baker Ridge Tunnel, Brightwater Conveyance tunnels, Link Tunnels, and pending Alaskan Way Viaduct Tunnel). These fractures cause the hard soils to behave like fractured soft rock, with slabbing and wedge failures occurring during or shortly after excavation.

Sticky Clogging Clay

The plastic clays tend to be moderately to highly sticky, adhering to portions of the muck chamber, blocking the rotation of disc cutters, clogging augers, and adhering to conveyor belts and muck cars. Clogging potential of these soils can be defined with Atterberg Limit Tests. On the Beacon Hill project, about ⅓ of the clays were moderately to highly clogging.

Abrasive Granular Soils

Most glacial and interglacial granular soils have been derived from metamorphic and igneous rock and consequently have a high quartz content, resulting in high rates of abrasion and wear on metal surfaces. High rates of abrasion have occurred on several recent projects involving rotary head TBMs, including heavy wear of slurry pumps on microtunneling machines for the Brightwater Conveyance Project, heavy abrasion of the cutterhead perimeters on the Alki Sewer Tunnel, Denny CSO Tunnel, and three of the four Brightwater Conveyance tunnel boring machines (two Herrenknecht slurry TBMs, and one Caterpillar EPBM). On the Alaskan Way, Brightwater, Northgate and University Link projects, abrasivity of soils has been categorized with the Miller Abrasion and the Norwegian Soil Abrasion tests (Kohler et al., 2011). X-ray mineral diffraction for quartz content, and optical counts of quartz grains and grain shape are also useful for assessing abrasivity.

Methane

Methane gas has been encountered in borings and intermittently on several tunnel projects in the Seattle area. The Mathews Beach, Fort Lawton, and West Seattle Sewer Tunnels encountered pockets of methane that were controlled with normal tunnel ventilation and shutdowns of a few hours. Most recent tunnels in the Seattle area have been classified as “potentially gassy,” but the Denny CSO, Brightwater Conveyance and Link LRT tunnels experienced no gas-related shutdowns, possibly because of the combination of closed-face TBMs, gasketed segmental concrete linings, and high ventilation rates.

Natural Contamination and High pH

Laboratory tests have indicated higher than permissible concentrations of naturally occurring arsenic, asbestos, and various heavy metals in some soils, likely derived during their erosion from igneous and metamorphic bedrock. These naturally occurring materials have occasionally limited groundwater disposal. Glacial soils also tend to have an elevated pH, ranging from 6.5 to over 9. Spoils from the Beacon Hill Tunnel were frequently measured at 9 to over 11, likely due to the use of alkaline surfactant soil conditioners and grouting. Washington State statues (Chapter 173-350 WAC) requires that construction waste with a pH over 8.5 be dumped at more remote and costly disposal sites.

TUNNELING IN THE SEATTLE AREA

Seattle's hilly topography was the original motivation for tunneling, and current dense development of the ground surface has continued that trend. Once a tunnel alternative is considered, then the geologic conditions, including the factors discussed above, should be assessed for their impact on the selection of tunneling methods. With advancements in metallurgy, laser guidance, hydraulics, concrete additives and soil conditioners, concrete segment forming, hardened computers for tunnel environments, and a number of other areas, there have been creative suppliers, contractors, designers, and owners willing to try new and innovative means and methods of tunnel construction to cope with the difficult ground conditions in the Seattle area, and making the tunneling process safer, more efficient and better able to handle more challenging conditions.

A wide range of tunnel diameters and lengths have been used for various infrastructure applications. The longest combined lengths of tunnels to date have been constructed for sewers. Recent government mandates for reductions in storm water related sewer overflows into surface waters have fostered a need for new storm and sanitary sewers. Tunnels have also been built for railroads and highways. Small-diameter tunnels have been built for gas, water, power, and fiber-optic lines. Several short tunnels have been constructed for pedestrian walkways. Major projects are currently underway involving several kilometers of tunnels for light rail, sewers, and highways.

Tunnel construction methods during the past 130 years can be divided into four separate historic phases of tunnel excavation and support. A more thorough discussion of the first three phases of tunnel construction is presented in Robinson (et al. 2002). This paper briefly describes those initial three phases, and then focuses on the current fourth phase of tunnel construction.

Phase 1 Tunneling—1880 to 1925

Prior to about 1925 tunnels were excavated primarily with picks and shovels, without the benefit of a protective shield, and the muck removed with wheelbarrows or rail-mounted cars pushed by men, pulled by mules, or occasionally hauled by electric or diesel locomotives. Ventilation, if used, was powered by animals at ground surface.

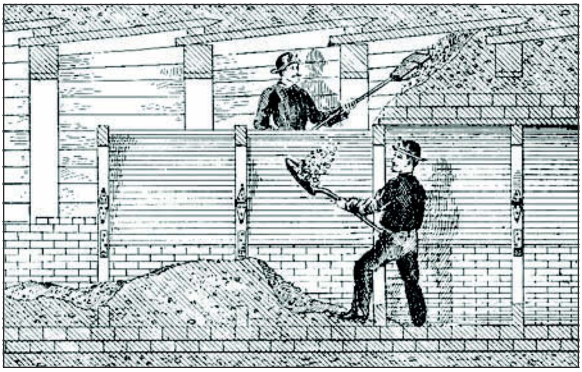

Figure 2. Excavation and support of the Lake Union Sewer Tunnel (City Engineer of Seattle 1893)

Figure 3. Pneumatic spaders used in the hard clay heading of the timber supported Henderson Street Sewer Tunnel, ca. 1936 (Seattle Municipal Archives Photograph Collection Item No. 10381)

Dewatering was accomplished with pipes hammered into the tunnel heading or groundwater inflows were controlled by working under compressed air. Initial support was with timber sets, timber lagging, and steel rail or timber spiling and forepoling. Final support was provided by brick and/or concrete lining. Settlements above these tunnels were generally measured in centimeters to meters. At least 20 tunnels, totaling over 20 km of alignment were constructed during this phase. The first tunnel recorded in Seattle is the Lake Union Sewer Tunnel (Figure 2), a 1,749 m long hand excavated 2.5 m square tunnel lined with brick and still in use. Over a dozen sewer tunnels were constructed as part of 37 km of new sewers. Several water tunnels and the state-of-the art, stacked-drift 11.5 m diameter Great Northern Railroad Tunnel were also constructed.

Phase 2 Tunneling—1925 to 1960

Tunnels were generally excavated with pneumatic spaders augmented in large diameter tunnels with a steam shovel, but without a protective shield. Muck was removed with rail mounted cars hauled occasionally by gasoline and subsequently by diesel locomotives. Compressed air pressures up to 220 kPa, augmented with well points and pumped deep wells were used on some projects. Initial support transitioned from timber ribs to steel ribs after 1946. Final support was provided by a concrete lining. Settlements above these tunnels were generally measured in centimeters to meters. At least 20 tunnels, totaling over 7 km of alignment were constructed during this phase. Examples of this construction phase included the S. Hanford St. Tunnel (telescoping metal forms), SR-20 Highway Tunnels (metal shuttle forms, steam shovel, stacked drifts), and Henderson Trunk Sewer tunnels shown on Figure 3.

Phase 3 Tunneling—1960 to 1990

Advances between 1960 and 1990 in the mechanization, safety, and efficiency of tunneling prompted the construction of a number of tunnels that had previously been considered to be infeasible. Movable cylindrical steel “shields” were introduced to protect the workers, support the tunnel face as the tunnel was advanced, and as on the Lake City and 2nd Avenue sewer tunnels to provide a housing for an excavator such as a full-face rotating cutterhead or a backhoe or “digger,” respectively. Compressed air was

Figure 4. Mt. Baker Ridge Tunnel on Interstate 90 complete in 1986 by a unique stacked drift method to form a semi-flexible compression ring liner. The 24 cylindrical drifts are 2.5 m diameter and the excavated core is 19.8 m diameter.

still used to control groundwater flows on several tunnels, but groundwater levels were often partially lowered with deep dewatering wells and well points. The compressed air specifications introduced on Seattle tunnels in the 1960s were eventually modified for a national compressed air standard. Two-pass lining systems were used, consisting of temporary steel ribs and lagging or precast concrete "junk" segments, followed by a cast in-place concrete lining, with or without an intervening waterproofing membrane. Muck was removed by rail-mounted diesel powered muck trains. Ventilation using fans and metal ducting was carried forward with the advancing tunnel. Safety rules prohibited gasoline powered engines, due to high concentrations carbon monoxide in the exhaust. Innovations also included the introduction of Geotechnical Data Reports (GDR) and Geotechnical Baseline Reports (GBR) as part of the contract documents, and Disputes Review Boards (DRB) and escrowed bid documents, resulting in better informed bidders, and a fairer and more equitable contracting approach. Settlements over these tunnels were generally measured in centimeters. Over 60 tunnels, totaling about 50 km were completed in this phase, including the Mt. Baker Ridge Highway Tunnel shown on Figure 4, which is still the world's largest tunnel in soil at 19.8 m I.D. (Robinson, et al. 1987).

Phase 4 Tunneling—1990 to Present

A wide variety of new tunnel methods and equipment were developed and introduced in the 1990s to present day. Most notable with regards to Seattle tunnels was the introduction of earth pressure balance (EPBM) and slurry pressure balance (SPBM) machines for water-laden flowing silts and sands, with groundwater heads of several bars. Further advances in the 1990s included the addition of rock disc cutters on these soil machines to grind up boulders (Dowden and Robinson, 2001), more reliance on soil conditioners (polymers and surfactants), improved metallurgy for increased longevity of cutterhead and cutters, operation of closed-face TBMs to control ground and water pressures, hardening of computers for underground use on TBMs, enhanced measurement and communication of data from hundreds of sensors on the TBM used to monitor machine performance, and the use of single-pass gasketed, bolted, segmental linings. Advances in geotechnical exploration systems such as the use of sonic-core drilling techniques, downhole in situ pressure meters and cross-borehole tomography have enhanced the information gained from explorations. Settlements over these tunnels are generally measured in millimeters. Over 55 tunnels totaling over 49 km were

Figure 5. Gasketed segmental lining installed behind trailing gear of the EPBM in the Mercer Street Tunnel

constructed during this phase of tunneling. The following examples of tunnel construction in the Puget Sound area in the last two decades were selected to exemplify the application of closed-face tunneling methods in glacial soils and lessons learned from these projects:

Denny CSO/Mercer Street Tunnel

Construction of this 1,882.4 m-long, 5.1 m-O.D. tunnel beneath Mercer Street comprised the first use of a fully functional EPBM in the northwest (Cochran, et al., 2005). Ground conditions consisted of mixed glacial and inter-glacial soils and groundwater levels ranged up to 19 m above tunnel crown. A full EPBM was specified to adequately limit ground losses and resulting settlements. Alignment and EPBM operational parameters were continuously monitored by the machine operator and transmitted to the construction management team with a computerized TACS guidance and data acquisition system. Support of the tunnel was provided by a gasketed, bolted and pinned, segmental concrete lining, as shown on Figure 5.

Heavy abrasion of the cutterhead and cutters stopped the machine after about 850 m of advance for over a month for full replacement of the completely eroded 5 cm thick hardened steel rim-bar around the perimeter of the cutter-head and replacement of all cutters. Up to this point little or no soil conditioning other than water had been used. However, for the remainder of the tunnel increased use of polymer and surfactant conditioners and weekly inspection an replacement of cutter teeth preserved the repaired condition of the cutterhead and enabled EPBM advance at rates of up to 25 m/day in the highly abrasive till and outwash soils. Settlements were less than 10 mm along most of the alignment due to adequate balancing of external soil and water pressures with the EPBM, along with the abundant use of soil conditioning additives. However, over the initial 30 m of tunnel from the launching portal, settlements of 10 cm occurred at ground surface due to incomplete grouting of the annulus around the liner, and shallow soil cover of 4 to 6 m.

Beacon Hill Transit Project—Link Light Rail

This project consisted of a 1.6 km-long twin-tunnel alignment beneath Beacon Hill, south of downtown Seattle, as part of the Sound Transit light rail transit (LRT) system. The 6.4 m O.D. tunnels and deep mined station were excavated through highly overconsolidated, interbedded, glacial and inter-glacial soils with 4 distinct perched water tables.

A wide variety of soil exploration techniques had been used to assess ground conditions. Conventional explorations included mud rotary drilling with split spoon and Dames & Moore sampling. However, these explorations reveal only a portion of the soil conditions. Additional explorations with a sonic-core drill rig and NQ rock coring techniques yielded continuous samples of the hard and dense soil and provided details on geologic contacts and variations that could not otherwise be determined except by tunnel and shaft construction.

The deep mined station was constructed using the Sequential Excavation Method (SEM) with up to 12.1 m diameter openings in hard fractured and slickensided clay (Akai et al. 2007). Jet grouting and deep well dewatering was used very effectively to

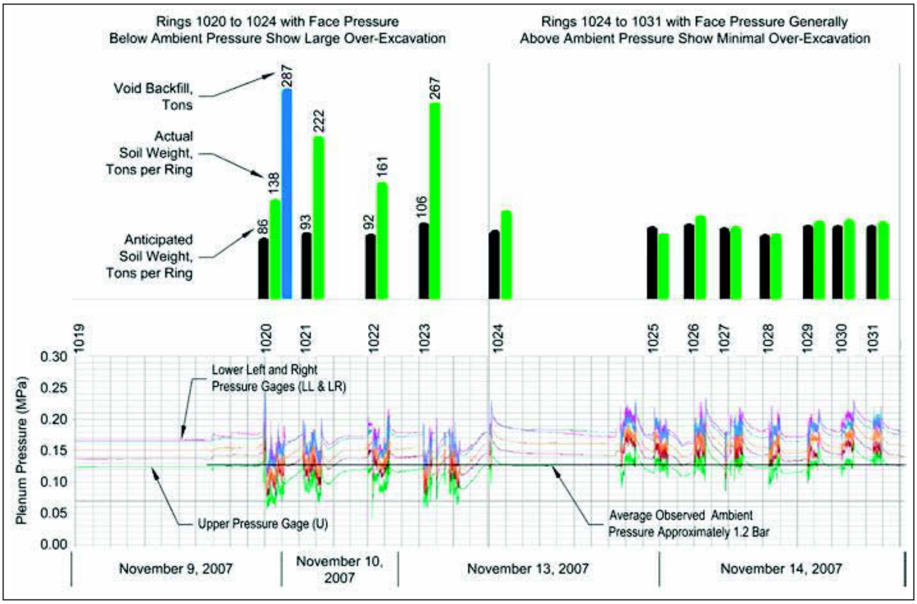

Figure 6. Comparison of muck conveyor belt weigh scale data, plenum pressures, ground losses and grout volumes

replace lenses an layers of flowing sand and enable SEM construction in dry conditions. Settlements above the 48 m deep station were less than 25 mm.

The running tunnels were excavated with an EPBM augmented with water, polymer and foam conditioners. A single EPBM was used for both tunnels, and was launched from the west portals and then relaunched from the east ends of the platform tubes of the SEM station. Surface settlements were less than 10 mm, however, 18 months after completion of the tunnels subsurface cavities were discovered with exploratory drilling at 9 locations and filled with 2,257 m^3 of grout and controlled density fill (CDF). The drilling was guided by data from conveyor belt weigh scales, which were part of the roughly 200 sensor instrumentation package used to monitor TBM operations. The erratic fluctuations of the weigh scales in this area had originally been considered to be erroneous by the contractor. Review of weigh scale data indicated that 2,520 m^3 of excess muck had been removed over a 90 m length of the parallel bores, as the EPBMs were launched eastward from the SEM station. The over-excavation was related to face pressures that were about half of the 0.12 MPa that would have been needed to balance the in situ water and soil pressures over this length of alignment (Robinson et al. 2012). Figure 6 shows the relationship between plenum pressures during each shove and excavated soil weights, relative to theoretical soil excavation weights. When plenum pressures were below ambient water and soil pressures in a mixed-face of wet sand and hard clay, then large over-excavation volumes occurred.

Brightwater Sewer Project

King County has completed the longest continuous tunnel to-date in the Puget Lowland, with a total length of 21km, divided into four tunnel segments and three construction contracts (Gwildis et al. 2009). The hilly topography made tunneling an obvious solution for long portions of the gravity feed sewer system beneath up to 150 m of topographic relief. As with most tunnels in the Seattle area, soil conditions were expected to be highly variable and multiple perched groundwater tables were measured with heads of up to 73 m (7.3 bars) at tunnel crown. Sticky clogging clays combined with abrasive sands and till are reported to have contributed to severe abrasion and wear to the cutterhead rim bar, disc cutters and drag picks. After advancing about 2.1 km to the east and 3.1 km to the west, the two SPBMs on the central two segments were both shut down for nearly a year for repairs. The eastbound SPBM was repaired and successfully completed its 3.5 km segment, but the westbound SPBM was abandoned, and the remaining 3.1 km of tunnel was completed by the western segment EPBM that had already constructed 6.4 km of tunnel. After repairs and modifications to the EPBM were completed at the target shaft to accommodate an expected 7.3 bars of water pressure, the EPBM continued eastward for 3.1 km and docked with the abandoned SPBM in frozen stabilized ground (Gwildis et al. 2012).

Although settlements have reportedly been negligible, likely due to the depth of the tunnels in hard and dense soils, at least one 9 m diameter by 4.5 m deep sinkhole developed due to excess excavation volumes for 5 rings on a night shift.

University Link Corridor Transit Tunnel

The University Link Corridor section of the Sound Transit LRT system includes 5 km of recently completed twin tunnels and three cut-and-cover stations. At the south end, the tunnels pass just 3 m beneath Interstate 5, and at the north end crosses about 2.5 m beneath the bottom of the 24.3 m wide Montlake Cut. The alignment passes through a wide range of highly variable soil conditions consisting of glacially overridden silt, sand, clay, and till. Multiple perched groundwater heads of up to 40 m above crown, combined with possible isolated pockets of methane further complicated construction requirements.

The tunnels were constructed under two contracts with 3 EPBMs. Each EPBM was monitored with up to 200 gauges, including 10 earth pressure gages in the plenum and in the screw auger casings, 2 conveyor belt weigh scales, as well as gages to monitor soil conditioner quantities, advance rate, pressures on all thrust cylinders, and a wide range of operational parameters, all of which were available to the EPBM operators, as well as contractor and construction management staff in nearby offices. Plenum pressure data and conveyor belt weigh scales indicated that pressures were maintained above in situ soil and water pressures, and muck volumes were in reasonable agreement with theoretical excavation volumes. Adequate quantities of soil conditioner were injected into the cutterheads and screw augers, and the cutters were frequently inspected and replaced as necessary. Wear indicators were included on some cutters, and the overcut was measured to assess gage cutter wear. Due in part to this level of care, ground losses were calculated to be less than 0.5% and surface settlements were measured at less than 15 mm (Banerjee et al. 2012).

Alaskan Way Viaduct Replacement

A 17.5 ft O.D tunnel with 2 levels of twin-lane highway was selected for the 2.7km long alignment (Scheibe et al. 2011) to replace an earthquake damaged, elevated, double-deck highway along the west side of Seattle, built in the 1950s. When completed, this will be the world's largest diameter TBM driven tunnel in soil, although still only the second largest tunnel in Seattle, behind the Mt. Baker Ridge Tunnel. Excavation will be accomplished with a Hitachi Zosen EPBM. Tunneling is expected to start in early summer 2013. As presented in the GBR, ground conditions are anticipated to consist of complexly interbedded glacial and interglacial soils, and with a groundwater head 0 to 27.3 m above tunnel crown. The ground conditions are such that the TBM is expected to always be in mixed-face conditions, including hard bouldery till, sticky clogging clay, abrasive sand and gravel, man-made obstructions of logs and random fill debris and several of the other issues experienced on other local tunnels.

FUTURE TUNNEL POSSIBILITIES

Several major tunnel projects are planned in the Seattle area over the next 10 years totaling over 18 km of new alignments. None of these projects would be possible without the recent state-of-the-art advances in geotechnical exploration, tunnel design, tunnel excavation technology, tunnel support systems, and ground improvement techniques. Pending underground projects include:

- Sound Transit's Northgate Link that will extend from the nearly completed University Link segment at Husky Stadium northward to Northgate Mall, a distance of 7 km, and will include 5.6 km of twin 6.4 m diameter tunnels and 2 cut and cover stations.
- Freemont Siphon—Twin 145 m long by 1.5 m O.D. steel pipes constructed beneath the Lake Washington Ship Canal by microtunneling
- Ballard Siphon—550 m of 2.6 m O.D. tunnel to be constructed by EPBM beneath Salmon Bay
- Magnolia Sewer—910 m long by minimum 1 m diameter tunnel to be constructed by horizontal directional drilling along a curved alignment or using the Direct Pipe® method.

LESSONS LEARNED

The Seattle area has benefited from world-wide innovations in tunneling technology over the last 130 years in constructing tunnels through highly varied soil and groundwater conditions. Innovations such as compressed air regulations, sodium silicate grouting, waterproofing membranes, finite element method analyses, Disputes Review Boards, Geotechnical Baseline Reports, soil tunnel support with shotcrete, 19.7 m diameter stacked drift semi-flexible tunnel linings, and large diameter SEM station tunnels have all been applied in the Seattle area. Recent innovations in soil tunneling technologies, such as closed-face TBMs, computerized monitoring of TBM operations, laser guidance, and gasketed and pinned/bolted segmental concrete linings have also been applied by several tunnel contractors to local projects and culminating in the use of a 17.5 m diameter EPBM on the Alaskan Way Viaduct replacement tunnel.

The new closed-face TBMs involve a complex variety of characteristics, capabilities, means, and methods. Consequently, understanding of the ground conditions is even more important now than 40 years ago, due to the impacts of soils properties on design face pressures, types and quantities of conditioner, maintenance and repair frequency, slurry separation plant requirements, etc.

There have been a number of lessons learned from 130 years of tunneling in the Puget Sound area. The following list is by no means comprehensive, but it does provide an indication of geotechnical issues to be considered on all tunnel projects in the Puget Lowlands. These include:

- Complexly interbedded glacial and inter-glacial soils are typical of the Seattle area.
- Multiple perched and artesian groundwater levels occur on many projects.
- Borehole spacings of about 100 m are generally appropriate to assess the variable glacial geology, however spacings of as close as 10 m have been necessary to define geologic variability in very complex geologic conditions.
- New exploration techniques such a sonic-coring, cross-hole tomography and downhole pressuremeters provide useful data on the nature of soil contacts and in situ variability of soil properties as input for sophisticated analyses of soil/structure interaction for tunnel excavation and support.

- Immediate settlement is not a reliable indicator of ground loss, which may take months or years to progress up to the ground surface through glacial soils.
- Granular soils tend to be highly abrasive, resulting in rapid wear of cutterheads, screw augers, slurry lines and pumps and other TBM components unless adequately monitored and maintained.
- Clayey soils may be sticky and tend to clog cutterheads and bind up rotating cutters, and contributing to more rapid abrasive wear.
- Cuttings from timber piles and logs may clog TBMs, slurry lines and separation plants.
- Appropriately used conditioners have proven effective in reducing abrasion, torque and clogging.
- Closed-face TBMs are capable of constructing longer and deeper tunnels than were possible just 20 years ago at water pressures above 6 bars.
- Recent projects have demonstrated the value of real-time reporting and evaluation of monitored TBM operational sensors including plenum pressure sensors, cutterhead rpm, screw auger rpm, conditioner injection volumes, advance rate, and conveyor belt weigh scale data during each excavation and ring build cycle.
- The use of GDRs, GBRs, DRBs, and escrowed bid documents have been effective in establishing a more equitable sharing of risk between owner and contractor.
- Face pressures in closed-face TBMs must be maintained above ambient water and soil pressures to limit over-excavation and ground losses to acceptable values. When face pressures exceed in situ water and soil pressures, ground losses are generally limited to less than 0.5% and associated settlements are small. This requires the continuous review of TBM operational data during mining and the frequent assessment of TBM operating parameters as the tunnel advances through varying soil and groundwater conditions.

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RECOMMENDATIONS ON HOW GEOTECHNICAL BASELINE REPORTS CAN BE PREPARED FOR ROCK TUNNEL PROJECTS

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ABSTRACT

The issues associated with using Geotechnical Baseline Reports on tunneling projects have been the subject of extensive debate for several years. This paper attempts to summarize these issues and it presents research to show that despite the ongoing debate, there appears to have been limited changes or improvements made to address these problems over the last 5 to 10 years. The paper then provides recommendations to help mitigate some of the issues identified, which include recommending changes to what baselines should be used and how they should be presented. In particular it recommends only developing baselines that directly address specific aspects of ground behavior and/or other possible claims, as opposed to simply providing baselines for individual rock properties, which are often inconsistent, open to misinterpretation or have no direct relevance to a potential claim.

INTRODUCTION

Geotechnical Baseline Reports are commonly used on large tunneling projects to help mitigate the risks associated with unforeseen ground conditions. However, despite being commonly used there is still an extensive and ongoing debate regarding their effectiveness and there are still many perceived problems associated with using them.

The intent of this paper is to provide recommendation on how GBR's for rock tunnel projects can be improved. In the first part of the paper we identify and discuss the perceived problems with using GBR's. We then investigate and identify the causes of these problems before providing recommendations on how these problems and issues can be mitigated. The paper focuses specifically on hard rock tunneling projects, however many of the issues identified and recommendations provided are considered to be equally applicable to other ground conditions. It is hoped that this research will subsequently help to improve the effectiveness of future Geotechnical Baseline Reports.

BACKGROUND AND RESEARCH

In preparing this paper we wanted to understand why GBR's were not being more effectively in helping to reduce claims and mitigate risks on many of the recent high profile tunnel projects within the US. As a basic concept GBR's should work well, so we wanted to understand how they could be improved and what are the type and extent of the problems being encountered when using GBR's. We started our research by undertaking an extensive literature review of technical papers and magazine articles, all published over the last 5 years, which discussed the various problems with using GBR's. This allowed us to compile a list of the perceived problems associated with GBR's as shown in Table 1. In the industry, a wide range of problems have developed and we discuss the reasons for and extent of these problems in more detail below.

Table 1. Summary of problems associated with geotechnical baseline reports for rock tunnel projects

Problem			Discussion
Conceptual Issues	1	Is a GBR a *risk transfer* or *risk sharing* tool?	Many believe GBR's are not used to help share risk as intended, but are used to transfer ownership of the ground risk from the owner to the contractor.
	2	GBR's only baseline individual rock properties they *do not baseline ground behavior* or *construction issues*.	GBR's often only provide baselines for individual rock properties and they do not specifically address construction or design issues.
	3	The GBR is often *not consistent* with the rest of the Contract documents.	GBR's should be consistent with specifications, drawings and any other contract documentation.
Baseline Definition Issues	1	It is often unclear what statements in a GBR *can be relied* upon as a baseline.	Extensive interpretation is often provided (i.e., geological sections and/or factual tables) as Appendices and it is unclear if these form part of the baseline.
	2	GBR's do not provide baselines that are *relevant* and they do not adequately describe the conditions to be expected. Many GBR's also baseline an extensive array of geotechnical parameters, including many baselines that are not relevant to tunneling.	While this appears to represent a comprehensive GBR which should protect the Owners, experience shows it often provides more problems and chance of errors, contradiction or misinterpretation by the Contractor (see Table 2 for more information).
	3	Baselines often *conflict* with other baselines or with other information provided in the report.	This can occur when multiple baselines are provided that address the same or similar issues, such as providing Q, RQD and RMR values to describe the same rock type.
	4	GBR's often include overly *conservative* baselines and/or baselines that are not *consistent* with the site investigation data.	GBR's that present conditions that are more adverse, arbitrary and/or unrealistic often are perceived as an attempt by the Owner to unfairly transfer risk to the Contractor. Lists of relevant rock tunnel baselines are shown in Table 5 and Table 6.
	5	Baseline statements are often *indefinite, ambiguous or qualitative*.	GBR statements often use the terms like may, could and possible, which make it difficult for the Contractor to rely upon the statements.
	6	The *assumptions* used in developing many baselines are often not provided, leading to uncertainty and misinterpretation.	For example, many ground classification schemes such as the Q System (Barton, 1974) require certain assumptions to be made,. i.e., the selection of an appropriate SRF value. However, this is often not provided with the baseline, leaving it open to debate later.
	7	How should a baseline be *quantified or presented*? Consultants tend to provide a full range of possibilities. i.e., maximum, minimum, averages, ranges and graphical techniques such as contouring.	Providing multiple baselines for the same property leads to confusion and uncertainty regarding what constitutes a change in a baseline. For example, if a single UCS value at one location is outside of the specified range but the average of all samples is close to the average, is this different?

(table continues)

Table 1. Summary of problems associated with geotechnical baseline reports for rock tunnel projects (continued)

Problem			Discussion
Practical Implementation	1	How *should the baselines* be *evaluated*?	If baselines are exceeded, there is often no guidance given in the GBR as to what should be done and what the Contractor can be compensated for, i.e., direct or in-direct costs.
Practical Implementation	2	How should the baselines be *measured* and *verified* during construction?	The baselines should be presented in a way that are easily measurable during construction, considering the proposed means and methods.

In addition to this literature review we then independently reviewed twenty-five (25) recent GBR's that were prepared for large rock tunnel projects within the US over the last 10 year. We also reviewed a number of GBR's from international tunneling projects in order to compare and identify any differences. In reviewing these GBR's we wanted to see how effective they were in describing the anticipated ground conditions. We also wanted to understand what baselines are typically being provided, how relevant these baselines are, and how they are typically being presented. This review work allowed us to verify the extent of the problems which were identified in our literature review. It was necessary to employ a certain degree of judgment in attempting to quantify how effective these baselines, as we were not involved in preparing these reports ourselves. In selecting GBR's to review, we sought reports prepared by a wide range of consultants and clients from various geographical locations across the US, to avoid any bias and to get a representative picture of how GBR's for rock tunnels are currently being prepared. Based on our research, we have reduced the problems with GBR's into three basic types.

DISCUSSION

Problems Associated with How the Baselines Are Defined and Presented

Our research shows that a common problem with GBR's is that many of the baselines provided are poorly defined. The baselines are not clearly presented; they can often be ambiguous, overly conservative, and irrelevant, they can also be inconsistent or conflict with other contract documentation such as drawings or specifications. Figure 1 was prepared following our review of existing GBR's and helps to show the significance of these problems.

Figure 1 shows a list of baselines that were provided in the Rock Tunnel GBR's we reviewed; it also shows the frequency with which these baselines occurred. It also shows for several of the key baselines (i.e., UCS and Joint Orientation) how realistic or 'useful' the baselines were. We can confirm that baselines are often presented with very wide ranges, which make them of limited use in practice. Our research shows that many of the key rock properties for design and construction are also not always provided; in fact we found that very few GBR's reviewed provided a comprehensive list of relevant baselines. For example Q values, which are a key design and construction parameter for describing rock quality, were only provided in fewer than 25% of GBR's reviewed. In Table 2 we show the relevance and importance of these baselines in terms of design and construction issues.

Our research also highlighted that the way in which baselines are presented is extremely important in helping to reduce ambiguity or contradiction. Baseline values can be presented in a variety of different ways including maximum, minimum and

Category	Baseline Properties	Significance / Relevance
(A) Geological Interpretation	Rock Description (Lithology)	HIGH
	Weathering Grades	HIGH
	Top of Rock / Rock Cover	HIGH
	Rock Mass Types	HIGH
(B) Intact Rock Properties	Rock Mineralogy (incl. Hard Rock	HIGH
	Unit Weight	LOW
	Elastic Properties (E)	LOW
	UCS	HIGH
	Shear Strength	LOW
	Tensile Strength (Brazilian Tests)	LOW
	Point Load Strength	LOW
	Cerchar Abrasivity Index (CAI)	HIGH
(C). Rock Mass Properties & Classification	Strength Properties (φ & c)	HIGH
	Stiffness Properties (E & ν)	HIGH
	Hydraulic Conductivity (Permeability)	HIGH
	Seismic Velocity	LOW
	GSI (incl. Hoek & Brown Constants)	HIGH
	Q System (Barton)	HIGH
	RMR (Bienawski)	HIGH
	Rmi (Palmstrome)	LOW
	RQD (Deere)	HIGH
(D) Rock Jointing	Number & Orientation of Joint Sets	HIGH
	Joint Shear Strength	HIGH
	Joint Spacing	HIGH
	Joint Persistence	LOW
	Joint Condition (incl. water conditions)	HIGH
(E). Faulting & Shear Zones	Number & Location	HIGH
	Orientation	HIGH
	Fault Thickness & Strength	HIGH
(F) Groundwater	Groundwater Levels	HIGH
	Groundwater Inflows (flush)	HIGH
	Groundwater Inflows (steady state)	HIGH
(G) Insitu Stress	Vertical Stress	HIGH
	Stress Reduction Factor (SRF)	HIGH
	Ko	HIGH
(H) Specific Ground Risks	SPT N (for Soft Rock)	AS REQUIRED
	Seismicity	AS REQUIRED
	Slake Durability	AS REQUIRED
	Swelling Potential	AS REQUIRED
	Solution Voids or Features	AS REQUIRED
	Groundwater Chemistry	AS REQUIRED
(I). Ground Behavior	Failure Types	HIGH
	Rock Loading	HIGH
(J) Construction Considerations	Excavation Techniques	HIGH
	TBM Properties (FF, CLI etc)	HIGH
	Construction Sequence	LOW
	Support Req's (pre, initial and final)	HIGH
	Grouting Requirements	HIGH
	Volume of Excavated Material	LOW
	Overbreak	LOW
	Gas	HIGH
	Obstructions	HIGH
	Flooding	LOW
	Contaminated Ground / Groundwater	LOW

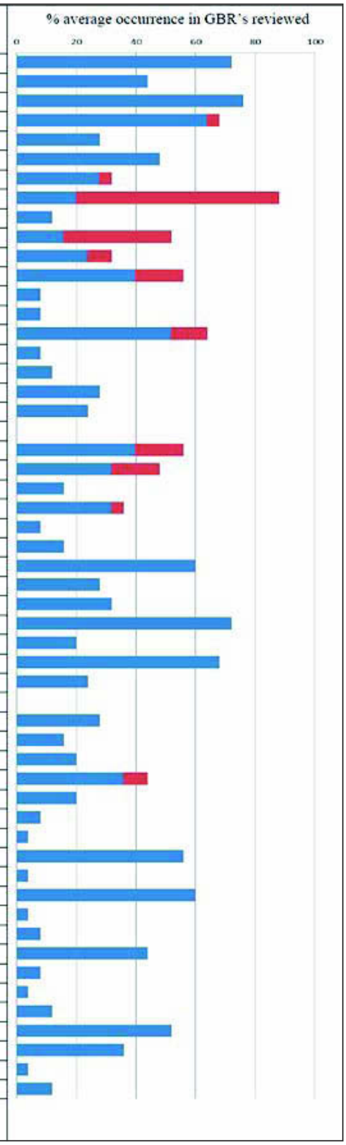

Figure 1. Distribution of baseline properties in geotechnical baseline reports reviewed

average values; they can also be defined using ranges and/or even graphically using graphs, geological sections, or contours. The problem with using multiple approaches is that several baselines are provided for the same property, often leading to confusion. Our research also highlighted the importance in a GBR to clearly identify what statements are baselines and what statements are not. This is especially important if sections including interpretation or discussion based on previous tunnel experience are provided. We found only 36% of the GBR's reviewed clearly identified and defined what were contract baselines. We also found that only 48% of reviewed GBR's provided any type of glossary to help define the terms used in the report and only 29% of

Table 2. Relevance of geotechnical properties for design and construction

The following table illustrates the relevance of certain rock properties in terms of the tunnel design and construction.

It also shows how an individual baseline can be used related to a range of different design and/or construction issues.

Construction Considerations						Design Considerations															
Grouting Requirements for Groundwater inflow	Overbreak	Mixed Face Ground	Initial Support Requirements	Excavation Equipment Selection	Advance Rates / Length	Numerical Modeling: Discontinuum (i.e. UDEC)	Numerical Modeling: Continuum (i.e. FLAC)	Analytical Design(s): Shotcrete Capacity Calculations	Analytical Design(s): Bischoff Rock Arch Calculation	Analytical Design(s): Ground Reaction Curves	Analytical Design(s): Unwedge Calculations	Empirical Design(s): Groundwater Inflow: Heuer	Empirical Design(s): Groundwater Inflow: Goodman	Empirical Design(s): Rock Loading (Q System)	Empirical Design(s): Unsupported Length (Q System)	Empirical Design(s): Standup Time (RMR System)	Empirical Design(s): Initial Support Requirements: RMi	Empirical Design(s): Initial Support Requirements: RMR	Empirical Design(s): Initial Support Requirements: Q	Property	
			O			O	O	O	O	O										Unit Weight	Intact Rock Properties
			O		O	O	O			O						O		O		Uniaxial Compressive Strength	
			O			O	O													Elastic Modulus (Young's)	
				O																CAI/ DRI/ etc	
		O	O				O			O		O	O	O	O				O	Rock Cover	Rock Mass Properties
			O			O	O		O											Angle of Friction	
			O				O													Cohesion	
			O				O			O										Elastic Modulus & Poisson's Ratio	
O												O	O							Rock Mass Permeability	
																				Seismic Velocity (P & S Waves)	
			O		O											O		O		Rock Mass Rating (RMR)	
			O											O	O				O	Rock Quality Designation (Q)	
			O							O										Geological Strength Index (GSI)	
			O		O										O	O		O	O	RQD	
O	O		O		O	O					O			O	O				O	Number of Joints	Joint Properties
O	O		O		O	O					O			O		O		O		Joint Orientation	
O	O		O		O	O										O		O		Joint Spacing	
			O		O	O									O	O		O	O	Joint Alteration	
O			O		O	O								O	O	O		O	O	Joint Roughness	
O	O		O			O								O						Joint Persistence	
			O		O	O					O									Joint Shear Strength	
O			O		O	O					O	O	O	O	O	O		O	O	Joint water flow	
			O			O														Joint Stiffness	
O						O	O					O	O							Groundwater Level	Groundwater & Stress
			O		O	O	O			O	O			O	O				O	Vertical Stress	
			O											O	O				O	SRF	
			O		O	O	O			O	O									Ko	

GBR's provided any information on what assumptions or classifications had been used in developing the baselines.

Problems Associated with the Practical Implementation of the Geotechnical Baselines During Construction

A commonly overlooked yet vitally important part of any GBR should be a discussion on how baselines should be measured and evaluated during construction. This is a common problem encountered when using GBR's in practice. Only 16% of the GBR's reviewed discussed how baselines should be measured during construction and only 20% discussed any allowable tolerances to the baselines.

For example, consider two baselines commonly provided for rock tunnels, the UCS of the rock and the amount of rock cover above the tunnel. In practice, how are UCS values to be measured in the event that a differing site condition exists related to rock strength? In order to justify the rock strength, are additional borehole and core samples required to be taken along the alignment and, if so, when, where, and how many tests are needed? If point load testing can be used on collected representative rock samples, what correlations should be used to determine equivalent UCS values? In order to verify or demonstrate changes in rock cover, are additional boreholes required? If probe hole data can be used, then when, where and how many probes should be used?

Problems Associated with the Concept or Intent of the Geotechnical Baseline Report

The principal purpose of a GBR as defined in the UTRC Geotechnical Baseline Reports for Underground Construction (1997), is "to set baselines for geotechnical conditions anticipated to be encountered during underground and subsurface construction, in order to provide clear indications in the contract for resolution of disputes concerning subsurface conditions."

GBR's are needed because there needs to be a fair way to manage the ground risks, especially for design build projects. Traditionally, on these projects the Owners essentially pass on the ground risks to the Contractor, who relies on a contingency to help mitigate these risks. However, due to the competitive nature of these contracts Contractors often find themselves with insufficient contingencies and are unable to complete these projects if they incur significant cost increases resulting from any changes in the ground conditions.

A commonly reported problem is that GBR's are often used as a risk transfer tool as opposed to a risk management tool. This typically manifests itself through the use of overly conservative and/or unrealistic baselines. Based on our research, it is clear to see that this is still true. As illustrated in Figure 1 many of the most common baselines provided, such as the UCS, Cerchar Abrasivity, RQD, hydraulic conductivity, and joint orientation, have often used, in our opinion, conservative and/or unrealistic ranges for the baselines. We believe this is the single biggest problem with using GBR's by far. There needs to be a greater effort in getting all parties to understand the importance of approaching GBR's in a fair and reasonable way. It takes all parties to understand their roles and responsibilities, otherwise the concept and approach is destined for failure. If GBR's are not prepared properly, it is our opinion that a bad GBR is worse than having no GBR at all.

Another conceptual issue raised is the lack of focus on design and construction issues when it comes to developing baselines. Our research shows that there is still a tendency to focus on describing and providing baselines only for individual rock properties. For example GBR's typically provide baselines for rock permeability and not groundwater inflow, or rock quality and not initial support requirements, or rock strength and not cutter wear. Figure 1 shows that this is still true; there is still a tendency to focus on rock properties that are obtained from the site investigation as opposed to specific construction or design issues.

RECOMMENDATIONS FOR FUTURE GBRs

In 1997 the UTRC report stated the following:

"Improvements are needed to overcome the following shortcomings in contractual geotechnical interpretative reports:

- *Baselines may not adequately describe the conditions to be expected.*
- *Baseline statements are often indefinite, too broad, ambiguous or qualitative, resulting in disputes over what was indicated in the contract.*
- *Baselines may present conditions that are more adverse than indicated by the data, or just plain arbitrary and unrealistic, without discussion or explanation for such apparent discrepancies.*
- *Discussion often repeats material on drawings or specifications*
- *Baseline statements are sometimes in conflict with the drawings or specifications*
- *The effects of means and methods of construction on ground behavior are not well described…"*

Based on our research these recommendations from 16 years ago are still true today and this demonstrates that as an industry there is still scope for improvement in how we prepare and use GBR's. The problems with GBR's highlighted in the first part of this paper are associated with a variety of different reasons, including the general approach and intended use (or misuse) of these reports, the way in which the data is presented and the way in which the baselines are measured and used in practice. We have provided recommendations on how these problems can be addressed; these are summarized in Table 3 and discussed in more detail below.

In terms of solving the conceptual problems there is no simple fix, this simply requires a change in mindset and approach and an acceptance that all parties involved need to play their part in making GBR's work. If the Owner, Contractor or Consultants do not work openly with a spirit of fairness, GBR's will continue to be of limited use. Consultants should work with Owners to develop meaningful baselines and Owners should understand that by providing realistic baselines they are in fact reducing the cost and eliminating the need for contingencies. If different ground conditions are encountered later this should not be seen as error but as an adjustment to what is the true cost of the project. Consultants should present baselines that reflect their own understanding of the expected ground conditions; they should avoid the approach of providing conservative baselines in the belief that they are helping the Owner to eliminate ground related claims.

The recently revised GBR guideline (Geotechnical Baseline Report for Construction, 2007, ASCE) specifically addresses this issue and the need to provide more realistic baselines. Although beyond the scope of this paper international experience particularly in Hong Kong, Singapore and Australia show how the use of two stage GBR's or the use of 'balanced baselines' could be useful tools in helping to ensure that ground risks are shared fairly between the Owners and Contractors and help to ensure that GBR's are used more effectively. For example the use of two stage GBR's are now standard practice on all current MTRC tunnel project in Hong Kong. Two stage GBR's allow the Contractor as part of his bid to provide his own interpretation of where he believes the baselines should be set and to identify were he thinks are the areas of

Table 3. Recommendations for preparing geotechnical baseline reports for rock tunnel projects

	Recommendations	Discussion
Conceptual Issues	Be clear about what is a baseline. We recommend allowing the Contract to rely on all information within the report, including any geological plans and sections. It is recommended to limit interpretation or make this consistent with the baseline statements.	This should eliminate confusion and help avoid contradiction and confliction. Often the extensive interpretation provided in the GBR conflicts with the actual baselines provided.
	Prepare baselines that relate directly to a construction or design issue, as opposed to simply providing a list of individual rock properties. Care should however be taken to incorporate the impact of the Contractors means and methods.	We are never going to be able to accurately describe all ground encountered but it should be possible to provide a minimum criterion for key aspects of the tunnel support and construction. For example provide baselines for groundwater inflow instead of rock permeability, or baseline a minimum initial support or rock quality (Q values) instead of simply providing individual joint or rock properties.
	Provide reasonable baselines based on the understanding of the interpretation and expected ground behavior. Avoid 'playing contractual games' in an attempt to minimize claims.	Avoid unnecessary or overly conservative baselines. Contractors should have the right to expect that the baselines presented are reasonable. Unrealistic or ultraconservative baselines that shift unreasonable risk to a contractor should be discouraged and are contrary to the overall intent of the GBR.
	Consider the use of alternative contractual approaches when using GBR's, including the use of a 2 stage GBR or by using Balanced Baselines (Doyle, 2006).	The use of a 2 step GBR allows the Contractor as part of his tender to show how he has interpreted the risk and where he believes fair baselines should be set. This allows the impact of the proposed means and methods to also be considered. In the spirit of partnering Balanced Baselines could be used as a way to share the risk and/or reward and to encourage a fair and open approach to determining baseline.
Baseline Definitions	See Table 4 for discussion and recommendations on how specific baselines can be presented.	
	Only provide one baseline for any rock property or design/construction issue.	This will help to reduce contradicting and duplicate baselines.
	Use Rock Mass Types or Classes to characterize the rock along the alignment, as opposed to lithology.	The use of Rock mass classes allow you to group different rock types with similar properties and behavior together. This allows variations in rock properties to be better defined and helps to eliminate variations.
	Do not baseline properties that can be heavily influenced by the quality of the contractor's means and methods.	For example it is not recommended to provide baselines for rock over-break.
	Be careful in providing baselines that require interpretation or assumptions.	It is not recommended to provide baseline of properties where there is the need for interpretation or where there is the need to make assumptions. For example if Q baseline values are specified then the SRF values to assume should be provided.

(table continues)

Table 3. Recommendations for preparing geotechnical baseline reports for rock tunnel projects (continued)

Recommendations		Discussion
	Show GDR test data where possible to show that you are being consistent and transparent.	It is recommended to be transparent and show the data that has been used to help develop the baselines. If the baseline provided is different from the testing data then these differences should be clearly explained.
Practical Implementation	Clearly specify how during construction the baseline rock properties will be measured. This testing should start from day one and baselines should be continually assessed by the Owner or CM during construction even when ground conditions are as expected.	It is important to clearly specific the type and frequency of any testing that is required to evaluate the various baselines. This will help to eliminate any uncertainty or misinterpretation in the event of a change condition.
	Clearly specify how during construction the baseline rock properties will be evaluated and clearly identify what costs or delays will the Contractor be compensated for.	The baselines should clarify the what in the question "conditions materially different to what?" Just because the ground is different this does not necessarily mean that this will impact the Contractor.
	Require the Contractor to develop contingency measures to address what would happened if there was a differing site condition at the start of the project.	This will help everyone to understand the cost and schedule impacts involved and help to reduce delays caused by differing site conditions .This should ultimately help save time and eliminate debate and uncertainty.

concern. In reviewing the resulting tender submissions the Owner can assess more clearly how the Contractors have used the baselines as part of their proposal and can chose to adopt the revised version.

Balanced baselines are an approach proposed by (Doyle 2006). "*If balanced baselines are used in addition to Contractors receiving extra payment if the conditions are more adverse than those in the baseline, it is suggested that the Owner should also receive a reduction in the contract price, for any less adverse site conditions that are encountered. In this situation both the Contractor and the Owner would have balanced risks in regards to subsurface site conditions."* In addition to these ideas the use of Geotechnical Contingency Funds can also help to ensure the partnering and effectiveness of GBR reports, a good example of the use of this approach is on the Port of Miami Tunnel Project.

In terms of addressing the structural problems associated with GBR's, such as providing more relevant baselines and clarifying how we present baselines to help eliminate ambiguity and contradiction we believe that this can be more easily addressed. Specific recommendations on how baselines can be presented and used during construction of rock tunnels are shown in Table 4.

We also recommend when developing future baselines for rock tunnel that we focus more on the behavior of the ground and specific design and construction issues, as opposed to simply providing a list of rock properties. Consulting Engineers and Geologists have understandably difficulty trying to develop specific numerical baselines for a wide range of geotechnical properties. This often results in the development of wide ranges for the various baselines. It is unlikely that we can ever expect to accurately describe miles of varying rock conditions, so it is recommended to focus on design and construction requirements which could be easier to quantify. This recommendation was in fact made in the UTRC (1997) report; a checklist was provided

Table 4. Design and construction considerations for GBR's (taken from UTRC Report 1997)

Design Considerations		Construction Considerations	
Description of ground classification schemes used.	25%	Required sequence of construction	10%
Criteria and methodologies used for the design of ground support and ground stabilization, including ground loadings.	5%	Anticipated ground behavior in response to construction operations.	45%
Criteria and basis for final design.	0%	Rational behind ground improvement.	0%
Environmental performance considerations such as limitations on settlement and lowering of groundwater levels.	0%	Identification of specific construction difficulties.	75%
The manner in which different support requirements have been developed for different ground types, and the protocols to be followed in the field for determination of ground support types for payment, reference to specifications for detailed descriptions of methods/sequences	0%	Rational behind baselines for groundwater inflow to be encountered during construction.	20%
The need and rational for ground performance instrumentation included in the drawings and specifications.	50%	Identification of sources of delay, faults, gas, obstructions etc.	75%

outlining what should be provided in a GBR, where in addition to providing baselines for ground characterization, it is also recommended addressing the following design and construction issues. Based on the results of our research we have also added an estimate in Table 5 and Table 6 of how frequently these considerations were provided in the GBR's we have reviewed.

Contractors are often frustrated because they feel that they are not always provided with the baselines they need, the results of our analysis clearly show this to be true (see Figure 1 and Table 4). Owners also feel taken advantage of when individual baselines are used to justify claims in a manner not intended or the baselines are not respected in the dispute resolution process. These concerns can be helped by Consultants providing more relevant baselines and baselines that address the design and construction considerations highlighted in Table 4, Table 5, and Table 6. We should only baseline rock properties that are necessary for a contractor to evaluate means and methods, estimate ground behavior for his initial and/or permanent support requirements and develop a construction schedule.

Finally it is recommended that we pay more attention to how baselines should be measured and assessed during construction. In the GBR's we reviewed, measurement and payment was discussed in less than 20% of the GBR's. It is important to clearly state how baselines are to measured using the expected means and methods, it is also important to explain how any changes in baselines will be evaluated. If a baseline is exceeded it is important to understand how this has impacted the work, and what cost or delays may occur.

CONCLUSION

In summary our research has highlighted a wide variety of problems can be encountered with using GBR's on rock tunnel projects and we have shown that many of these issues are continuing to occur on recent projects. To help mitigate these issues the following recommendations and conclusions have been made.

Table 5. Recommendations and checklist for how rock tunnel baselines can be presented (1 of 2)

Geotechnical Baselines		Priority	Recommendations	Testing and Sampling
Geological Interpretation	Rock Type (Lithology)	High	Provide clear descriptions of the rock units, using a recognized rock classification system.	Rock Mapping
	Top of Rock/ Rock Cover	High	Tabulate the minimum rock cover expected along the alignment, not recommended to provide contour plots, especially if they are computer generated.	Probing and Additional Site Investigation
	Rock Mass Types	High	Different rock types should be grouped into Rock Mass Classes if they have similar characteristics and behavior. Rock Mass Classification schemes such as RMR, Q and/or GSI should be used to define each Rock Class.	Rock Mapping & Additional Site Investigation
	Weathering Grade	High	A standard weathering classification scheme should be used (i.e., ISRM) and each grade should be clearly defined for example by using SPT(N), RQD or TCR testing results.	Rock Mapping & Additional Site Investigation
Intact Rock Properties	Rock Mineralogy (i.e., Hard Mineral Content)	High	Provide a range of values; this will typically include an assessment of the Quart Content.	Field Sampling (Testing)
	Unit Weight	Low	Provide a realistic range of unit weights.	Field Sampling (Testing)
	Unconfined Compressive Strength	High	Provide a clear range of realistic UCS strength values (It is not recommended to allow correlations with Point load testing).	Field Sampling (Testing)
	Tensile Strength (Brazilian Test)	Low	Provide a clear range of realistic tensile strength values.	Field Sampling (Testing)
	Young's Modulus (E)	Low	Provide a clear range of E values, be clear about how E vales were measured i.e., Secant modulus E_{50}	Field Sampling (Testing)
	Cerchar Abrasivity Index (CAI)	High	Provide a clear range of CAI values.	Field Sampling (Testing)
Rock Mass Properties & Classifications	Strength Properties (Angle of Friction & Cohesion)	High	Provide a clear range of strength properties for the rock mass; be clear to specify how these were derived and what assumptions have been made.	Field Sampling (Testing)
	Rock Mass Stiffness (Young's Modulus & Poisson's Ratio)	High	Provide a clear range of E values, be clear about how the E values were measured or derived.	Field Sampling (Testing)

(table continues)

Table 5. Recommendations and checklist for how rock tunnel baselines can be presented (1 of 2) (continued)

Geotechnical Baselines		Priority	Recommendations	Testing and Sampling
Rock Mass Properties & Classifications (continued)	Seismic Velocity	Low	Provide a clear range of seismic velocity values, be clear about how seismic velocity values were measured or derived.	Additional Site Investigation
	Rock Mass Permeability	High	Provide a clear range of rock mass permeability for each rock mass class, although it may be advisable to provide joint aperture and/or inflow estimates.	Additional Site Investigation
	GSI and Hoek & Brown Constants	High	Provide a clear range of GSI values for each rock mass class, state clearly any assumptions made.	Rock Mapping
	Q System	High	Provide a clear range of Q' values (without SRF and J_w component) for each rock mass class, state clearly any assumptions made.	Interpretation during mapping
	Rock Mass Rating (RMR)	High	Provide a clear range of RMR values for each rock mass class, state clearly any assumptions made.	Rock Mapping
	Rock Mass Index (RMi)	Low	Provide a clear range of RMi values for each rock mass class, state clearly any assumptions made.	Rock Mapping
	RQD	High	Provide a clear range of RQD values for each rock mass class, state clearly any assumptions made.	Rock Mapping & Additional Site Investigation
Rock Jointing	Number & Orientation of Joint Sets	High	Provide a clear number and range of orientations for the discontinuities in each rock mass class, state clearly any assumptions made. Recommended for joint orientation to only specify general dip directions i.e., NE and avoid using stereonets to display results as they are open to interpretation.	Rock Mapping & Additional Site Investigation
	Joint Shear Strength	High	Provide a clear range of c and phi values for each rock mass class, state clearly any assumptions made.	Rock Mapping & Additional Site Investigation
	Joint Spacing & Persistence	High	Provide a clear range of joint spacing and persistence values for each rock mass class, state clearly any assumptions made.	Rock Mapping & Additional Site Investigation

Table 6. Recommendations and checklist for how rock tunnel baselines can be presented (2 of 2)

Geotechnical Baselines		Priority	Recommendations	Testing and Sampling
Rock Jointing	Joint Conditions (including water)	High	Provide a clear range of joint condition descriptions; where possible provide joint aperture, alteration, roughness and waviness data for each rock class. This data must be consistent with the data used to develop any rock mass classification systems such as Q, RMR or GSI estimates.	Rock Mapping & Additional Site Investigation
Faulting	Number, Location & Orientation	High	Provide the number and location of know faults on the geological sections and clearly define their orientation.	Rock Mapping & Additional Site Investigation
	Fault Thickness & Properties	High	Provide a clear range for a fault thickness and strength properties, state clearly any assumptions made i.e., if true or apparent thickness has been used.	Rock Mapping & Additional Site Investigation
Stress	Insitu Stress (including Ko)	High	Provide a clear value for assumed insitu vertical stress and a range of values for horizontal stress (including K_o ranges), state clearly any assumptions made.	Additional Site Investigation & Field Sampling (Testing)
Groundwater	Groundwater Level	High	Provide a clear range of groundwater levels along the tunnel alignment, state clearly any assumptions made. This should include any allowance for flood levels and seasonal variations should be considered.	Additional Site Investigation
	Groundwater Inflows	High	Provide a clear range of groundwater inflow values for each rock mass class, state clearly any assumptions made. Values for immediate flush flows and steady state conditions should be provided.	Field Sampling (Testing)
Specific Ground Risks	Slake Durability	As Required	If appropriate provide a range of values to address the potential for slaking for each rock mass class.	Field Sampling (Testing)
	Swelling Potential	As Required	If appropriate provide a range of values to address the potential for swelling for each rock mass class.	Field Sampling (Testing)
	Solution Features and Voids	As Required	Recommend to identify the length of tunnel that may be impacted by the presence of solution features and voids. Avoid trying to identify specific void volumes as this tends to result in the development of conservative baselines.	Additional Site Investigation

(table continues)

Table 6. Recommendations and checklist for how rock tunnel baselines can be presented (2 of 2) (continued)

	Geotechnical Baselines	Priority	Recommendations	Testing and Sampling
Ground Behavior	Ground Failure Types	High	Provide a clear range of expected ground behaviors for each rock mass class, state clearly any assumptions made. Recommended to use an acceptable ground behavior classification scheme such as that proposed by Terzaghi 1977.	Rock Mapping
	Rock Loading	High	Provide a clear range of expected rock loading for each rock mass class, state clearly any assumptions made. Recommended to use an acceptable classification such as that proposed by Barton 1974 (Q System).	Rock Mapping
Construction Considerations	Overbreak & Volume of Excavated Material	Not Recommended to provide baselines for these properties are they are strongly related to the quality of the Contractors means and methods.		
	Excavation Techniques	High	Provide a clear discussion of anticipated excavation techniques, including any limitations or potential problems for specific means and methods.	Site Observations
	Construction Sequence	High	Provide a clear discussion of anticipated construction sequences, including expected maximum unsupported excavation lengths, standup time and the need to use of split heading/bench construction.	Site Observations
	Initial Support Requirements	High	Provide a clear discussion of anticipated initial support requirements, including the need for pre-support, face, crown and wall support and final lining support.	Field Mapping & Site Investigation
	Contaminated Ground or Groundwater	As Required	If appropriate provide discussion to address the potential for contaminated ground and/or groundwater.	Site Investigation & Field Sampling (Testing)
	Gas	High	Provide a clear statement on the classification (i.e., OSHA) of the tunnel in terms of gassy or non-gassy.	Field Testing
	Obstructions (natural or man-made)	High	If appropriate provide discussion to address the potential for encountering either natural (i.e., boulders) or mad-made (i.e., foundations) obstructions. The location of these should be clearly identified and a description of the obstruction provided. In the case of boulders avoid specifying the size and number of actual boulders as this tends to result in overly conservative baselines, it is recommended to identify a length of tunnel that may be impacted by this.	Site Observations and additional Site Investigation

- GBR's should in addition to characterizing the expected ground condition also provide baselines for specific design and construction issues.
- Realistic and relevant baselines should only be provided (see Table 5 and Table 6).
- Baselines should be clearly presented and repetition and confliction should be avoided.
- Assumptions or terminology used should be clearly provided; this should include providing a glossary of terms and any other references or testing used in developing the baselines.
- Discussion should be provided on how baselines are to be measured considering the expected means and methods to be used.
- Discussion should also be provided on how the baselines will be evaluated in the event of a change condition.

Finally GBR's are intended to be a risk sharing not a risk transfer tool, it is therefore important that all parties involved understand their role. GBR's are intended to be a true measure of ground behavior based on a reasonable interpretation of the available data not simply a conservative description of the site investigation data. Based on our research we found the best GBR's were those that provided a realistic interpretation of the expected ground conditions that included an assessment of ground behavior and construction implications.

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SMALL FOOTPRINT, BIG CHALLENGES— DESIGN AND CONSTRUCTION OF THE ALLEN PARK SANITARY DISTRICT 1 STORAGE TUNNEL

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ABSTRACT

Integrating new infrastructure into urban areas requires precision in design and construction. The new infrastructure has to virtually thread the needle of existing infrastructure and buried utilities while meeting the requirements of regulatory agencies, property owners, and other entities. These factors make it particularly challenging for large-diameter pipelines like the Allen Park Sanitary Sewer Overflow (SSO) Tunnel that had to cross an interstate highway, several railroads, gas and oil pipelines, a transmission water main, two creeks with limited cover, as well as a residential area and college campus. Rigid mining shafts, detailed specifications for tunneling materials, a fully breasted TBM, and various trenchless technologies were successfully utilized in conjunction with robust geotechnical instrumentation and monitoring, to mitigate settlement risks and overcome historical challenges with tunneling in the area.

INTRODUCTION

The Allen Park Sanitary Sewer Overflow (SSO) Tunnel and Relief Sewer project, located in the City of Allen Park, MI, is a long-term corrective action designed to bring Sanitary District One's sanitary system into compliance with their 2005 Consent Order and their service contract with the Detroit Water and Sewerage Department (DWSD). The $20 million project is intended to reduce Allen Park's wet weather discharges to DWSD, reduce bypass pumping to the Ecorse Creek, and limit the future risk of basement flooding, by providing storage during wet weather events, and eliminating hydraulic bottlenecks in the sanitary sewer system.

A year of final system flow monitoring was performed in 2006 and 2007 to collect data after short-term improvements were implemented. The data was used to calibrate a computer model that was then used to analyze the performance of the system through a statistical analysis of a continuous simulation which included 36 years of rainfall and snowmelt data. This helped verify the size, type and location of improvements needed to bring the sanitary system into compliance. The project was a decade in the making when it received a jump-start from the American Recovery and Reinvestment Act of 2009 stimulus funding. This resulted in 40% principal forgiveness on Allen Park's SRF loan and set the "shovel-ready" Project's 2-year construction period in motion.

The tunnel is sized to transport and store 1.34 MG of wet weather flow. Designed to be empty during dry weather and smaller wet weather events, it is estimated that the tunnel will convey wet weather sanitary flow an average of 10 times per year. Approximately three times per year, the excess sanitary flow entering the tunnel will exceed the downstream pump station capacity and the flow will be temporarily stored in the tunnel until it can be dewatered. The tunnel will need to be flushed with flow stored in upstream portions of the system one to four times a year to prevent the buildup of solids and gasses that can generate excessive odor and degrade the tunnel lining.

The tunnel will convey flow to a new 8.4-cfs submersible dry weather/wet weather lift station at the north tunnel connection on Outer Drive near Baker College's campus. Flow will be carried to a new 14-inch-diameter force main that will outlet to an existing trunk sewer outlet north of Outer Drive. This arrangement will replace the existing 18-inch gravity sewer that was unable to deliver the maximum outlet capacity to the Outer Drive Lift station without significant surcharge upstream. The new 8.4-cfs submersible pump station and 14-inch force main will lower the surcharge at the existing upstream junction manhole and eliminate this hydraulic "bottleneck" in the system.

Located within the Ecorse Creek Watershed, the 4,100 ft long tunnel alignment minimizes disturbance to existing wetlands and adjacent property owners. A portion of the alignment was designed to minimize the impact to the proposed greenway alternative from the future widening of the North Branch of Ecorse Creek. Implementing a storage tunnel and other trenchless methods for improving the wastewater infrastructure were utilized to provide an environmentally sound and sustainable solution to meet the City of Allen Park's needs, now and into the future.

PROJECT DESCRIPTION

Located in an urban area congested with existing utilities and structures, the Allen Park SSO Tunnel was designed and constructed to minimize impacts on surrounding areas while meeting the requirements of regulatory agencies, property owners, and other entities. To facilitate the proposed storage and conveyance improvements, while delivering a sustainable and environmentally sound project, tunneling and other trenchless methods were selected by the project team. The overall alignment crosses an interstate highway I-94, Canadian National and Norfolk Southern railroads, gas and oil pipelines owned by various utilities, a 54-inch DWSD transmission water main, a natural drain at two locations, as well as a residential area and Baker College's campus. The alignment even included a mining shaft located in the shadows of the famous "Uniroyal Giant Tire," which is a local landmark that consists of the repurposed Ferris wheel attraction from the 1964/65 World's Fair in New York.

A dynamic mix of five different trenchless construction and rehabilitation methods were used to complete 1.5 miles of sewer, minimizing impacts on existing structures and residential, commercial and environmental properties. A tunnel boring machine (TBM) was used to install 3,045 Linear ft (Lft.) of 8 ft diameter tunnel sewer in primary and secondary lining. A 24-in diameter, 700 Lft. section under the interstate highway was constructed utilizing microtunneling methods (MTBM). Pipe bursting was used to install a 400 ft section with only one service connection to increase the sewer diameter from 15 to 18 inches. A combination of directional drilling, slip-lining and open-cut techniques was used to install 1,300 Lft. of 14-in force main. The alignment also included open cut construction of 790 Lft. of 8 ft and 5 ft diameter sewer and 1,450 feet of 18-in diameter upstream relief sewer improvements.

The overall alignment of the soft ground tunneling portion along with an aerial view of the surrounding setting is shown in Figures 1 and 2. The following describes the various project elements.

Run 0 (North Tunnel Access Structure [NTAS] to Westerly Tail Tunnel)

Prior to commencing the production tunneling, a tail tunnel was constructed to accommodate the tunnel locomotive and associated muck cars. The tail tunnel was constructed by hand mining and placing liner plate (10 foot in diameter) through the secant pile shaft wall, and extended 38 ft from the west face of NTAS. This run was constructed below and perpendicular to a 54-in DWSD water transmission line.

Figure 1. Project alignment and overview

Run 1 (NTAS to ETAS)

This run progressed east out of NTAS to the East Tunnel Access Shaft (ETAS) approximately 1,375 ft in length with an invert approximately 30 ft below ground surface. This run consists of 12 ft diameter rib and lagging primary liner, with 8 ft diameter secondary liner, that traverses below a primary Wayne County Drain (Ecorse Creek), two tracks of the Canadian National Rail Road, three tracks of the Norfolk Southern Rail Road, and 10-in diameter British Petroleum Oil pipeline.

Run 2 (Pump Station Access Shaft [PSAS] to NTAS)

This run is 309 ft long, parallel to the 54" DWSD transmission main, approximately 35 ft easterly thereof. Consistent with Run 3 and 4, the tunnel consists of 144-in rib and lagging primary liner with 96-in reinforced concrete pipe as the secondary insertion.

Run 3 (NTAS to South Tunnel Access Shaft [STAS])

This run once again crosses beneath the Ecorse Creek and also beneath the retention pond of Baker College's storm system. The 850 ft tunnel run is also approximately 35 feet in depth, and is comprised of 144-in rib and lagging primary liner with 96-in reinforced concrete pipe as the secondary liner.

Run 4 (STAS to the East Junction Chamber [EJC])

This tunnel is constructed below the 54-in DWSD water main, 8-in diameter Sunoco Oil pipeline, 22-in diameter Wolverine high pressure gas main, 6-in diameter Sunoco Oil pipeline, a 16-in diameter Wolverine high pressure gas main, and an existing 12-in sanitary sewer in which there was 5 ft of clearance between each of the utilities. Cover over the tunnel crown ranged from 4.5 ft to 19 ft.

Run 5 (EJC to the West Junction Chamber [WJC])

This 800 ft run which crosses beneath 7 lanes of Interstate highway I-94 with a depth of 40-45 feet is constructed by micro-tunneling with a 54-in steel primary liner and 2 ft

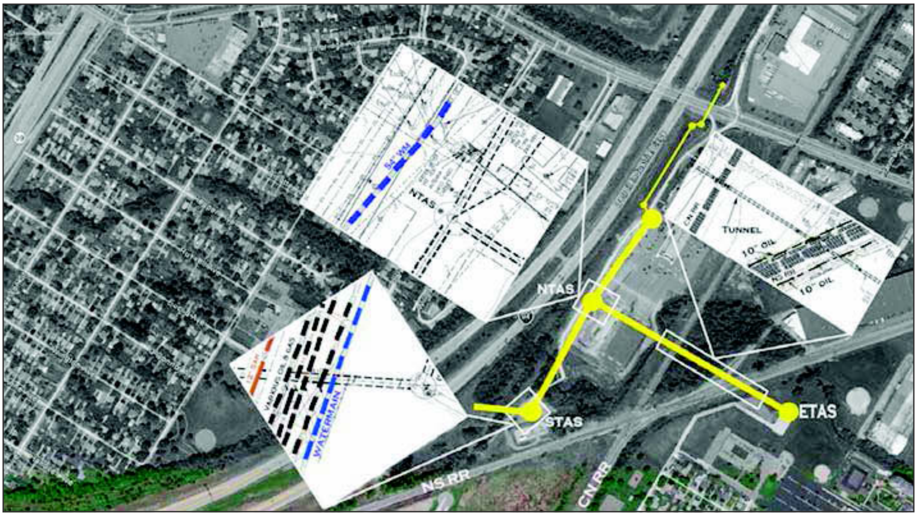

Figure 2. Critical utility crossings

diameter secondary liner. This was a late design change dictated by the governing highway agency. The mining shaft for this run was located approximately 30 ft from the "Uniroyal Giant Tire," one of the world's largest roadside attractions.

Runs 6, 7 (WJC to West Tunnel Access Structure [WTAS] to West Diversion Chamber [WDC])

These runs comprise 780 ft of 5 ft diameter concrete pipe approximately 30 ft deep and are constructed by cut and cover methods between the Ecorse Creek and Rogers Elementary School.

Run 8 (WDC to Sanitary MH 14-3)

Pipe bursting of 15-in vitrified clay with an existing CIPP liner upsizing to a 18-in PVC C900 fusible pipe. The length of sewer was 450 ft approximately 19 ft below ground surface.

Runs 9, 10 (MH 14-3 to Diversion Chamber 14-1 at intersection of Russell and Larme Streets)

Upsize of existing rear yard 12-in sanitary to 18-in pipe of 962 ft in length on south side of Shenandoah and Russell Streets with complete street replacement. This portion of the project required to be completed between July 5th and August 31st during which Rogers Elementary students are on summer vacation.

Run 11 (PSAS Going North Toward Existing Sanitary MH 228)

Run consists of directional drilling of a portion of the new Pump Station's force main (632 ft) with subsequent placement of 14-in HDPE pipe.

Run 12 (Sanitary MH 228 to the Existing Pump Station)

Slip lining of existing 21-in sanitary sewer with 183 ft of 14-in HDPE beneath the major thoroughfare of Outer Drive.

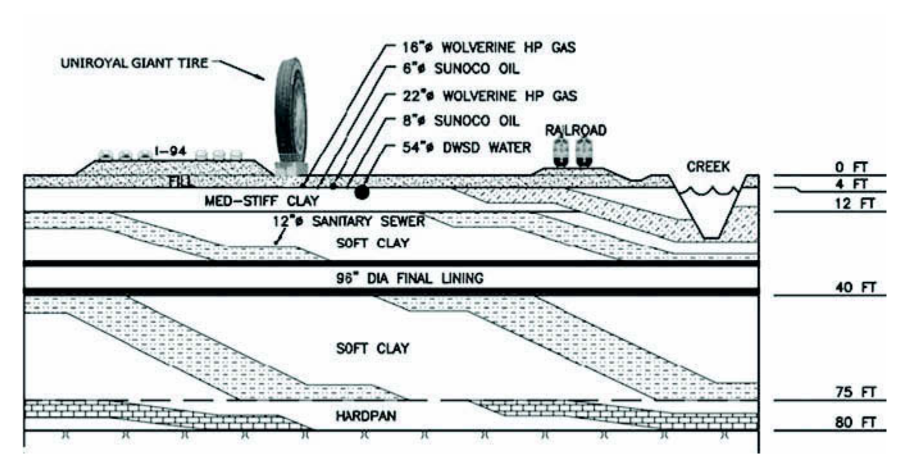

Figure 3. Generalized soil profile

GROUND CONDITIONS

The ground conditions in southeastern Michigan consist of glacial drift, characterized as clayey tills, outwash sands and gravels, and glaciolacustrine silts and clays that are underlain by glacial till. The clay soils frequently contain intermittent sand and gravel layers with cobbles and boulders produced from glacial river deposits as glacial lake levels fluctuated. Overlying the bedrock is often a layer of highly over consolidated glacial till locally referred to as "hardpan." The underlying bedrock consists of layers of sedimentary rocks comprised of shales, limestones, and dolomites that slope or dip inward from the rim of the Michigan Basin toward the center of the basin.

Geotechnical Investigation

To support the design of the tunnel, shafts, and other underground structure design, a detailed geotechnical investigation was conducted along the project alignment. The investigation included 19 borings along the alignment with depths ranging from 7.5 to 90 ft. The borings included 14 shallow borings within the overburden, and 5 borings extending into the underlying bedrock. The borings were advanced in the overburden with hollow-stem augers and rotary wash equipment, with sampling by split-spoon or thin walled tube. Field testing included SPT testing by split spoon, and in-situ vane shear testing. Laboratory testing program included determination of dry density and moisture content, unconfined compressive strength, and Atterberg limits. Investigation of the bedrock included continuous sampling with NQ conventional core barrel tooling, recording percent recovery, RQD, and fractures per foot. The data and investigation results were summarized in geotechnical data (GDR) and interpretive (GIR) reports that were appended to the contract documents.

Generalized Soil Profile

The subsurface stratigraphy along the proposed tunnel alignment is relatively uniform (Figure 3), consisting of a thin layer of variable surficial fill extending from the ground surface down 3 to 5.5 ft. Below the fill layer are natural soil deposits consisting of a thin desiccated layer of medium to stiff silty clay that extends to a depth of about 12.5 ft below ground surface, underlain by a thick layer of soft to medium silty clay that extends to a depth below ground surface ranging from 67 to 77 ft. The deep portion of the soft to medium clay strata contained occasional thin granular stratum consisting of silt and silty sand. The unconfined compressive strength of the soft to medium clay, which comprises most of the tunnel alignment, varies from approximately 1,200 psf near the top of the deposit, to less than 600 psf for the lower portion of the strata. The soft to medium clay layer is generally underlain by a thin layer of hard to very hard silty clay (“hardpan”) that extends to the limestone bedrock encountered at a depth of 83 to 90 ft below ground surface.

Groundwater and Gas Conditions

The long term static groundwater is typically located 15 to 20 ft below ground surface, and is dependent upon seasonal variations of perched groundwater from the adjacent floodplain and granular surface deposits that are hydraulically influenced by adjacent water bodies. Additionally, groundwater is also present in silt and sand seams found within the deeper glaciolacustrine clay deposits. Deep granular layers at or near the soil-rock interface may be hydraulically connected to the underlying bedrock aquifers and have greater recharge capacity. The confinement of these layers from the overlying clay often results in artesian groundwater conditions in the area.

Low levels of hydrogen sulfide gas are typically present within the substrata throughout the project alignment. Hydrogen sulfide was detected during exploration at concentrations ranging from 2 to 3.5%.

DESIGN CONSIDERATIONS

During the design phase of the project, several performance criteria were established as critical for achieving project success. These included minimizing risk of 3rd party damages resulting from excessive ground deformation, minimizing cost overruns, and meeting the project schedule for construction. In order to achieve these goals, provisions were incorporated into the Contract Documents that were intended to minimize the Owner’s risk of damage to existing utilities and adjacent infrastructure, facilitate a competitive bidding process, and to minimize financial risk to the Owner while still enabling the Contractor enough flexibility to address his perceived risk accordingly when tendering a bid.

Shafts

In order to accommodate the variety of subsurface improvements, the project required construction of 7 shafts, ranging in size from 12 ft diameter for the smaller sanitary sewer improvements, to 40 ft diameter for the pump station mining shaft. The shafts ranged in depth from 18 ft to 60 ft, with the deepest shaft required for the permanent structure of the dewatering pump station.

Rigid mining shafts were specified for three critical shaft locations to minimize potential for ground movement during tunneling operations. These included PSAS, NTAS, and STAS, each of which was to be located approximately 30 ft from the utility corridor which included the 54-in DWSD transmission main as well as the series of high pressure gas and oil pipelines. Many of these aging utilities had been in service for 60 years. The contract included provisions for the use of secant piles, diaphragm slurry wall, or sinking caisson methods of shaft construction at these locations. The contract included detailed performance criteria including minimum structural requirements and ground deformation limitations; however, it required that the successful contractor ultimately select and take design responsibility for the temporary support of excavation.

Primary and Secondary Tunnel Lining

A two-pass tunnel liner design was specified which required steel ribs and timber lagging for the primary liner and 96-in reinforced concrete pipe for the secondary liner. The contract requirements for the primary tunnel lining included minimum rib spacing, as well as structural and dimensional properties of lagging to mitigate potential difficulties encountered in previous tunneling projects in the area's soft ground.

Minimum allowable bending strength for design of timber lagging was specified as to not exceed 1,200 psi where kiln dried product would be used, and not to exceed 750 psi where non-kiln dried product would be used in the execution of the work. Actual board thicknesses were required to be used in the design of the lagging.

Secondary lining consisted of 8 ft long sections of ASTM C76, Class IV, Wall B, Reinforced Concrete Pipe, fitted with cast in place fittings in the pipe wall as necessary for the proper application of grout between primary liner and secondary liner. Although groundwater penetration was not a significant concern given the geotechnical characteristics, ASTM C443 gasketed joints with grouted inside annulus were specified to ensure a water tight sanitary storage vessel as well as to ensure a smooth finished surface to allow efficient transport and effective tunnel flushing. Maximum allowable groundwater infiltration was specified to not exceed 20 gallons per inch of diameter, per 500 feet of pipe, per 24 hours for the individual runs.

Settlement Tolerance

Strict requirements for geotechnical instrumentation and monitoring were specified to further manage owner risk by monitoring soil movement and utility settlement/heave from shaft, tunnel and cut-and-cover construction activities. Inclinometers, tell tales, monitoring point arrays, and deformation monitoring points were utilized. A specific action plan was developed to respond to ground movements encountered in the field, to mitigate risk of settlement and/or damage to the critical utilities and infrastructure within the tunnel zone of influence.

The specifications identified a maximum allowable surface settlement of 1-in and maximum allowable heave of 0.5-in. Where the tunnel crosses the MDOT right-of-way for Interstate 94, the maximum allowable surface settlement was further restricted to 0.5-in. The contract required that the contractor restore the site to pre-existing grades and profile, and repair any damage should these threshold values be exceeded.

Boulders

During the design phase, it was documented that historical data indicated that boulders were likely to be contained with the silty clay throughout the tunnel alignment. Accordingly, the contract documents incorporated measures to advise the contractor that cobbles and boulders may be encountered at the tunnel face. The tunneling specifications indicated that boulders less than 24 inches in the average of 3 dimensions as measured protruding into the bore would be incidental to the project. The specifications also required that the mining machine include provisions for removal of boulders at the tunnel face. In addition, a contingency bid item was included in the construction contract to cover unforeseen physical conditions which might be encountered during construction. These measures ultimately minimized changed condition claims from the contractor during tunneling operations.

TBM Features

Face stability analyses during design indicated that for a tunnel mined in the soft to medium clay strata utilizing open face mining would result in overload factors in the range of 6 to 9. This indicated a marginally stable tunnel face that may be subject to excessive squeezing. Based on other underground projects in the area, however, it was believed that the clay soils in the area would be capable of short term self-support even with overload factors up to 10. As such, it was determined that a conventional mining shield with positive face control would be suitable for installation of the primary lining.

The specifications required that the selected tunnel boring machine (TBM) was to be compatible with anticipated ground and groundwater conditions, be capable of providing full-face support, and be equipped with face closure doors. The face was to be accessible through the cutter head for the removal of obstructions.

CONSTRUCTION AND PERFORMANCE

The construction contract was awarded in October 14, 2009, and mobilization commenced in early-November 2009. The first mining shaft (NTAS) construction commenced May 5, 2010 and was completed by the end of June 2010. The TBM was assembled and mining of Run 1 began on August 6, 2010.

3rd Party Coordination and Community Relations

During the preliminary phases of construction, extensive coordination with the various utilities, railroads, transportation agencies, and other impacted property owner's was undertaken to ensure that the work progressed according to the project schedule.

Community Relations

In an effort to minimize public inconvenience due to construction activities, and to ensure appropriate precautions were taken to protect public lives and property, several public outreach meetings were conducted to present the schedule and scope of activities near residential areas. As work activities were ready to commence in a given area, a door-to-door campaign was instituted to remind residences of pending work which would include street closures, equipment deliveries, and heavy truck traffic at muck haul routes.

School Influences

The construction schedule for the project was controlled indirectly by the needs of 3 schools that were interlaced within the project environs. Rogers Elementary School was situated at the west end of the project and was to be impacted by the installation of new 18" sanitary sewer and associated excavation and paving work. Additionally, the haul route for Runs 7 through 10 traversed the area adjacent to the school and through the surrounding residential area. In order to avoid the conflict with school traffic consisting of 9 buses and 250 cars per day between the hours of 8 AM to 9 AM and 3 PM to 3:30 PM, the contract specified that the work was to be performed between July 1st and August 31st. The contractor successfully completed this work in the summer of 2010.

A mining and access structure (ETAS) was located on the project's east end and served as the retrieval shaft for tunnel Run 1. This structure was situated on Inner City Baptist Schools property, particularly on the east end of the school's junior varsity soccer field. Decommissioning of the mining shaft, construction of the permanent 30 ft diameter, below-grade flushing chamber, and restoration of the playing field was required to be complete for the fall 2011 season. The contractor successfully completed all activities to meet the schedule milestone.

The most crucial coordination necessary for project progress was with Baker College. The site included the main mining shaft (NTAS), the pump station shaft (PSAS), and the south tunnel shaft (STAS). Access to the site, as well as the muck hauling route, was via the campus' entrance drive. The work site temporarily occupied

Figure 4. Each shaft location presents unique challenges

approximately 6.5% of the campus parking area, which typically accommodates 1,000 students daily attending both day and evening classes on campus. Daily coordination and routine meetings with Baker College representatives took place to ensure that the safety and daily activities of the students and administrators were not adversely affected.

Transportation Agencies

During the FHWA and MDOT review of the final design documents, which ultimately extended into the bidding period, a decision was rendered that required approximately 800 ft of the 8 ft diameter storage tunnel to be downsized to 2 ft finished diameter, such that "storage" would not occur within the right-of-way. The excavation was further limited to 4.5 ft, and a jack and bore operation was proposed and accepted by the MDOT. The design was revised by addendum, adding two additional shafts and permanent structures to accommodate the transition in pipeline size. Ultimately, the contractor proposed a 4.5 ft microtunnel (MTBM) approach and successfully worked with the MDOT to revise the permit for the crossing (Figure 4).

Railroad Crossing

Based on the permit for crossing the NSRR right-of-way, liner plate was required to be used as the primary liner. The contractor proposed to utilize steel channel lagging and steel ribs, in lieu of liner plate. It was also believed that the expanded ribs and lagging would minimize ground settlement while tunneling under the tracks. Typically, many railroads require the use of fixed steel liner plates that bolt together when tunneling under track. This method often results in greater settlement as the plates cannot be expanded to meet the ground beneath the TBM and the operation proceeds more slowly. In the preliminary phase of construction, it was demonstrated to the railroad decision-makers how steel rib and lagging materials would provide a greater degree of protection against above ground settlement during construction. Ultimately, the proposed rib and steel lagging alternative was accepted for use and was successfully installed, resulting in maximum track settlement of less than 0.1 in.

Shaft Selection and Construction

For the rigid shaft locations at the pump station (PSAS), NTAS, and STAS, the contractor selected to utilize 33-ft diameter shafts comprised of secant piles with reinforced-concrete ring wales. The contract required 3 ft minimum diameter for secant piles;

Figure 5. Secant pile shaft and TBM prior to insertion

however, the contractor successfully proposed the use of 2 ft diameter piles, with the secondary piles reinforced with HP12x53, and concrete ring wales.

The secant pile shafts were installed using the continuous flight auger (CFA) method. Initially, grout was maintained at constant pressure of approximately 25 psi and injected at the base of the auger stem during withdrawal. It was observed, however, that the excavated clay soils exhibited better strength properties than anticipated, and as such, the contractor elected to attempt excavation of the piles without grouting the hole during the drilling process. It was determined through observation and measurement that the excavated piles indeed held up without appreciable deformation. Ultimately the remaining secant piles were constructed in this manner, with the open holes ultimately being filled with grout or structural concrete by pump and tremie tube.

As the excavation of the rigid shaft for the pump station progressed, it was observed that many of the 80 ft long piles were not within vertical tolerance within the lowest ⅓ of the excavation. The use of smaller diameter piles compounded the effect of this problem. This required modification to the ring beam design and resulted in encroachment into the clear working diameter of the shaft. Upon completion of the excavation, three-dimensional laser scanning was employed to document the as-built shaft conditions and to determine what modifications to the permanent structure would be necessary (Figure 5).

Flexible shafts consisting of steel sheet piling and reinforced concrete ring beams were utilized for the ETAS mining shaft and the MTBM mining shafts. The contract specifications had less stringent requirements for these locations due to their proximity to adjacent utilities or infrastructure.

TBM Selection and Performance

The contractor employed a 12 ft diameter, Lovat model ME 142/150 PJ/RL tunnel boring machine, which is a bi-directional, rotary head, soft ground machine. The machine incorporated a fully enclosed forward shield and a soft ground cutterhead equipped with spade/ripper type teeth and flood control doors at the face of the machine. Muck removal was accomplished by a 300 degree much ring, mounted in the center of the forward shell, which transferred muck through pressure relief gates to a conveyor in open mode or to a screw conveyor in closed mode, and ultimately transported to the rear of the machine by conveyor for final removal by muck carts and locomotive. Sawdust, obtained from a local producer, was used to condition the soft clay at the tunnel face (see Table 1).

Table 1. Summary of TBM performance

Run	From–To	Linear Feet Mined	Actual yd^3 Mined	Total Days of Operation	Total Days Mined	Linear Feet per Day	yd^3 Mined/ Day	Average Settlement per Run
#1	NTAS–ETAS	1357	6092	41	36.5	37.1	166.9	0.84"
#2	PSAS–NTAS	309	1322.5	23	17	18	77.7	1.48"
#3	NTAS–STAS	770.3	3148	21	19	40.5	165.6	0.06"
#4	STAS–EJC	396.5	1677	15	15	26.4	11.8	0.21"

Production Rates

The typical mining operation included 2 shifts of 9 hours per day. When mining within the zone of influence for the railroad and critical utility crossing, the work proceeded 24 hours per day, utilizing 2 working shifts of 12 hours. Maintenance was generally performed on Saturdays when no mining was taking place. The average downtime over the duration of the project for maintenance or repairs was approximately 45 minutes per day.

As would be expected, the production rates varied considerably between the 4 major runs of the 12 ft bore, with the higher production rates occurring during the longer runs of tunnel. The average production rate for the TBM-mined tunnel was 30.5 ft per day. The best production day was 72 ft, while the worst day was 3 ft, with only a single set installed due to mechanical failure and subsequent repair of the rib expander.

Boulders

During the mining operation, the excavated material was primarily soft clay that was conditioned with sawdust, to allow efficient removal from the face (Figure 6). Cobbles were routinely encountered and easily removed by cutterhead and conveyor. Throughout the project, 13 boulders ranging in size from 12 to 32 inches in average dimension were encountered during mining. Since the contract required that boulders less than 24 inches in the average of 3 dimensions were to be considered incidental to the project, only 1 boulder, encountered in Run 3, resulted in additional cost to the project.

Settlement Analysis

Due to the location of the tunnel with respect to critical utilities and infrastructure, a detailed instrumentation and monitoring plan was developed during the design phase and identified in the contract documents. Instruments included inclinometers, tell tales, monitoring point arrays, and deformation monitoring points installed at critical utility locations, shaft locations, and rail/highway crossing. The monitoring program was designed, installed, and maintained by the owner, with daily communications transmitted to the contractor to allow appropriate action to be taken should threshold levels of deformation be encountered.

The frequency of monitoring varied, but typically consisted of weekly measurements of ground deformation in the vicinity of shafts, and daily measurement of monitoring points and arrays within the vicinity of the tunnel face. The tunneling induced settlement measurements ranged from 0.06 in to 6.24 in, the largest occurring due to significant ground loss that occurred at the tunnel eye when the TBM was launched from the shaft for Run 3. The average measured surface settlement for the project was 0.97 in, which equates to approximately 2% of the excavated volume.

Measurements indicated that the largest surface settlement occurred during the maintenance shifts, when the TBM was not advancing. Twenty-four-hour tunneling operations were thus used to minimize settlement in critical locations, particularly the

Figure 6. Mining operation with effective boulder removal

crossing of CN and NS Railroads. The maximum settlement of the 7 sets of tracks that were crossed for this project was found to be only 0.09 in.

Other Trenchless Methods

As indicated above, the project consisted of a variety of trenchless methods to not only incorporate existing utilities into the improved sanitary system but also to accomplish the existing system tie-in without interrupting the 24 hour per day, 7 day a week capability of the pump stations. The following is a commentary on these trenchless methods including location, success thereof, and issues encountered, as well as significance to the project:

Run 5 (800 ft Long, 54" Diameter, MTBM)

The contractor proposed an alternate to the proposed 48 in boring and jacking method that is shown in the contracts documents for the crossing of I-94. This alternate eliminated a bore pit and a manhole in the median and consisted of increasing the casing diameter to a 54", .563 w/steel casing placed using a purpose built Akkerman 54" micro-tunnel machine. The MTBM used a rotating wheel to loosen and remove the spoil. This change was advantageous in that it was performed with a manned machine and operator at the face, monitoring the soil conditions constantly, as well as being articulated and steerable and guided by a laser guidance system. This change was ultimately accepted by MDOT and the owner assumed an appropriate credit to the contract. This run was successfully completed within the specified allowable settlement tolerances of less than 0.5 in.

Run 8 (450 ft Long, 18" Diameter, Pipe Bursting)

This portion of the project proved to be extremely difficult and quite problematic to the contractor. With the depth and upsizing required the burst could be classified as “Challenging” according to Tables 1 and 2 Project Classification as depicted on pages 20 and 21 covered in NASTT publication “Pipe Bursting Good Practices.” The contractor incurred excessive overburden pressures on the C-905 PVC pipe due to delays in shaft preparation. This resulted in exceeding the maximum pulling pressures of the pipe (greater than 64.2 tons). This necessitated some unexpected additional excavation and restoration in the work area. Nonetheless, the work was completed, upsized and the sewer flow was reestablished through the pipe until the new pump station was ready.

Run 11 (632 ft Long, 14" Diameter, Directional Drill)

This portion of the new force main located in the green belt just west of Enterprise Drive was designated to be constructed by slip lining 14" PVC C-905 through the existing 21" sanitary sewer. Contractor proposed to change the force main to a direction drill using 14" HDPE with tracer wire to be placed approximately 8 feet above the existing line. By using this approach the temporary bypass line and pumping of the existing sanitary line could be eliminated as the extent of the tie-in on the new main was significantly reduced (Run 12). This change resulted in an appropriate credit to the owner and eliminated the MDOT mandated 30-day maximum period for the temporary bypass line that was to be installed along the east guard rail of the Outer Drive bridge along I-94. This work was accomplished successfully within several days of time.

Run 12 (183 ft Long, 21" Diameter, Slip Lining)

The slip lining and ultimate tie-in of the new system was successfully completed during a 3 day weekend. The existing flow in the sanitary sewer was stored in the wet well of the new pump station and its contents pumped into the new discharge manhole upon completion of the tie-in of the new force main.

CONCLUSIONS

In addition to the typical engineering and construction challenges associated with underground construction, the Allen Park Storage Tunnel Project, nearly a decade in the making, required thorough coordination with multiple federal, state, and local agencies, two rail roads, three schools, and several bustling residential neighborhoods in order to achieve success. The proactive and coordinated approach to informing and interfacing with the community and the other 3rd party stakeholders, was well-received and resulted in well-informed project participants that have worked together to see this project through completion without significant changes, delays, or disruptions.

Detailed, performance based specifications provided for successful risk management through the design and contracting phase, yet allowed the contractor adequate flexibility in determining the most appropriate and cost-effective approach to perform the various types of shaft, tunnel, and other trenchless installations. A collaborative effort between the Contractor and Owner/Engineer during the pre-construction activities ensured that the project performance expectations with respect to shaft and tunnel construction, and settlement limitations were understood and achieved. Ground deformation was successfully minimized in the vicinity of the critical utility, railroad, and highway crossings, resulting in no adverse impact to any of the project stakeholders.

The project alignment, dictated by the constraints of the existing infrastructure, both at the surface and below, required detailed engineering solutions and precise construction in order to successfully utilize the underground space for the much needed sanitary storage and conveyance improvements. In addition to successfully achieving the technical goals of the project, substantial completion was achieved in January 2013, ultimately meeting the project's schedule and budget.

ACKNOWLEDGMENTS

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PART 7

Geotechnical Instrumentation—Settlement Control

Chairs

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CASE-HISTORY BASED SETTLEMENT TROUGH CHARACTERISTICS FOR PRESSURIZED TBM TUNNELING IN GLACIAL SOILS

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Richard Sage ▪ Sound Transit

Robert A. Robinson ▪ Shannon & Wilson

ABSTRACT

The potential impact of tunneling-induced ground displacements on structures depends on the geologic setting and the nature of the ground surrounding and above the tunnel. This paper summarizes case history data from several tunnels constructed using pressurized tunnel boring machines (TBMs) in glacial soils in Seattle, Washington. A finite element model developed for the Alaskan Way Viaduct Tunnel and calibrated by using the case history data is used to extend our understanding of the condition where looser or softer soil is present above the glacial soils. The results of the finite element model and case history data are used to refine the settlement trough width characteristics for glacial soils for use in estimating settlements using the inverted normal distribution.

INTRODUCTION

The purpose of this paper is to provide a combination of case-history and numerical-model based inputs to the empirical method for estimating transverse surface settlement resulting from pressurized face TBM tunneling in glacial soils commonly encountered in the Seattle area. The inputs should be applicable to the initial assessment of tunneling-induced risks along proposed tunnel alignments during the conceptual or preliminary phase of a project as part of a larger risk assessment program. The inputs are not applicable to special cases such as break-in/break-out at tunnel portals or shafts, the so-called "learning curve," areas of abrupt changes in subsurface conditions, improper or inadequate operation of equipment, or other situations or conditions where excessive ground volume loss could occur.

The empirical method used is based on measured ground surface settlement associated with tunneling case histories and provides an experience-refined method for developing inputs to utility, building and structure impact assessments along potential tunnel alignments. The method's relatively simple formulation allows parameters to be varied over a range for use in qualitative procedures to assess uncertainty and risk. The empirical method is two-dimensional and primarily accounts for ground movements associated with the inevitable elastic response of the soils due to excavation, overexcavation or loosening of soil, and insufficient support of soils.

Experience suggests that tunnels constructed through glacial overconsolidated soils and using TBMs tend to produce wider and shallower settlement troughs, and depending on the subsurface conditions and depth to tunnel crown, could result in less settlement than would be calculated using typical inputs to empirical methods. This paper presents a review of case history data and analyses using a calibrated finite element model to refine the settlement trough width characteristics for glacial soils for use in estimating settlements using the inverted normal distribution (Gaussian function). The case history data reviewed was derived from the following projects:

- Mercer Street Combined Sewer Overflow (CSO) Tunnel (King County)
- Henderson Street CSO Tunnel (King County)
- Beacon Hill Light Rail Tunnels (Sound Transit)
- University Link Tunnels (Sound Transit)

GEOLOGIC SETTING

Tunnel projects in the Seattle area are most commonly constructed through soils deposited during or between one or more of the six or more glacial advances and intervening interglacial periods that have occurred within the Puget Sound area within the last 2 million years. The ice for these glaciations originated in the Coast Range and Rocky Mountains of British Columbia, Canada, and generally advanced (flowed) southward into the Puget Lowland where Seattle is now located. Each glaciation deposited new sediment and partially eroded previous sediments. During the intervening periods when ice was not present, normal erosion and depositional processes occurred further complicating the geologic setting.

During the last glaciation in the Puget Sound area, glaciolacustrine clay and silt, outwash sand, and lodgment till were deposited by the glacier. These soils and the underlying previous sediments were consolidated by the weight of about 1 km of ice. As the last ice to reach the Puget Lowland retreated to the north, deposits of gravel, sand, silt, and clay were laid down by meltwater streams issuing from the glacial ice front. These deposits are termed “glacial recessional soils” and are not glacially consolidated. Since the last glacier retreated, alluvial, lacustrine, colluvial, landslide, beach, and estuarine soils were deposited. Where development has occurred, these soils have been covered with fill, structures, and roadways, or potentially removed.

The review of case history data and numerical modeling is restricted to tunnels constructed in glacially overconsolidated sand and clay. Lodgment till, typically a very dense, gravelly, silty sand to silty, gravelly sand with a consistency and strength similar to very soft rock, has contributed to reducing the surface expression of tunneling-induced settlement in the Seattle area. However, lodgment till was not encountered in sufficiently thick layers along the tunnel alignments reviewed to be included in the analysis.

EMPIRICAL METHOD

Tunneling-induced ground surface settlements are a function of excess volume of soil excavated beyond the theoretical tunnel volume (ground volume loss) and the shape of the surface settlement trough. Ground volume loss is expressed as the percent fraction of excessive ground excavated around the tunnel over and above the theoretical or ideal excavated volume of the tunnel. It is generally accepted that the shape of the surface settlement trough can be modeled by a Gaussian function (Mair and Taylor, 1997). The following sections present a discussion of ground volume loss, maximum settlement, the shape of settlement trough, and the analysis.

Ground Volume Loss

For this paper, ground volume loss at the tunnel is assumed to be equal to the volume of the surface settlement trough (i.e., bulking and dilation are ignored). The ground volume loss parameter accounts for a variety of factors including, but not limited to, the geologic and hydrogeologic conditions, tunneling method, and operation of tunneling equipment. Ground volume loss is comprised of excess soil that moves toward the

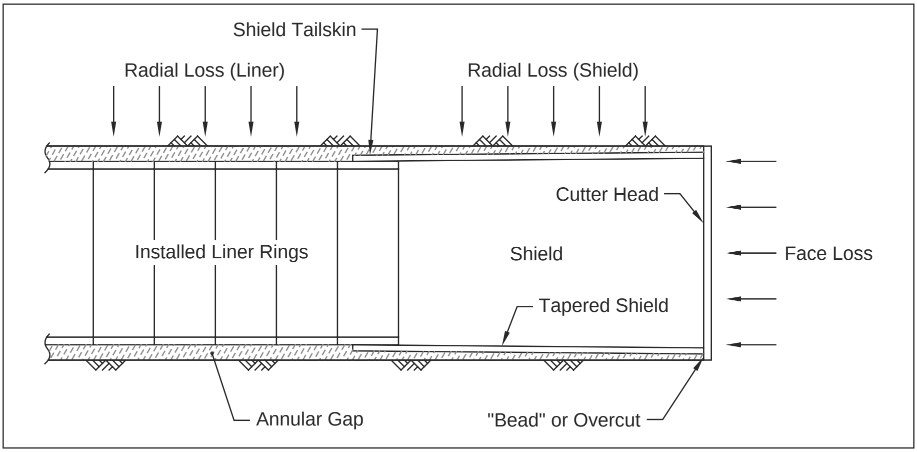

Figure 1. Typical sources of ground volume loss

TBM face, soil that moves in around the TBM perimeter and fills the annular gap due to the TBM overcut, and soil that fills the annular gap after passage of the tailskin and outside of the initial liner before sufficient grout can be injected to fill the gap. Sources of ground volume loss are shown schematically in Figure 1. Some minimal ground volume loss is inevitable due to elastic relaxation of the soils in the face and perimeter of the tunnel and the normal gage overcut at the face of the TBM needed to advance and steer the TBM. Ground volume losses in excess of these minimal values may be associated with:

- Learning curve associated with startup of tunneling activities or where conditions change significantly along the alignment.
- Inadequate face pressure, face control, or other tunneling mean and methods, which are not appropriate for the variable soil and groundwater conditions.
- Ovalling of the excavated tunnel perimeter by “crabbing” or “plowing” of the TBM through the ground including steering through curves.
- Removal of excess amounts of soils in response to the preceding item.
- Insufficient injection pressure and/or insufficient quantity of grout used to fill the annulus around the concrete segmental liner.
- Delaying grout injection into the annulus around the segmental liner for one or more rings behind the TBM tailskin.
- Harder or strong soils and weaker soils both being present at the face.

As discussed, even utilizing good quality means and methods, with experienced and capable personnel, ground volume losses along the tunnel alignment are likely to vary. It has been stated that “the correct choice of machine, operated without the correct management and operating controls is as bad as choosing the wrong type of machine for the project” (British Tunnelling Society and Institution of Civil Engineers, 2005).

An assessment of the probable average tunnel-induced ground volume loss is typically selected based on a review of case histories, local experience, engineering judgment, and consideration of the risk to adjacent facilities. Ground volume losses measured at the surface were reviewed for recent Seattle area closed-face TBM projects completed in glacial soils are provided in Table 1. The ranges of values represent typical running tunnel construction and not areas where problems occurred. In general,

Table 1. Seattle case history tunnel ground volume loss

Project (Owner)	Shield Diameter of Tunnel External Diameter (m)	Typical Range of Ground Volume Loss, V_l, (%)	Remarks
Mercer Street Combined Sewer Overflow (CSO) (King County)	5.1	<0.5	Lovat EPBM. Soil cover 6.1 to 52.4 m.
Henderson CSO Tunnel (King County)	5.1	0.1 to 1	Lovat EPBM. Soil cover 3 to 27.4 m.
Beacon Hill Light Rail Tunnels (Sound Transit)	6.4	<0.25 to 0.6	Mitsubishi EPBM. Soil cover 6.1 to 48.8 m.
University Link Light Rail Tunnels (Sound Transit)	6.4	<0.5	Hitachi Zosen and Herrenknecht

the ground volume losses ranged between 0.25 and 1 percent, but provided a mean of about 0.5 percent.

A review of select worldwide case histories for tunnels constructed in medium dense to dense soils using closed-face TBMs indicates that ground volume losses have generally ranged between about 0.2 and 1.3 percent, with a mean of about 0.5 percent (Mair and Taylor, 1997), provided the contractor is using very good construction means and methods.

Case history data presented by Leca and New (2007) indicate that with "special control measures" and "carefully managed" operation that ground volume losses of 0.25 to 0.5 percent are "readily achievable" for closed-face tunneling. However, the data from these projects also shows quite a bit of scatter, between 0.2 and 1 percent ground volume loss, over the length of the tunnels. The paper does indicate that "special control measures" and "careful management" were only required over limited portions of the alignment, which could account for some of the scatter.

Maximum Settlement

In our analysis, we used the following established relationship between ground volume loss and maximum settlement at the tunnel centerline:

$$\delta(\max) = V_l V_t / \mathit{sqrt}(2\pi) i \quad \textbf{(1)}$$

where: $\delta(\max)$ = maximum surface settlement at a tunnel centerline
i = distance from centerline to inflection point
V_l = ground volume loss (percent)
V_t = excavated tunnel volume

Settlement Trough

In our analysis, we assumed that the shape of the transverse settlement trough can be described by a Gaussian function as:

$$\delta(x) = \delta(\max)\, \mathit{exp}(-x^2/2i^2) \quad \textbf{(2)}$$

where: $\delta(x)$ = settlement at a distance × from tunnel centerline
$\delta(\max)$ = maximum surface settlement at a tunnel centerline
z = depth from ground surface to tunnel springline
x = horizontal distance from tunnel centerline
i = distance from centerline to inflection point

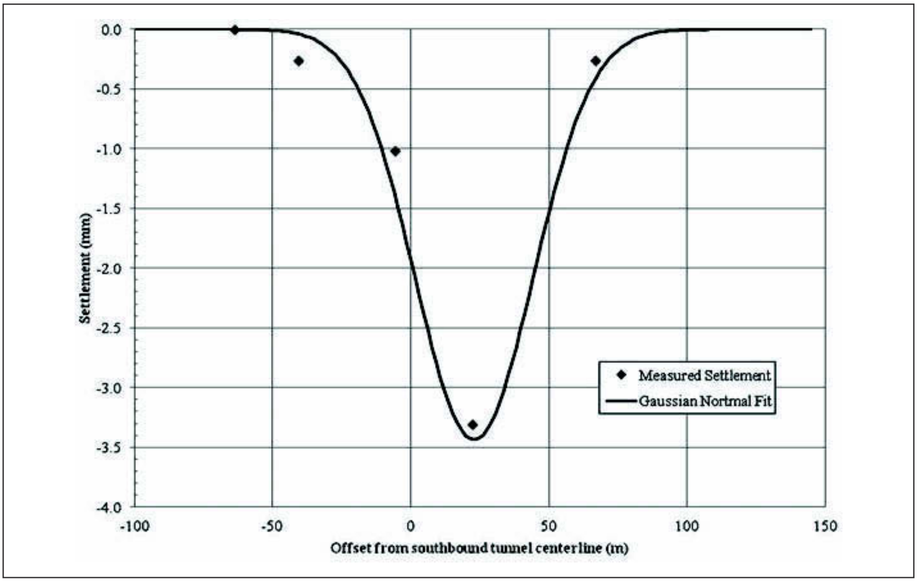

Figure 2. Example of a fit of Gaussian function to the case history data

The shape of the Gaussian function is a controlled in part by the distance from centerline to the inflection point. As the distance to the inflection point (i) increases, the curve is generally wider and flatter and as the distance i decreases, the curve is generally narrower and taller. Case history data presented in Mair and Taylor (1997) indicates i can be related to tunnel depth, where:

$$K = i/z \tag{3}$$

Mair and Taylor (1997) further indicates that worldwide case history data suggest an average value of K=0.35 and K=0.5 for tunnels constructed primarily in sand and clay, respectively. Rankine (1988) indicates that the parameter K ranges from 0.5 to 0.6 with an average value of K=0.55 for tunnels constructed in glacial deposits.

In our review of the data from both the Beacon Hill Tunnel project and the Mercer Street Tunnel project, we varied both the ground volume loss and the distance from the centerline to the inflection point to fit the Gaussian function to the surface settlement data. An example of fitting the Gaussian function to the case history data is presented in Figure 2.

The results from our analysis of nine surface settlement instrumentation sections from the Beacon Hill Tunnel project and data from University Link indicate that the parameter K ranges from about 0.35 to 1.0, with an average value of about 0.6 for cohesive and non-cohesive, glacially overconsolidated soils. The variation in the parameter K with tunnel depth for the case history data is presented in Figure 3.

NUMERICAL ANALYSIS

A series of numerical analyses using FLAC (Itasca, 2008), a two-dimensional explicit finite difference program, were performed to analyze the shape of the surface settlement trough in glacial soils. The analyses were originally performed as generic cases for an approximately 17.7 m diameter tunnel as part of early studies for the Alaskan Way Viaduct Tunnel. Twelve numerical models using typical engineering soil properties for the glacially overconsolidated sand and clay soil commonly encountered in

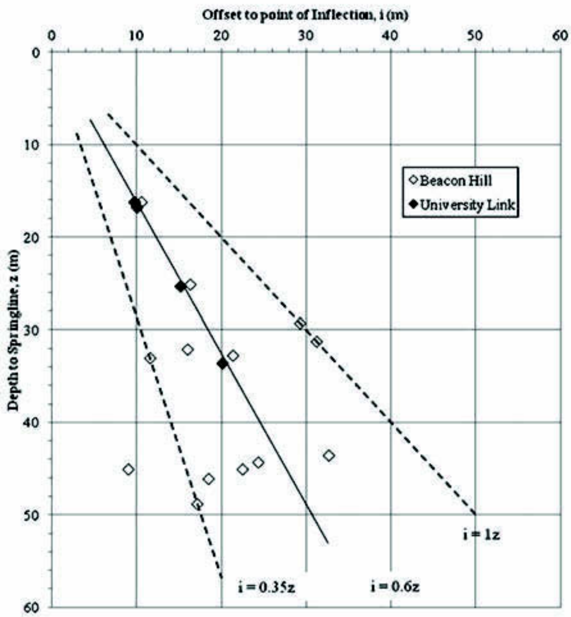

Figure 3. Variation in parameter K with tunnel depth for case history data

Seattle were constructed. Three tunnel depths, 22.9, 45.7, and 68.6 m, were analyzed. To account for at-rest earth pressures (Ko), the analyses were performed for Ko=1 and 2. Ground volume loss was modeled by effectively reducing the diameter of tunnel opening until ground volume loss at the surface was equal to the desired volume. Using the results of the FLAC analysis, we fit the Gaussian function by varying the distance to the inflection point. For the sand case and for tunnels with one diameter of cover over the crown, high Ko values tended to reduce the total maximum settlement and increase the width of the settlement trough. However, for all of the clay cases and for the sand cases with two diameter or more cover over the crown, varying Ko did not appear to have significant impact on the shape of the settlement trough. Typical fitting of the Gaussian function to the FLAC analysis for the glacial sand and glacial clay cases and for depths of 45.7 and 68.6 m are presented in Figures 4 and 5, respectively.

The results from our analysis suggest the parameter K ranges from about 0.6 to 1.2 for tunnels in glacially overconsolidated soil. Similar to the results from our review of the case history settlement data, for tunnels with one or more tunnel diameters of cover, the results of our analyses suggest an average value of about K=0.7 for both cohesive and cohesionless glacially overconsolidated soils. The variation in the parameter K with tunnel depth for the FLAC analysis is presented in Figure 6.

Additional FLAC analyses were performed to simulate varying thicknesses of normally consolidated soils and glacial soil over the tunnel crown. The results of these analyses suggest that the thickness of glacial soils over the tunnel crown plays an important role in the shape of the resulting settlement trough. For the cases analyzed, where at least one diameter of the cover is glacially overconsolidated clay, the shape of the settlement trough appears to be controlled by the glacial soils. For example, with two diameters of cover, and 50 percent of the cover consisting of normally consolidated sand, a Gaussian function with a distance to the point of inflection corresponding to K=0.5 appeared to be a reasonable fit. When less than two diameters of cover were

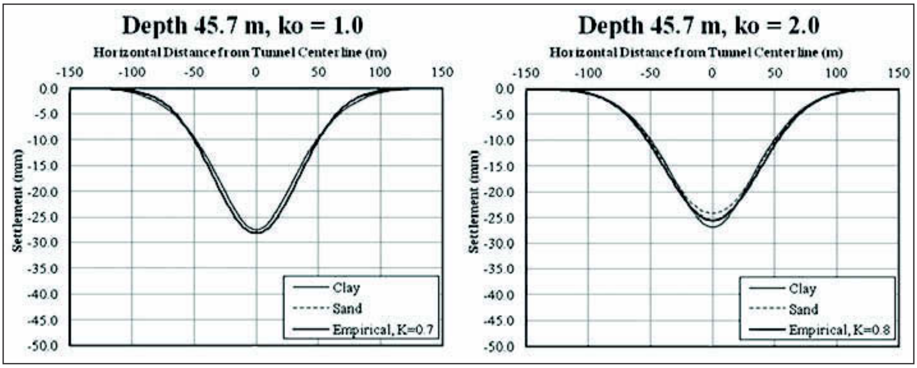

Figure 4. Example of a fit of Gaussian function to the FLAC analysis for tunnel depth of 45.7 m and Ko=1 and Ko=2

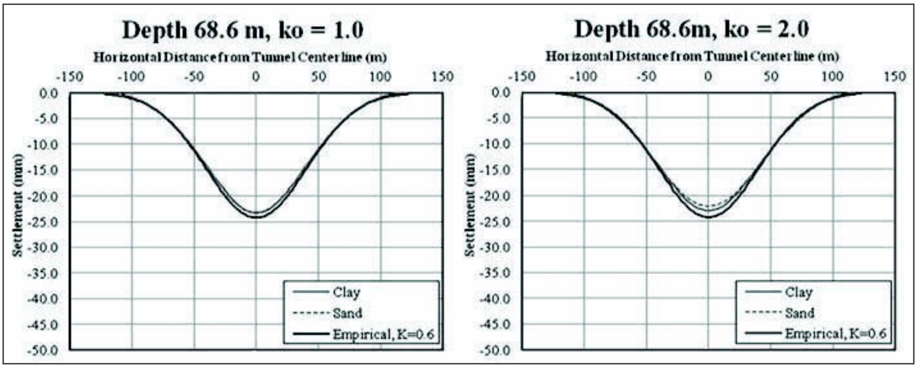

Figure 5. Example of a fit of Gaussian function to the FLAC analysis for tunnel depth of 68.6 m and Ko=1 and Ko=2

modeled and the percentage of normally consolidated sand increased above 50 percent, a Gaussian function with a distance to the point of inflection corresponding to K=0.35 appeared to be a reasonable fit. Based on the additional analysis and for the purposes of performing an initial assessment of tunneling-induced risks, in our opinion where less than two diameters of cover are present and less 50 percent of the soil present is glacially overconsolidated, average case history value for normally consolidated soils such as those presented in Mair and Taylor (1997) appear to be more applicable.

CONCLUSIONS

This paper provides ground volume loss and trough wide parameters that could be considered when performing an initial assessment of tunneling-induced risk along proposed pressurized TBM alignments in glacial soils. Case history data for recent pressurized TBM projects in glacial soil suggest ground volume loss values typically range between 0.25 and 1 percent. However, the ground volume loss parameter should be selected with consideration of the risk to adjacent facilities and the owners risk tolerance.

Both case history data and the FLAC analyses suggest a trough width parameter K of about 0.6 to 0.7 appears to be applicable for both glacially overconsolidated sand and clay where at least one tunnel diameter of cover is present. Where the thickness

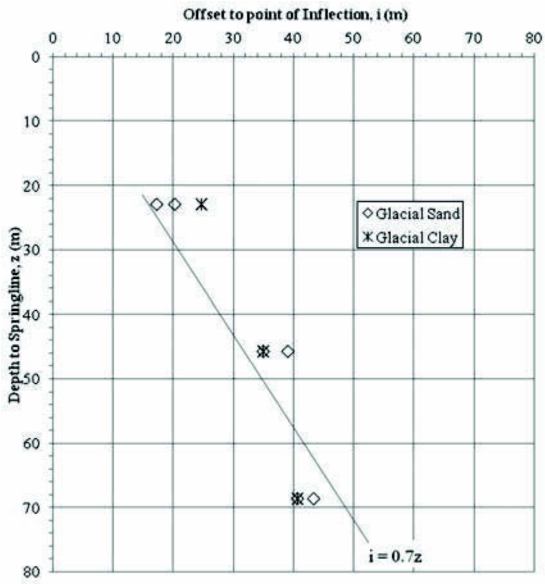

Figure 6. Variation in parameter K with tunnel depth for the FLAC analysis

of glacial soil comprising two tunnel diameters is less than about 50 percent, typical values for sand and clay, between 0.25 to 0.45 and 0.4 to 0.6, respectively provided in the literature (Mair and Taylor, 1997) appear to be more applicable.

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COMPARISON OF PREDICTED VERSUS OBSERVED STRUCTURAL DISPLACEMENTS OF EXISTING STRUCTURES AT THE PORT OF MIAMI

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ABSTRACT

The Port of Miami Tunnel Project is currently being constructed near downtown Miami, Florida to relieve congestion downtown due to port related traffic. The project consists of twin bored tunnels excavated by a hybrid Tunnel Boring Machine (TBM) within complex mixed face conditions beneath existing surface structures located on Dodge Island. Several structures were identified as critical within the zone of influence and were considered to be sensitive to ground settlements induced by the tunneling operation. Complex three-dimensional and two-dimensional finite element analyses were conducted incorporating the twin-TBM tunnels and existing Dodge Island structures. The effects of the TBM tunneling and the behavior of the structures due to the tunneling-induced ground settlements were explicitly modeled. This provided structural displacements which were compared to the respective capacities of the elements to evaluate the potential of damage to the structures. Accordingly, building elements were highlighted which were most sensitive to ground movements and were closely monitored. The results of the numerical modeling assured the owner that tunneling could be undertaken without the use of additional measures such as ground improvement or underpinning of the structures prior to the TBM reaching Dodge Island. Extensive instrumentation was installed on the existing structures and data was evaluated daily throughout the tunneling to ensure the structures were performing as predicted. A thorough comparison of observed structural displacements to predicted movements shows that the Dodge Island structures performed as or better than expected and validated that the structures were only affected within the range of permissible limits.

INTRODUCTION

The Port of Miami Tunnel project is currently under construction and consists of twin bored tunnels constructed between Watson Island and Dodge Island near downtown Miami (see Figure 1). Once completed, port related traffic will be diverted away from downtown city streets and have direct access to I-395 and I-95. To complete the project, a Public-Private Partnership (PPP) was established between the Florida Department of Transportation, Miami-Dade County, the City of Miami, Meridiam Infrastructure Finance, and Bouygues Travaux Publics as part of the design-build-finance-operate-and-maintain (DBFOM) contract. Bouygues Civil Works Florida (BCWF) acted as the prime contractor for the project.

To construct the tunnels, a specially designed hybrid Tunnel Boring Machine (TBM) was built by Herrenknecht in Germany and is currently the largest diameter soft ground TBM in the United States (see Figure 2). Adding additional challenges, no tunnel project of a similar scale has been attempted in the South Florida region. The

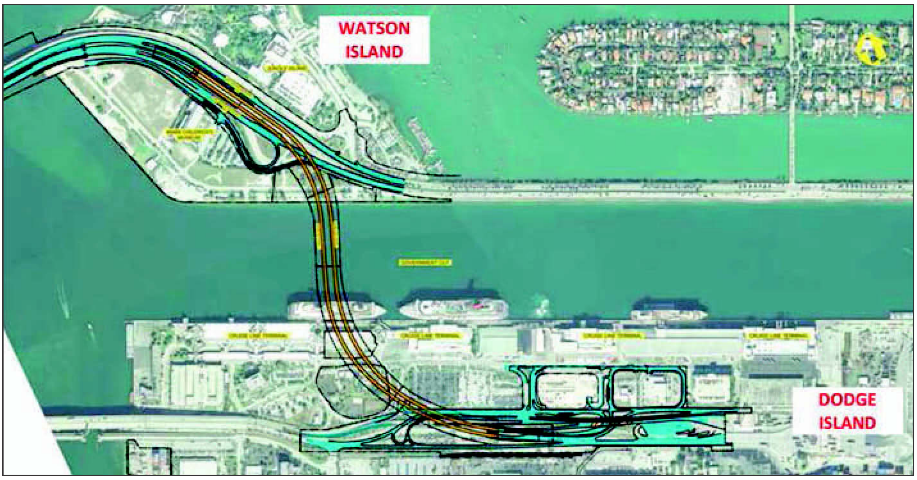

Figure 1. Port of Miami Tunnel project overview

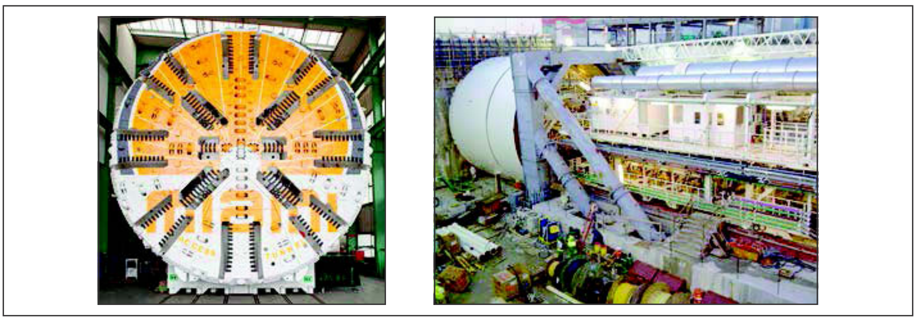

Figure 2. Port of Miami Tunnel TBM cutter head (left) and trailing gear (right)

TBM was launched on November 11, 2011 from Watson Island to begin the Eastbound tunnel boring and broke through at Watson Island on July 31, 2012 to complete the Eastbound tunnel construction. The machine was repositioned and launched again on October 29, 2012 to begin the Westbound tunnel with construction expected to be completed by the Spring of 2013.

On the northern shore of Dodge Island, the TBM was required to pass beneath several existing structures currently in use by the various cruise lines that operate out of the Port of Miami. As the cruise lines are so influential in the local economy, the tunneling operations were not permitted to negatively impact the structures in any way. Several critical structures were identified as posing risk of damage due to tunneling-induced settlements described below and shown in Figure 3 and Figure 4.

- Seawall & Bulkhead—Corrugated steel sheet piling seawall, reinforced concrete pile cap, and tieback to sheet piling "dead man" anchor system. Originally constructed in the late 1950s and exhibits corrosion and deterioration of concrete and steel due to seawater exposure.
- Pedestrian Bridge—Elevated steel structure providing access to loading gantries for cruise ship embarking. Supported by reinforced concrete columns on drilled pile foundations.

Figure 3: Pedestrian Bridge (left) and Shed #2 (right) on Dodge Island

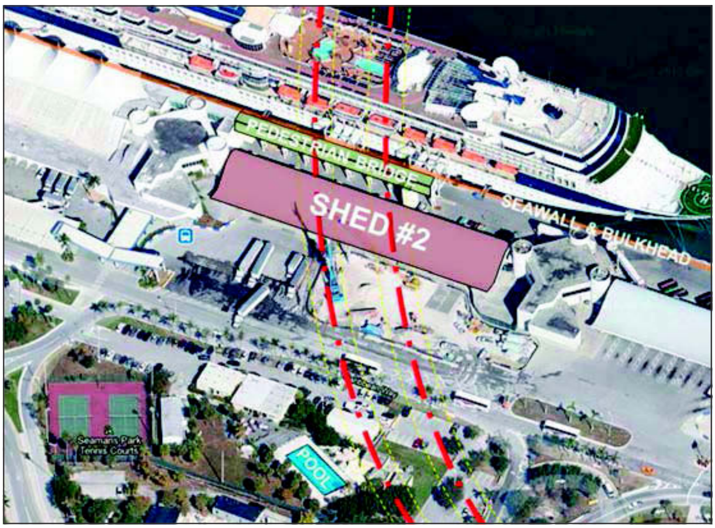

Figure 4. TBM orientation to existing structures on north shore of Dodge Island

- Shed #2—Open-air reinforced concrete frame structure currently used for cruise ship supply storage. Originally constructed in 1967, the building shows extensive superficial concrete deterioration and spalling due to environmental exposure.
- Seaman's Center Swimming Pool—Recreational swimming pool within the influence zone. The owner was concerned with potential concrete cracking and water loss due to ground settlements.

Additionally, the TBM was required to pass close beside drilled piles supporting a two-span bridge abutment. Other construction activities associated with the project required that the Mechanically Stabilized Earth (MSE) ramp structure be demolished prior to the TBM reaching the abutment location, resulting in the structure seen in Figure 5. While the TBM did not undermine the pile tips, lateral ground movements acting on the piles resulting in instability of the abutment were a concern.

GEOLOGICAL CONDITIONS

The geologic history of the Miami area provided challenging geotechnical conditions with respect to tunneling. A specially designed hybrid Tunnel Boring Machine

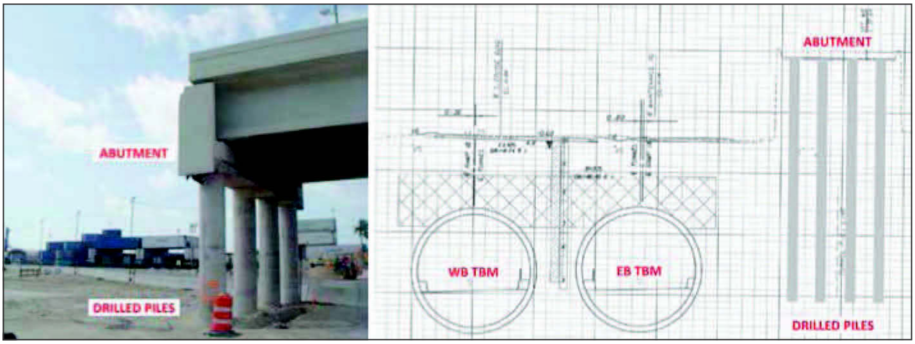

Figure 5. Bridge abutment structure (left) and orientation to TBM tunnels (right)

(TBM) was selected for the project which allowed for tunneling beneath the water table and channel through difficult mixed face conditions. Tunneling generally occurred through four geologic formations:

- Fort Thompson Formation
- Anastasia Formation
- Key Largo Formation
- Tamiami Formation

The Key Largo Limestone, the Anastasia and Fort Thompson formations, and the Miami Limestone present on Dodge Island were formed from expansive coral reefs which covered the Florida Platform at the end of the Pliocene Epoch—roughly 2.6 million years ago. Sediments from the eroding Appalachian Mountains to the north were deposited into natural grooves present in the coral reefs, which created localized areas of sands, silts and clays.

The geotechnical investigation program conducted for the Port of Miami Tunnel Project identified eight (8) distinct ground strata in the area, with most of the tunneling being within Strata 6 through 8. Dodge Island is a man-made island formed of reclaimed land which was dredged from Biscayne Bay and deposited during the deepening of the Port of Miami. These upper fill layers generally consist of sand, silty sand, and silt and overlay the rock formations. Beneath, the Fort Thompson Formation is characterized by a pale orange to yellowish-grey fossil-rich wackestone/packstone containing corals, bryozoans and mollusks (Stratum 6). Underlying the Fort Thompson Formation is the porous, coquina and coquinoid limestone of the Anastasia Formation (a grainstone) and the fossil-rich, coralline Key Largo Limestone (a boundstone), which contains coral heads, bryozoans and mollusks encased in calcarenite (Stratum 7, Figure 6). This unit also contains isolated zones of loose, uncemented sands and silts much weaker than the surrounding limestone. The Anastasia and Key Largo formations may occur as interfingered lenses and layers within the basal Fort Thompson Formation. The ground investigation terminated in the Tamiami Formation (Stratum 8), a grey, porous grainstone with layers and lenses of shelly sands and sands interbedded with clays and silts.

While the Fort Thompson and Tamiami Formation generally exhibit high degrees of cementation resulting in a strong competent limestone, the interbedded Anastasia and Key Largo Formations present grooves of uncemented sands and silts between the more competent rock material. Consequently, geotechnical parameters derived during the ground investigation of this stratum varied widely depending on whether the borehole penetrated the rock or the uncemented sands/silts. Modeling of this material proved to be difficult and predictions of surface settlements from the finite element analysis were highly dependent on the geotechnical parameters chosen for this

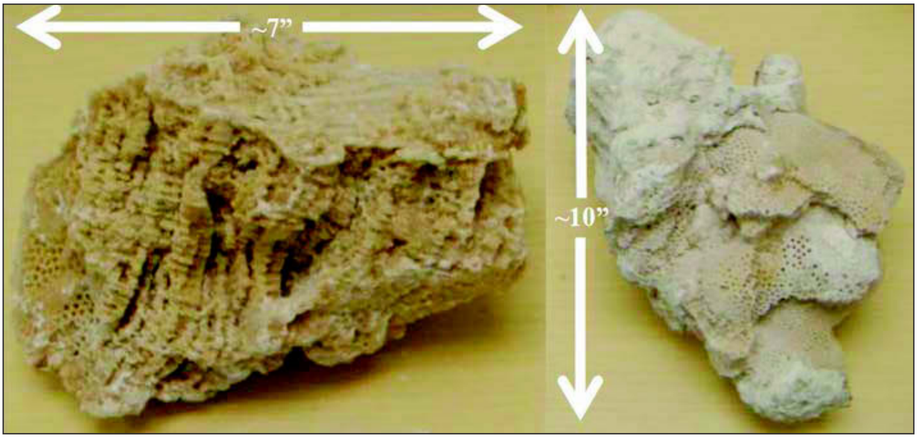

Figure 6. Samples of Stratum 7 material—Key Largo Limestone

stratum; however, due to the large variation of observed data, conservative estimations for compressive strength and Elastic Modulus were necessitated.

FINITE ELEMENT ANALYSIS

North Shore Structures

To evaluate the influence of the TBM excavation on the existing structures, an independent three-dimensional ground structure interaction analysis was performed for the northern shore of Dodge Island. Shed #2 and the Seawall & Bulkhead were modeled explicitly to assess the behavior of the structures due to the tunneling-induced ground settlements. Additionally, structural element loadings were calculated which were compared to the design capacity to determine whether the structure was still within acceptable loading limits. The model was constructed using the Midas GTS 2012, v.1.1 software utilizing the DIANA solver by TNO DIANA BV for nonlinear analyses. Ground materials were modeled with three-dimensional, 4-node tetrahedral elements with a Mohr-Coulomb failure criterion. The extents of the modeling geometry and various ground strata are shown in Figure 7.

Shed #2 is an open-air reinforced concrete frame structure consisting of prefabricated Double Tee roof beams resting on S-shaped support elements on drilled pile foundations (see Figure 3). The structure is nearly perpendicular to the tunnel alignment but does not provide structural connecting elements laterally between the frames besides the roof Double Tee's. Therefore, the structure behaves "flexibly" during the tunnel construction and is tolerant to differential displacements between the frames. In the numerical model, the structure was approximated as one-dimensional elastic beam elements and two-dimensional elastic plate elements (see Figure 7).

Horizontal displacements of the Seawall towards the respective TBM face resulting in global stability issues of the Seawall and dead man system were of concern. Therefore, the Seawall, dead man, and tie backs were explicitly modeled with two-dimensional elastic plate elements and one-dimensional tension-only elastic rod elements (see Figure 7).

Bridge Abutment

A two-dimensional finite element model was developed to analyze the bridge abutment behavior as the tunnels passed beside the drilled piles. In the area near the bridge abutment, soil material above the tunnel crowns and below the abutment ramp was

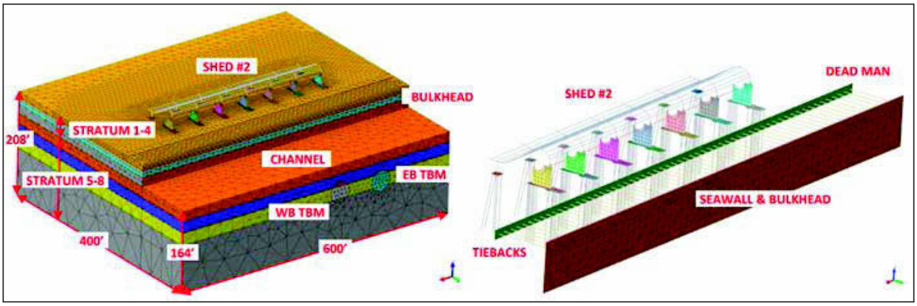

Figure 7. Overall model geometry (left) and existing structures discretization (right)

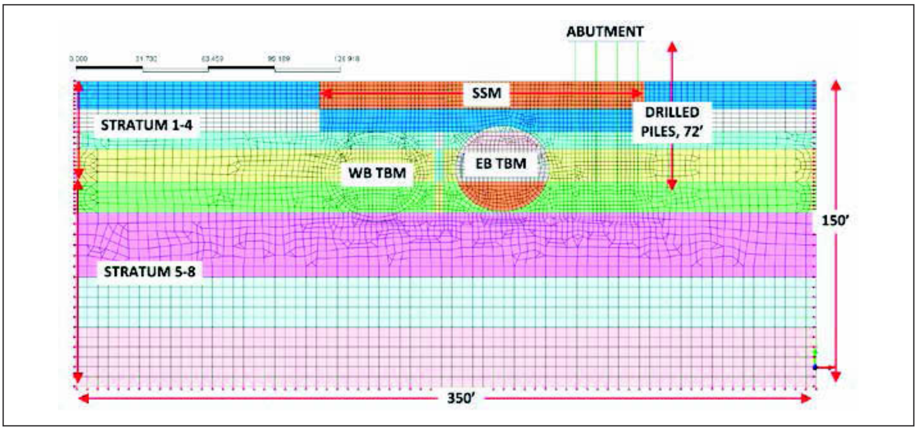

Figure 8. Overall geometry for bridge abutment model

improved using Shallow Soil Mixing (SSM) techniques and was included in the model. Abutment structural elements and drilled piles were modeled by one-dimensional elastic beam elements, while ground materials were modeled using 3-node triangular and 4-note quadrilateral elements with a Mohr-Coulomb failure criterion (Figure 8).

Assessing the TBM Excavation

To simulate the TBM excavation, the Elastic Modulus of the material within the excavated volume along the alignment was reduced by a "Softening Factor." This process models movement of ground material into the excavated area and into the face prior to installation of the tunnel lining and mobilizes the strength of the surrounding ground. While the softening factor is generally obtained from research or empirical relationships; in this case monitoring data was available for the Watson Island segment of the alignment and was used to calibrate the softening factor used in the model. A lower and upper bound Softening Factor of 30% and 50% were selected to incorporate the variety of ground settlements observed at Watson Island during the beginning of the TBM drive.

INSTRUMENTATION OF EXISTING STRUCTURES

Extensive monitoring installed on the existing structures provided the opportunity to observe the structural displacements during tunneling and compare to movements predicted by the finite element analysis. The Seawall & Bulkhead was monitored by

Figure 9. Instrumentation in Shed #2 (left) and automated total stations used for data collection (right)

mounted three-dimensional optical prisms surveyed by an automated electronic theodolite total station. Horizontal movements of the sheet pile toe were observed with the use of multi-point borehole extensometers (MPBX's) installed behind the seawall.

Mounted three-dimensional optical prisms were also installed throughout Shed #2 and the Passenger Bridge and were recorded twice per day by an automated total station when the TBM was within the vicinity. Rotations of the S-frame supports were monitored by two-dimensional electronic tilt meters, while existing concrete cracks and expansion joints were monitored by several crack meters (Figure 9).

PREDICTED VERSUS OBSERVED STRUCTURAL DISPLACEMENTS

North Shore Structures

As of the publication deadline the Westbound TBM has not reached the northern shore structures and therefore, data is only available for the Eastbound tunnel drive. The finite element analysis predicted a relatively wide settlement trough due to the more competent rock material above the tunnel crown acting to disperse the surface settlements laterally across the structure, as shown in Figure 10.

This behavior was confirmed by instrumentation installed on Dodge Island and led to only minor differential displacements and rotations of the structures as shown by the vertical structural settlements in Figure 11.

As expected, settlements observed at the Seawall & Bulkhead structure were less than those observed at Shed #2 due to the stiffening effect of the steel sheet piling. Vertical settlements exhibited a wide, shallow trough resulting in minimal differential displacements as shown in Figure 12.

Bridge Abutment

The model predicted a maximum vertical displacement of 0.35 in after the Eastbound TBM has passed, whereas approximately 0.3 in were observed from monitoring data and closely followed the global behavior of the abutment structure reacting to the passing TBM (see Figure 13). As the Westbound TBM passed the structure, only slight deformations were actually observed by the installed instrumentation. The model predicted slightly more displacements by the structures as the second drive was constructed due to the interaction between the SSM improved ground and the non-improved ground.

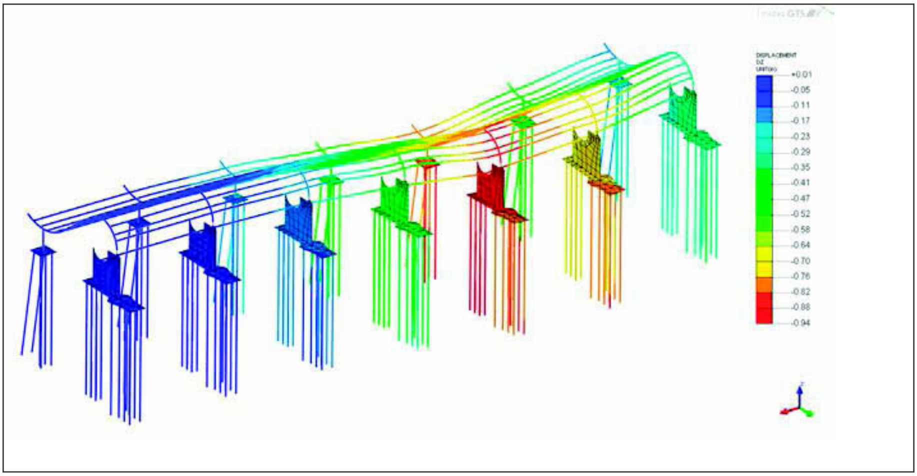

Figure 10. Vertical structural displacements predicted by 30% softening model of Shed #2

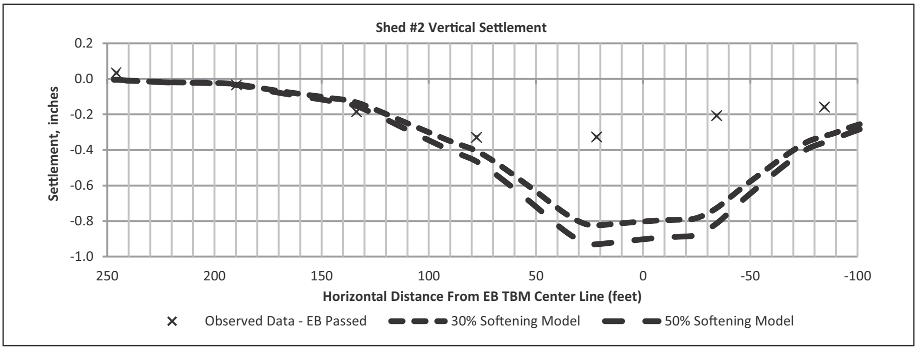

Figure 11. Comparison of predicted to observed structural settlements for Shed #2

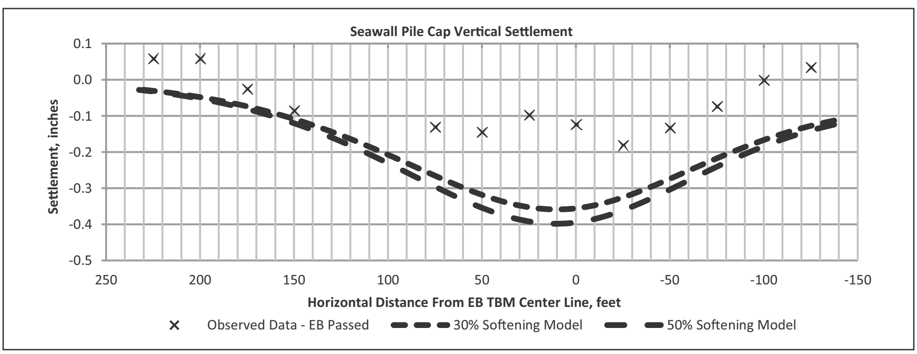

Figure 12. Comparison of predicted to observed structural settlements for the Seawall & Bulkhead

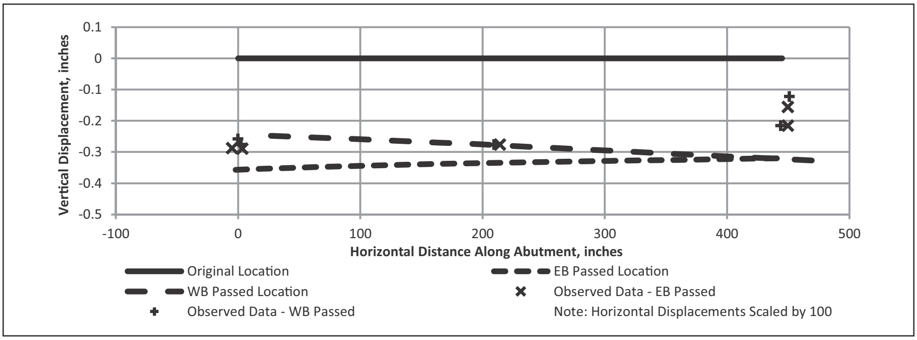

Figure 13. Comparison of predicted to observed structural displacements of bridge abutment

CONCLUSIONS

While the settlements observed from instrumentation of the structures proved to be less in magnitude than those predicted by the analysis, the settlement troughs displayed the wide and shallow trough anticipated by the three-dimensional finite element model. The two-dimensional analysis performed for the bridge abutment closely projected the observed behavior of the structure due to the TBM excavation adjacent to the drilled piles. This would suggest that the Softening Factor selected during the calibration exercise was relatively accurate; however, the geotechnical parameters chosen for Stratum 7 were conservative. As a conservative assumption, lower bound parameters were selected for this material; however, it has been shown at the Port of Miami Tunnel Project that Stratum 7 is more competent than was observed at the localized zones of silt and sand. The composite structure of the interbedded Anastasia and Key Largo Formation can provide competent support for the overlying strata and has not exhibited the tendency for high volume loss potential.

Potential of damage to the existing Dodge Island structures was successfully evaluated using the structural displacements calculated from the numerical analyses, in spite of the challenging geotechnical conditions observed at the project site and lack of local related projects from which to draw experience. Due to the highly three-dimensional nature of the interaction between the tunneling-induced settlements and the movements of the northern shore structures, a three-dimensional numerical analysis was considered critical. By explicitly modeling the twin tunnels and the existing structures, the structural deformations caused by the underground operations on Shed #2 and the Seawall could be assessed. By undertaking the numerical analyses, the project owner was assured that the risk of damage to the structures was at a minimum. To ensure the structures would not be negatively impacted as a result of tunneling, extensive instrumentation was installed. During construction, monitoring data was evaluated daily to validate the results of the modeling and further reassure all parties involved of acceptable structural deformations.

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GEOTECHNICAL INSTRUMENTATION MONITORING SYSTEM FOR SHALLOW FREEWAY TUNNEL CROSSINGS WITH EPBMS

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ABSTRACT

The U230 Contract for Sound Transit's University Link Light Rail Project in Seattle, Washington, required two EPBM tunnel drives beneath a major arterial freeway. Each tunnel had roughly one diameter of cover or less at the crossing, with tight WSDOT-imposed limits on allowable ground movements. A redundant geotechnical instrumentation monitoring system was developed during design, and modified slightly and carefully monitored during construction. Precision excavation and close collaboration of all parties resulted in successful crossings. Resultant movements were well below allowable limits. This paper discusses this monitoring system and insights gained into monitoring and managing settlements for shallow tunnel crossings.

INTRODUCTION

The University Link (U-Link) Project is a critical extension to Central Puget Sound Regional Transit's (Sound Transit) light rail system in Seattle, WA. The U230 Contract extends the existing system from downtown Seattle up to the heavily populated Capitol Hill neighborhood to the northeast. As part of the alignment, the twin tunnels both cross beneath the Interstate 5 (I-5) freeway, the major arterial through the Downtown Seattle area. Cover above these tunnels is less than a diameter beneath the high-occupancy vehicle (HOV) lanes, and just over a diameter beneath the main travel lanes in the southbound direction. See Figure 1 for a picture of the I-5 at the undercrossings, and Figure 2 for a profile of the alignment.

Working closely throughout design and construction with the Washington State Department of Transportation (WSDOT), the project team implemented requirements to limit impacts to the freeway. As part of these requirements, a robust and redundant geotechnical instrumentation system was set up for monitoring movements throughout construction. After the first successful crossing of the freeway, modifications to the system were made for the second crossing, which also proceeded without incident. This paper discusses the background and requirements for the project, the geotechnical program developed during the design phase, modifications to the program during construction, results of settlement monitoring during the crossings, and recommendations for future crossings with similar conditions.

Figure 1. I-5 Freeway at undercrossing (looking North-Northeast)

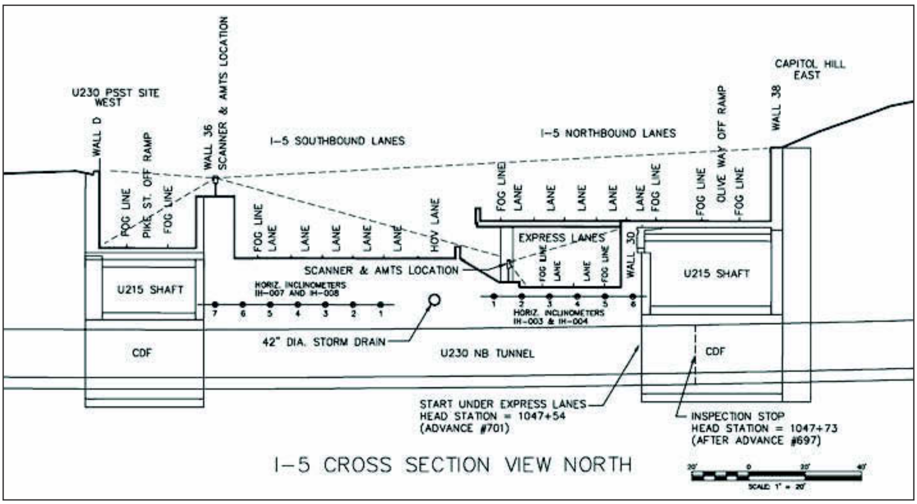

Figure 2. Cross section below I-5 (Source: JCM)

PROJECT BACKGROUND

The U-Link Project provides an extension to the current system that runs in the Downtown Seattle Transit Tunnel (DSTT). A previous contract created the Pine Street Stub Tunnel (PSST) beneath heavily traveled Pine Street, both to provide a turnaround point for trains in the DSTT and to provide for future extensions to the system. The U-Link Contract U230 extended the system from the PSST up to the Capitol Hill Station, with twin tunnels running approximately 1,158 m (3,800 ft) to a new underground cut-and-cover station.

As part of the alignment, the twin tunnels pass beneath the I-5 freeway. As part of the previous U215 Contract, large-diameter secant piles had to be removed from the path of the tunnel boring machines (TBMs). During both design and construction, Sound Transit worked closely with the WSDOT, as well as designers and the contractors, to provide a successful construction project. WSDOT set critical movement levels for the retaining walls adjacent to the freeway, overpass structures in proximity to the tunnels, and the pavement itself. A Yellow, or Threshold, Level was set at approximately 12.7 mm (0.5 in.), and a Red, or Maximum, Level was set at approximately 25.4 mm

(1 in.). Additional requirements for observations and repairs were also set, including regular monitoring of both settlement and TBM performance, real-time data presentation via a web-site, and emergency plans in case levels above were exceeded.

DESIGN DEVELOPMENT

A number of issues were carefully considered during the design phase, including requirements for tunneling beneath the I-5, estimating of settlements and setting of action levels, and development of geotechnical instrumentation requirements.

Tunneling Requirements

During the design phase, requirements were set for TBM capabilities, operations, and maintenance prior to and during the crossings. Primary requirements for the TBM to limit settlements during mining below the I-5 freeway included the capability to provide positive face pressure to resist hydrostatic and earth pressures (EPBM); and the ability to inject bentonite around the shield through ports to limit shield-related ground losses. Operational provisions included 24-hour mining during the crossing to limit the number of prolonged stoppages, and careful coordination between all parties and contingency plans during the crossings to prepare for any foreseen and unforeseen circumstances.

As part of the previous U215 Contract, controlled density fill (CDF) blocks were created on either end of the two TBM drives to replace removed material and stabilize the excavations. The blocks on the east side of the crossings were required to be used to observe TBM and cutterhead conditions under free air prior to the crossings, and to perform maintenance if needed. These inspection and maintenance stops were intended to limit potential TBM issues and maintenance during the crossings themselves.

Estimated Settlements and Action Levels

As part of the approval process from WSDOT for the crossings, estimates on the intended settlements were performed during the design phase. The two crossings were far enough apart to limit overlap of settlement profiles. However, the shallow alignments beneath the I-5 presented challenges for limiting settlements to levels acceptable to WSDOT. Further complicating the crossing was a 1,065 m diameter (42 in.) storm drain beneath the main southbound travel lanes with only limited access for monitoring and observing during construction.

A number of analyses were performed to estimate settlements, using a range of assumptions for anticipated ground loss and settlement trough width. Results from these analyses showed estimated settlements of between approximately 6.3 and 12.7 mm (0.25 and 0.5 in.). Two action levels were then set for settlements based upon allowable limits set by WSDOT, as discussed above. A Trigger Level of 12.7 mm was set, above which additional monitoring and consideration of additional contingency measures were required. A Maximum Level of 25.4 mm (1 in.) was also set, above which TBM operations could be halted, and the freeway itself shut down. Some post-construction verification of impacts to the freeway paving was also required if either or both levels were exceeded, including repairs to the pavement surfaces.

Geotechnical Instrumentation

Monitoring of movements during the crossings was critical to successful construction. The primary goals of the instrumentation system were real-time monitoring (data available hourly) of movements to allow contingency measures to be implemented; dissemination of the information to the team via a web-based portal; redundancy of instrumentation to provide a check on information, and in case one system was not working properly; and an accurate system as action levels were set at low values.

The previous U215 Contract required significant instrumentation to monitor movements during work adjacent to the freeway. Most of the instrumentation was associated with the retaining walls on either side of the freeway, as impacts of this contract were more focused on the modifications to the existing retaining wall structures. Instrumentation consisted of linked beam sensors and tiltmeters installed on subject walls, inclinometers, glass reflector targets installed on the retaining walls read by an Automated Total Station (AMTS), and pulsed laser scanning of the walls with reflective laser targets.

While access to the I-5 limited the installation and use of extensometers within the actual freeway limits, numerous wireless extensometers were installed along the alignments prior to the crossings. The intent was to carefully monitor these extensometers to determine how well the TBM operations were controlling movements. Wireless extensometers provide very quick feedback on movements and allow for adjustments to TBM operations to be made in real time.

Similar to the requirements of the U215 Contract, laser scanning of the freeway pavement was part of design requirements to monitor U230 pavement movements of I-5. This method was intended to provide detailed movements across the area of concern, and could be compared to previous readings to determine how movements were developing. As discussed later in this paper, issues with processing of data made it very difficult to have real-time information using this method. Concerns about accuracy had also been also raised during the U215 Contract.

As a redundant form of instrumentation for the crossing, horizontal in-place inclinometers (HIPI) were installed between the tunnel alignment and the overlying pavements. These devices were installed during the previous U215 Contract, and wiring was routed to locations that were accessible after completion of that contract. A total of eight HIPIs were installed—two HIPIs above each tunnel installed from each side of the freeway crossing.

Installed Instrumentation

During U215 construction, extensive monitoring of instrumentation was implemented, and some of the instruments installed during the U215 Contract were intended to monitor the U230 undercrossing as well. The discussion here will focus on the systems used, and important lessons learned. During the U215 Contract the eight arrays of HIPI's were installed from within shaft excavation as construction proceeded. Each array included six to eight 3 meter (10 foot) gage lengths, installed under the North HOV and South bound lanes of I-5. HIPI's were chosen as they could be installed in horizontal holes drilled form within the excavations, thus minimizing disruption of traffic on I-5. These installations precluded installation of other systems, such as vertical multiple position extensometers, as these would have required several days of closures on I-5 to drill and install the vertical instruments. The HIPIs were intended to provide the primary measurement of road movement. The instruments were installed and cables run through the CDF backfill in the U215 excavations, up to the data logger locations on the I-5 retaining walls. Running and protecting cables in CDF blocks during shaft construction was a challenge. Access to the HIPI's would be impossible, as they were to be buried in the CDF backfill. This unique application required careful planning, reading verification and system checking of instruments as no access to troubleshoot and/or check installations would be possible after the CDF was placed. See Figures 3 and 4 for a photograph of HIPI installation.

As a redundant monitoring method, the U230 Contract intended to use pulsed laser scanning to provide real time measurements of the road surfaces above the tunnel alignment to provide real time readings in road surface changes. Laser scanning is

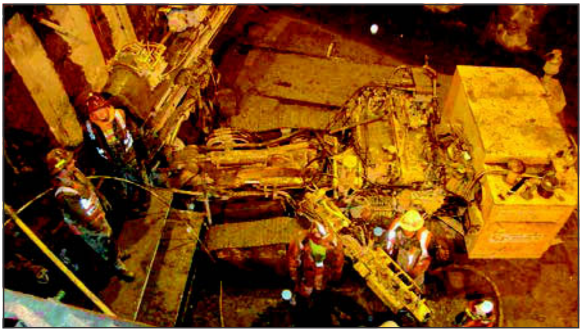

Figure 3. HIPI installation during U215

Figure 4. HIPI casing installation during U215 contract. Note temporary internal push rod and counterweight for casing grouting operation.

a powerful tool, and provided meaningful results on the U215 Contract with the use of scan targets for wall movements, but it became clear during use on U215 that this technology would be unable to provide the real time measurement needs (less than 30 minutes) and measurement accuracy of the project. The laser scanning option was replaced with the use of road prisms, consisting of reflective glass prisms installed in a protective plastic housing and affixed to the road surface with hot melt tar or epoxy. Road prisms were installed at the concrete pavement panel joints, and read with the AMTS system provided for use in the U215 Contract. See Figures 5 and 6 for photographs of a typical AMTS installation, and installed roam prisms, respectively.

For the U230 Contract, the first TBM crossing relied upon the HIPI, road prisms installed within the HOV lanes, and reflectorless EDM pavement monitoring for the I-5 main southbound lanes. The second TBM crossing relied upon the HIPI, and road prisms installed within the HOV lanes and within the I-5 main southbound lanes.

CONSTRUCTION AND MONITORING

Overall, construction of the tunnels below the freeway proceeded with no significant issues. However, significant efforts went into planning and monitoring the crossings.

Planning for Crossing

Prior to both crossings, multiple meetings were held by all parties involved, and responsibilities were carefully assigned. Instrumentation and remote reporting of the instrumentation system was checked and double checked, and personnel were assigned for remote monitoring during tunnel passage. A large part of the success of the crossings was due to this coordination effort. Provisions that were developed during these meetings included a calling tree with multiple individuals at each level; coordinating the start of excavation with the nightly HOV lane closures to limit traffic on the roadway during the initial excavation; having a clean-up team in close proximity in case of any pressure blow-outs of materials onto the roadway; having individuals within the HOV lanes during the road closure to directly observe conditions; having traffic control teams on

Figure 5. Typical AMTS installation

Figure 6. Road prisms at pavement panel corners

stand-by if needed; and round-the-clock tracking of settlement data and TBM performance.

First Crossing

For the first crossing (the Northbound track tunnel), continuous excavation took about 60 hours from November 15th to November 18th, 2011, with mining of approximately 60 m (200 ft) proceeding from east to west. TBM excavation began at approximately 9 PM in order to coincide with an early closure of the HOV lanes, which provided an additional precaution for initial excavation. Excavation parameters from the TBM were carefully monitored, and no unusual excavation volumes or excessive tail void grout volumes were noted. At the median area between the HOV lanes and the main southbound travel lanes, a small open conduit between the tunnel elevation and the ground surface allowed a limited amount of pressurized material from the cutterhead to escape to the surface. This material was quickly cleaned up, and no subsequent issues were observed.

Both the pavement and HIPI were carefully monitored throughout the crossing. Overall, the HOV road prisms provided the most accurate and timely information, and movements appeared to track the TBM progress well. See Figure 7 for a typical time-history plot of a road prism. The HIPI information for the HOV lanes also appeared to provide good correlation with the pavement markers. See Figure 8 for typical movements observed from one of these instruments. For the southbound main lanes, the HIPI showed results within the normal operating range of the instruments, indicating no significant movement during TBM passage. However, after years of proper function, and hours after the TBM completed, the system started reporting suspicious movements along every sensor in the HIPI arrays, indicating linear movement/rotation. While other available information seemed to indicate that no movements should be occurring, the post TBM passage data created significant consternation amongst all parties. The next morning, a systematic analysis of the measurement components related to the

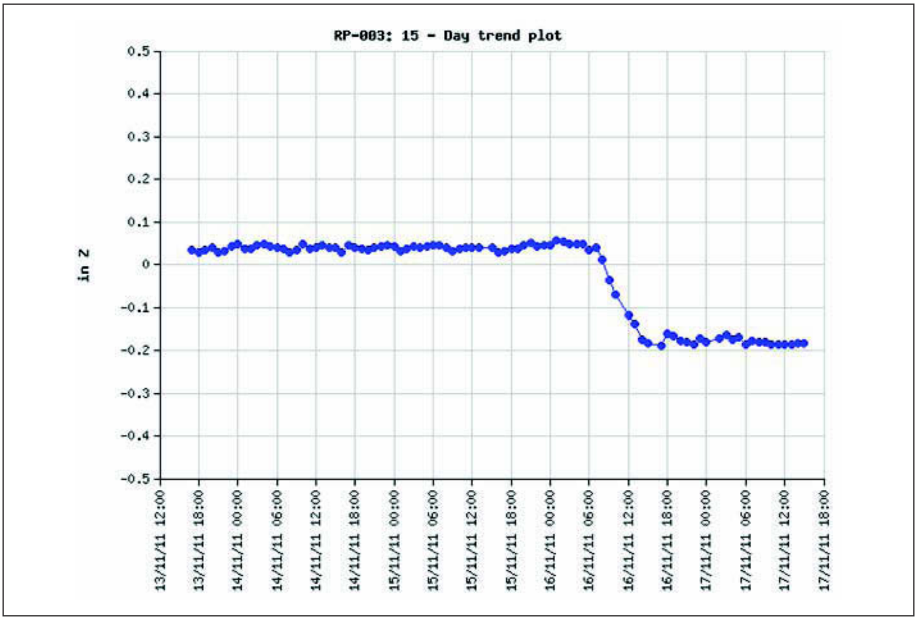

Figure 7. First crossing, typical road prism plot

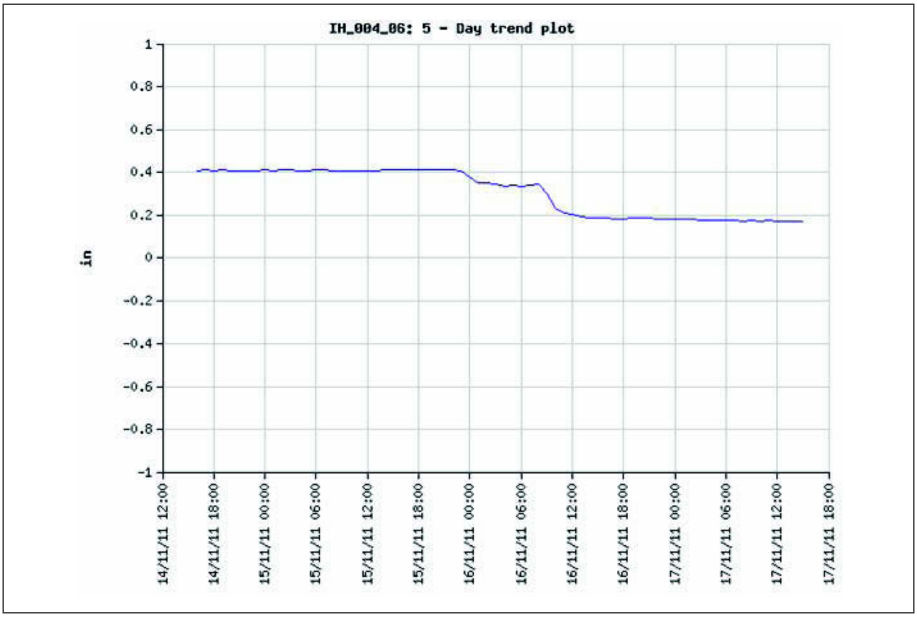

Figure 8. First crossing, typical HIPI deformation plot

HIPI was conducted, and manual and electronic readings were taken of the HIPI sensors, isolating the multiplexer and other measurement peripherals. This process determined that a multiplexer had malfunctioned. Multiplexers are used with datalogging systems to allow measurement expansion of multiple sensors to one logger resource. Normally these components work flawlessly. Since the system was installed and measuring for several years, with readings that were within the normal operating range if the system, this was the last place a problem was expected. With the new manual HIPI readings, road prisms and manual surveys, movements were confirmed to be less than WSDOT imposed levels. As a contingency measure, most of the rings below the southbound lanes were cored and proof grouted. Cores indicated solid grout backfill between the segmental lining and the excavated ground, and grout takes were low throughout. This coring and grouting further confirmed the TBM and manual survey information.

Ground movements were compiled and plotted, with the most complete information coming from the road prisms on the HOV lanes. Resultant movements were on the order of up to 7.6 mm (0.3 in.) throughout the crossing, with movements dissipating from the centerline of the tunnel. Movements were less than the Yellow Level limit set by WSDOT, and thus no additional actions were required.

Second Crossing

For the second crossing (the Southbound track tunnel), continuous excavation took about 67 hours from April 23rd to April 26th, 2012, with mining of approximately 60 m (200 ft) proceeding from east to west. TBM excavation began at approximately 11 PM in order to coincide with the normal closure of the HOV lanes, which provided an additional precaution for initial excavation. Excavation parameters from the TBM were carefully monitored, and no unusual excavation volumes or excessive tail void grout volumes were noted. Data logger systems components were manually checked prior to the tunnel crossing to obtain manual sensors measurements, and compared to automated readings. This minimized the potential for a system component problem such as determined with the multiplexer on the first crossing. No similar issues with multiplexers were detected.

Both the pavement and HIPI were carefully monitored throughout the second crossing, in a manner similar to the first crossing. Overall, the HOV road prisms again provided the most accurate and timely information, and movements appeared to track the TBM progress well. See Figure 9 for a typical time-history plot of one of the road prisms. The HIPI information also appeared to provide good correlation with the road prisms. See Figure 10 for a typical movement plot observed from one of these instruments. No issues associated with the multiplexers occurred for the HIPI for the second crossing, mainly because of careful examination of the units just prior to the crossing.

Ground movements were compiled and plotted, with the most complete information coming from the road prisms. Resultant movements were on the order of up to 7.6 mm (0.3 in.) throughout the crossing, similar to the first crossing, with movements dissipating from the centerline of the tunnel. Movements were less than the Yellow Level limit set by WSDOT, and thus no additional actions were required.

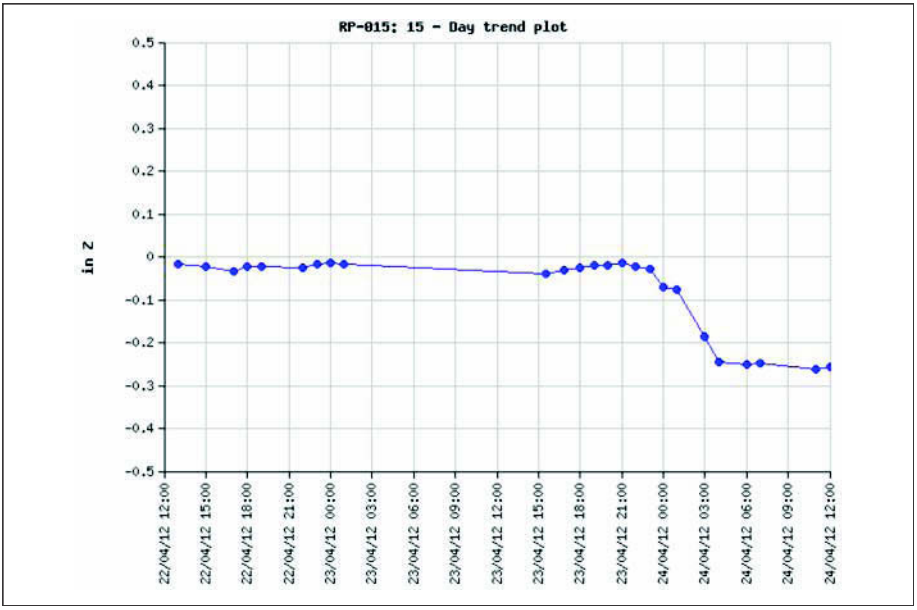

Figure 9. Second crossing, typical road prism deformation plot

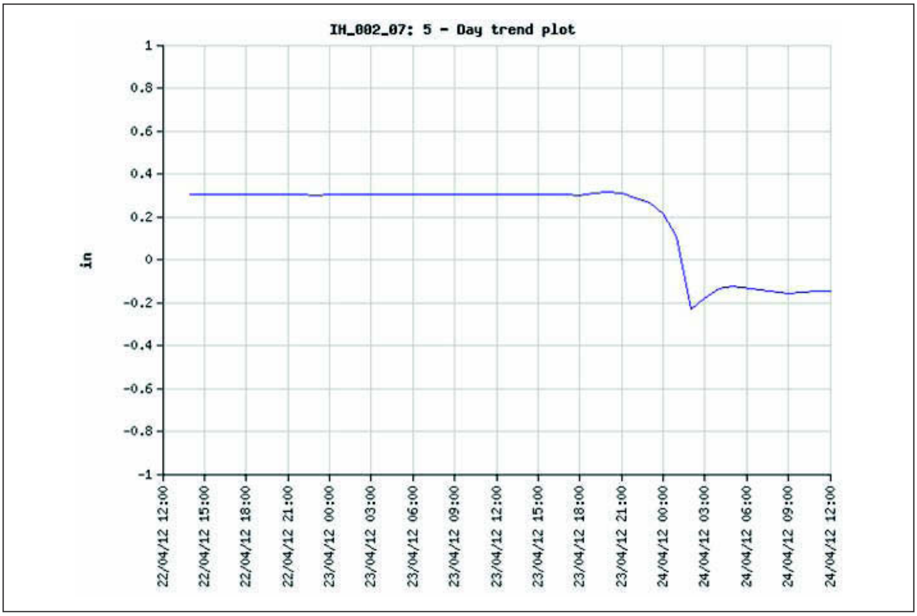

Figure 10. Second crossing, typical HIPI deformation plot

CONCLUSIONS AND RECOMMENDATIONS

Both tunnels were excavated with no significant impacts on the overlying I-5 Freeway. Movements did not exceed any of the limits set by WSDOT for the U230 Contract. For data analysis and reduction, plotting the data from both crossings on a cross section and using a best-fit Gaussian curve, a maximum resultant ground loss of approximately 0.1 to 0.2% was estimated, and a settlement trough width factor of 0.35 to 0.4. See Figure 11 for a plot for the HOV lanes for the first crossing, and Figures 12 and 13 for a plot of the HOV lanes and the main southbound lanes for the second crossing, respectively. The data for the southbound lanes for the first crossing were not considered to have the required accuracy for the development of reasonable correlations.

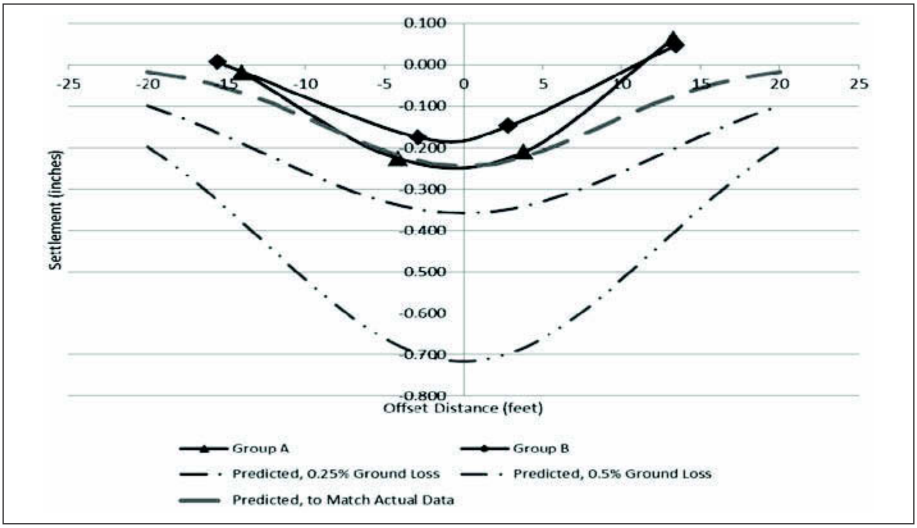

Figure 11. First crossing HOV lanes road prism data, actual versus predicted

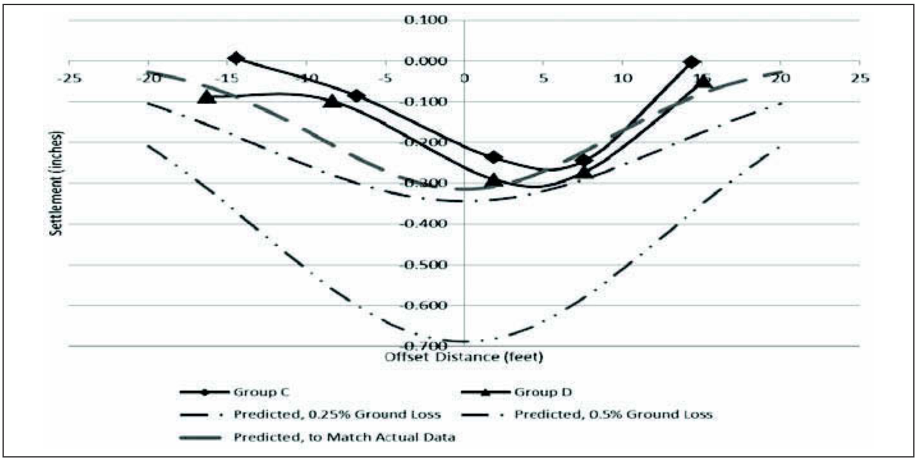

Figure 12. Second crossing HOV lanes road prism data, actual versus predicted

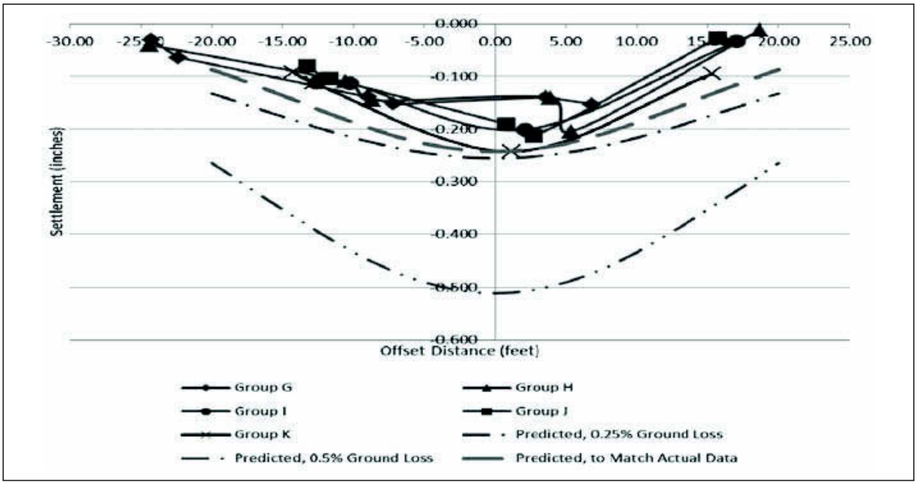

Figure 13. Second crossing main southbound lanes road prism data, actual versus predicted

Based upon experience from this project, a number of recommendations are provided for future projects with similar undercrossings of critical infrastructure:

- Throughout planning, design, and construction, closely work with impacted third parties to develop realistic expectations and goals, including setting points of contact, critical settlement limits for structures of concern, and contingency measures. Meetings at key times are also considered vital, including during early development of the project alignment, at key stages during design, during preconstruction meetings, and prior to the actual crossings.
- Develop detailed requirements for the construction team members, including assigning responsibilities, developing calling trees (include back-up individuals), developing contingency plans for potential occurrences, and making sure key information is being carefully observed and shared within the team.
- For geotechnical instrumentation systems, the following items are recommended:
  - Redundant monitoring systems.
  - Innovative approaches, such as the road prisms used on the U230 Contract. Real-time monitoring using AMTS with reading schedules of every 30 to 60 minutes, and download time to a web-based system of less than one hour.
  - Multiple teams or individuals concurrently watching information and communicating observations.
  - For long term (more than 6 month) applications requiring systems installed long before they are needed, a full check of all components including manual redundant measurement capability should be planned for, and executed within a time frame to execute changes if needed.
  - Do not become complacent with automated readings that are within the normal operating range of the sensor.
  - Question readings that are valid, but make no sense, for example linear readings of movement hours after events should have been detected.

- Provide for tie-in communications and data sharing between the TBM heading and the geotechnical instrumentation.

ACKNOWLEDGMENTS

The success of this project depended upon numerous individuals throughout all the organizations involved, and it would be difficult to list them all. Parties involved in this successful project include Sound Transit staff on both the design and construction management side; the U230 Contractor JCM (Jay-Dee, Michels Coluccio Joint Venture), including help with the undercrossing cross section; Geo-Instruments (subcontractor responsible for instrumentation); WSDOT key personnel; the START CM team (Jacobs Engineering and CH2M Hill Joint Venture); and members of Northlink Transit Partners (AECOM, HNTB, and Jacobs Associates Joint Venture), who were closely involved in project design and monitoring.

INSTRUMENTATION APPROACH FOR THE ALASKAN WAY BORED TUNNEL

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ABSTRACT

Establishing a robust geotechnical instrumentation and construction monitoring plan is one of the keys to the successful delivery of the world's largest soft-ground bored tunnel in Seattle, Washington. This paper presents the Washington State Department of Transportation's (WSDOT) initial instrumentation approach during development of the Request for Proposal (RFP); Seattle Tunnel Partners (STP) and its subcontractor SolData's state-of-the-practice proposal to construction instrumentation and monitoring; WSDOT/STP/SolData's collaboration on modifications to the RFP based on the unique needs of the project site; and, Third Party installation challenges for the over 1,000 instruments installed.

PROJECT OVERVIEW

The replacement of the Alaskan Way Viaduct in downtown Seattle, Washington had been contemplated for over a decade when the approach to use a large-diameter bored tunnel was selected and cleared the Federal and State environmental processes. The world's largest diameter soft-ground bored tunnel to date, the 17.1 m (56 ft) diameter Alaskan Way tunnel will be approximately 2.83 km (9,300 ft) long, starting in Seattle's stadium district, passing beneath downtown business and historic districts, and concluding in the shadow of the City's iconic Space Needle (see Figure 1).

To accomplish this engineering feat, the Washington State Department of Transportation (WSDOT) decided to use a Design-Build contracting approach. WSDOT has a solid track record delivering projects using Design-Build and believed this project, with all of its challenges and potential risks, would be more successful with Design-Build than traditional Design-Bid-Build. In keeping with this approach, quality control and quality assurance (QA/QC) responsibilities belong to the Design-Builder, with WSDOT maintaining an oversight, or quality verification, role only.

Regarding geotechnical instrumentation, WSDOT carefully weighed the responsibilities and assignment of risk to decide how the instrumentation specialist should be contracted. Two approaches common in the industry and considered for this project were (1) the Contractor provides the instrumentation specialist, with bid items and prequalification requirements, and (2) the Owner provides the instrumentation specialist. Ultimately WSDOT settled on the contractor-provided option, in keeping with the Design-Builders's QA/QC responsibility, and that the Design-Builder has an economic incentive (based on the structure of the contract) to produce good data and optimize tunnel boring machine (TBM) operation (Dunnicliff, 1993).

Notwithstanding the aforementioned, WSDOT still pursued a relatively prescriptive approach in the contract, or Request for Proposal (RFP). Since it did not select the instrumentation subcontractor, WSDOT wanted to ensure a very robust approach that provided deformation data redundancy. In addition, WSDOT's other principal goal

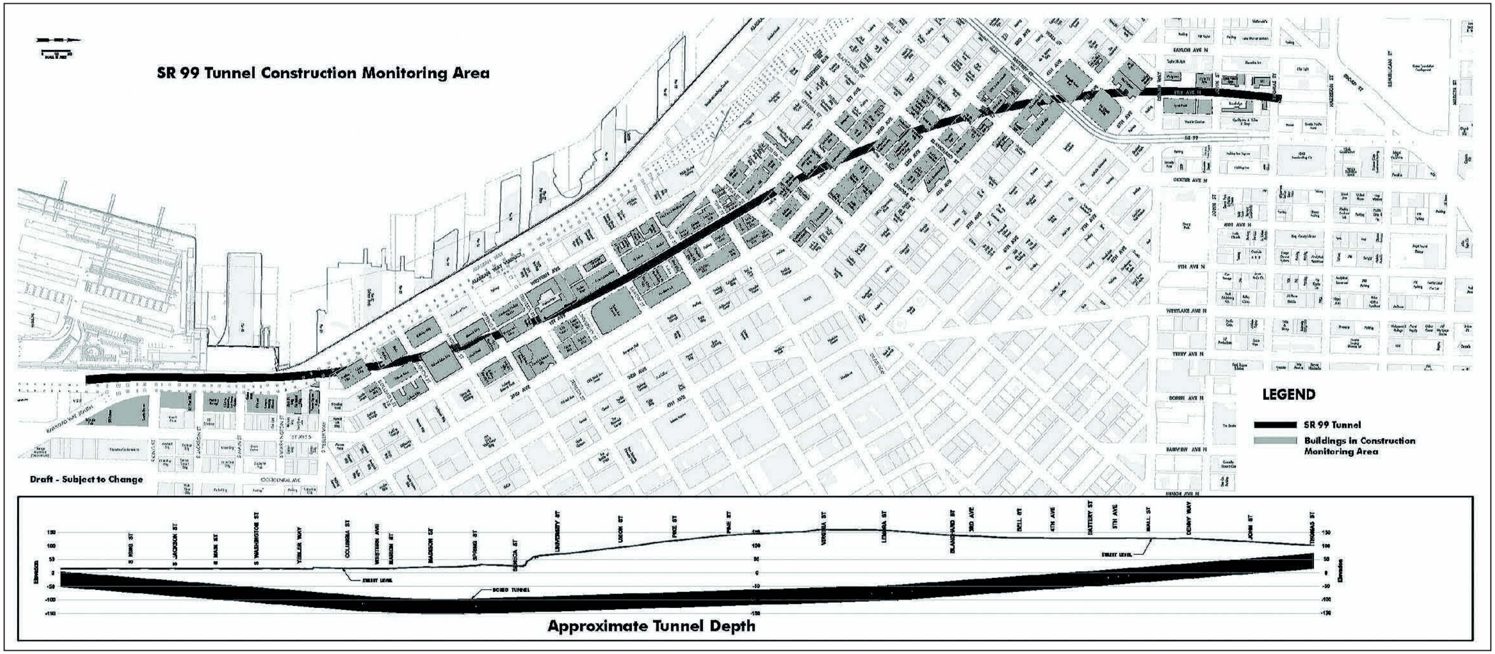

Figure 1. Downtown Seattle—tunnel alignment and instrumented properties

was to level the playing field and minimize bidder speculation on this important aspect of the project.

REQUEST FOR PROPOSAL AND SELECTION

The instrumentation and monitoring concept originally envisioned by WSDOT included a number of ideas absent from the Final RFP, including laser scanning the building interiors, extensometers at 25-foot spacings along the alignment, use of ground penetrating radar, and earth pressure load cells. These requirements were scaled back and the monitoring program became more heavily focused on building behavior and movement, considered the highest risk in the Project's urban environment. Furthermore, based on feedback received during one-on-one meetings with bidders, the Final RFP was modified to allow the Design-Builder to propose alternative instrumentation types or monitoring approaches, with WSDOT's approval. This flexibility provision in the RFP became one of the keys to the instrumentation program.

On December 17, 2010, Seattle Tunnel Partners (STP), a joint venture of Tutor-Perini Corporation and Dragados-USA, was selected by WSDOT in a competitive, best value approach that weighed the Design-Builder's technical approach and qualifications with the proposal price. STP did not select an instrumentation contractor before the RFP date, but chose to finalize this process a few months afterwards. STP issued an instrumentation RFP to potential interested companies. Out of 54 pre-selected companies, 11 proposals were submitted to STP, with nine of them considered responsive proposals. SolData was selected in September 2011.

HOW A FLEXIBLE DESIGN-BUILD CONTRACT ALLOWS EQUAL OR BETTER SOLUTIONS

For the Alaskan Way bored tunnel, as for many tunneling projects, the instrumentation program serves several principal purposes. Monitoring of deep subsurface behavior (e.g., extensometers and inclinometers) provides early detection of possible ground deformation trends and may be used to modify tunnel boring machine operations. Secondary monitoring points (e.g., surface and structure-mounted points) measure actual building and utility movements, important for on-going risk assessment and post-construction claims. And finally, the recorded ground measurements provide some level of calibration to the models used to predict the extent and magnitude of settlement, and thus advance our understanding of the current technology and tools for managing tunneling-induced deformation.

To achieve these goals and more, WSDOT's technical specifications provided a robust approach to instrumentation, while still providing flexibility to the selected instrumentation team to pursue innovations. Given this flexibility, SolData immediately began looking for ways to improve upon the technical requirements. Five examples of these improvements are as follows.

Monitoring Surface and Subsurface Movement

One of the original concepts included in WSDOT's Final Conformed RFP required two specific types of instruments to monitor ground behavior:

- Multiple Position Borehole Extensometers (MPBX): Over 100 automated, with three to five anchors per unit equally spaced along the centerline of the tunnel alignment at 30.5 m (100 ft) apart, with a tighter spacing of 15.2 m (50 ft) at the start of the excavation
- Near Surface Settlement Points (NSSP): Over 350, at 7.62 m (25 ft) spacings between MPBX's and in streets perpendicular to the alignment

Figure 2. AMTS on building roof

While the MPBX provides continuous information about settlement, the NSSP's were planned to be read manually by survey crews at a frequency depending on the actual location of the tunnel excavation, but no more than once a day. Apart from the practical aspect of traffic control and associated safety risks during installation and manual readings, and the later cost of pavement restoration, the use of NSSP's presents two main technical problems:

- Time-delay between the physical settlement and the notification to the construction monitoring team, with a negative impact on the efficiency of potential corrective measures, and
- Limited coverage of surface settlement, with information only at the NSSP-specific locations.

On this premise, and given the flexibility offered in the RFP, SolData sought approval of the reflectorless capabilities of Automated Motorized Total Stations (AMTS), shown in Figure 2. Automatic real time non-intrusive techniques are becoming the industry standard for several reasons. First, technological progress has made them economically viable in comparison with manually handled equipment. Secondly, survey information must be available as soon as possible so that the decision making process can be reduced to a minimum. Finally, the impact on public activities (noise, traffic disruption) has to be minimal and authorizations are always more difficult to obtain (see Third Parties section). AMTS technology has been used in Europe with proven results on recent tunneling projects: Barcelona L9, Amsterdam North-South Metro Line, Toulon South Tunnel Highway, and CrossRail. SolData's application of this technique (self-titled Centaur) was proposed to replace most of the NSSP's on the project with Reflectorless Surface Monitoring Points (Tamagnan and Beth, 2011).

Centaur is based on the evolution of the capabilities of the AMTS distance meter, in terms of laser power and signal repeatability, and also on the recent developments of data processing software. Similar to conventional AMTS use for real time monitoring, Centaur uses physical reference prisms to correct the position of the AMTS before any measurement. However, unlike conventional AMTS, there is no "physical monitoring point," so the system uses the surface properties to reflect the laser signal. Since there is no physical target (only "virtual" points) that can be automatically detected, this technique is limited to monitoring movements perpendicular to a surface (i.e., settlement). For the Alaskan Way bored tunnel, and more generally for a majority of tunnel projects, the critical parameter to control during the excavation is settlement.

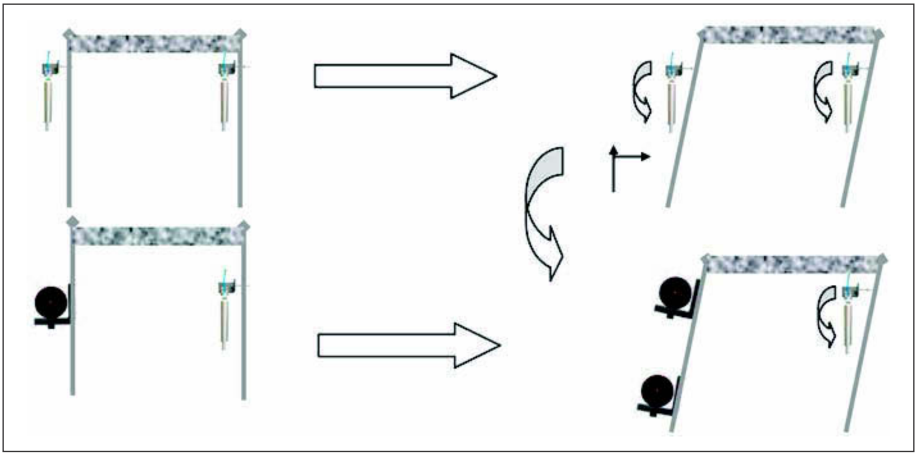

Figure 3. Tiltmeter configuration—RFP versus SolData proposal

The same AMTS hardware equipment can be used for both standard and reflectorless use of the instruments, so the additional cost of using this technique is minimal and only three additional AMTS (36 originally planned) were installed to cover the tunnel alignment.

Monitoring Viaduct

The RFP called for the monitoring of cracks larger than 6 mm (0.25 in) on existing structures using manually-read grid crack gauges. However, not all the cracks are located on equally sensitive structural elements, nor do they always afford reasonable access. While cracks on buildings are oftentimes cosmetic when located on non-structural elements, cracks on bridges and other civil structures are often visible evidence of problems related to structural integrity. SolData proposed to install automatic crack gauges on existing cracks located on the Alaskan Way Viaduct, knowing that the structure was extensively damaged during a 2001 Earthquake (WSDOT, 2002). The use of automatic crack gauges allows:

- a complete understanding of the changes in the cracks due to external effects with continuous readings.
- a faster reactivity with an hourly reporting frequency that could be adapted to the construction activities.
- an improved safety management with no necessity to access the instrument after installation (most of the cracks being located under the deck or on bents).

The RFP documents called for tiltmeters to be installed on every column of the viaduct in the vicinity of the launch pit and the tunnel alignment. SolData proposed replacing one of the two tiltmeters with the use of two offset prisms read with an AMTS (see Figure 3). While a precise tilt measurement is still provided by one tiltmeter on each bent (we could assume that two columns attached to the same bent would tilt equally), the pair of vertically offset targets provide both tilt and 3D movement information at each of the target locations.

Monitoring Tunnel Lining

Monitoring the interior of the tunnel can be challenging. One of the most common devices to measure the deformation inside a tunnel is a tape extensometer. The tape extensometer measures the length of convergence chords between two diametrically opposed points. This instrument was required in WSDOT's RFP. However, given the tunnel internal diameter of 15.8 m (52 ft) and the length of the TBM conveyor, this would have made the measurements difficult to collect. The measurements could only have been taken once the whole machine and trailing gear had left the space free to take the diametrical measurements. The first approach proposed by SolData was to install strain gauges and load cells in the precast lining elements at key sections and complete the tunnel lining deformation measurement by running several scans of the whole alignment. While the first part of this approach was discarded the second was not. Four complete laser scans of the tunnel's inside geometry will be performed after completion of 25%, 50%, 75%, and 100% of the tunnel construction. While this is closer to an as-built survey, the overlap of the scanning will provide information about tunnel lining deformation during the course of the project, allowing remediation actions to be taken before the commissioning of the tunnel. The millions of points gathered by the scanner (1cm × 1cm grid) give an almost comprehensive surface monitoring of the whole tunnel profile, impossible to achieve with a traditional scattered tape extensometer monitoring program.

Monitoring Groundwater

The specifications required a fully automated MPBX with three to five anchors every 30.5 m (100 ft). There were no requirements for additional piezometers along the alignment, but the Project team believed changes in water level or water pressure caused by the tunnel excavation can often have a direct influence on the surface settlement profile. Drilling in downtown Seattle is a challenge by itself, so a solution that could minimize traffic disruptions was welcomed. SolData, STP and WSDOT decided to complete the design of each MPBX along the alignment with the installation of a vibrating wire piezometer (the same sensor technology as the MPBX sensor) to measure changes in water pressure. The cost of the installation is minimized as most of the data logging equipment can be optimized to read both the MPBX and the piezometer sensors, and, more importantly, no additional drilling is required.

Beyond the RFP: Monitoring from Space

Satellite radar interferometry is another innovative, state-of-the-art technique used on this project to detect settlement caused by tunnel excavation. The Interferometry Synthetic Aperture Radar (InSAR) technique has been used for years to detect and monitor large scale natural hazards (earthquake, volcanoes and landslides) and in the mapping industry to build Digital Elevation Models. It is only very recently that it is being used to measure settlement over extended urban areas during tunnel projects. This particular application of the InSAR technology compares the difference in phases between two satellite images so that it is possible to retrieve a precise map of settlement over large areas. The technique uses existing structures or fixed elements as "natural reflectors," with each of these reflectors then being used as a settlement point. With technological advances, the typical density of reflectors is now around 10,000 points per square kilometer, with one image capturing an area about 5 square kilometers. The accuracy of the technique is better than 3 mm (0.125 in). Images are taken approximately every 10 days.

This technique is not aimed to replace real time monitoring techniques, but is used as a tool to detect settlement over a larger areas than the typical anticipated zone of influence and does not require any field installations. If available, the satellite images can be processed and allowed to "monitor the past" and detect any movement prior to the actual start of the monitoring readings.

WSDOT and STP also consider the radar interferometry a necessary tool to assist the project team addressing potential third party claims. Despite the extensive instrumentation program described herein, both WSDOT and STP believe that some risk to damage claims from property owners outside of the predicted settlement zone (non-highlighted buildings in Figure 1) is possible. Adjudication of these claims would be time consuming without any pre-construction data. The selection of radar interferometry provides one solution to addressing possible false claims. Its usage on this project only adds to this fledgling technology, one that is expected to see more wide-spread use in the years to come.

THIRD PARTIES

The project scope and location requires extensive third party coordination. Third parties are defined herein as people or groups that are not party to the bored tunnel design-build contract, but who have an involvement and vested interest in the successful outcome. For this project, third parties include local jurisdictions, such as historic district preservation boards and City of Seattle utilities, as well as private property owners above and adjacent to the alignment. With some of the third party approvals taking months to acquire, WSDOT and STP had to modify the project schedule to ensure timely instrumentation installations.

Preservation boards representing the historic districts in Seattle (and appointed by the mayor), were consulted on instrument placement. This included the historic Pioneer Square district and the famous Pike Place Market district. These neighborhoods contain stone and brick buildings constructed in the late 1800s and early 1900s, complete with an occasional cobble-stone street. The Board approvals also required WSDOT to consult with a historic preservation architect who could advise on repair of brick, concrete, or stucco facades after the instrumentation was removed.

For property owners whose buildings are directly above the tunnel alignment, the requirement for WSDOT-acquired subsurface property sometimes drove additional instruments, with sometimes limited effectiveness, for monitoring and the property owner's desire for oversight. For property owners that only bordered the alignment (i.e., subsurface acquisitions were not required), the project team still had to obtain a "Right of Entry" to access and install the instruments (typically at no cost) and coordinate electrical power requirements. Once installed, private and public property owners were universal in their interest for continuous monitoring data; this information transfer becoming yet another challenge for the instrumentation and construction monitoring team.

City-issued street use permits were also required for installing instruments, as much of the tunnel alignment is within City right of way. These permits were necessary to address vehicle and pedestrian traffic disruptions due to sidewalk or traffic lane closures. On top of this, SolData had to juggle the impacts of a Holiday Moratorium (no street disruption from late November to January 1) in certain shopping and business districts in Seattle.

Utility owners, private- and publicly-owned, required extensive coordination on instrumentation placement and application. While many of the communications and electrical utilities are relatively settlement tolerant, the waterlines along the alignment, some as large as 30 inches in diameter and 100 years old, require monitoring and contingency planning in the event of a leak or sudden failure. SolData's Centaur system, direct utility monitoring points, and STP's automated leak detection system all help to quickly inform the project team of any movement.

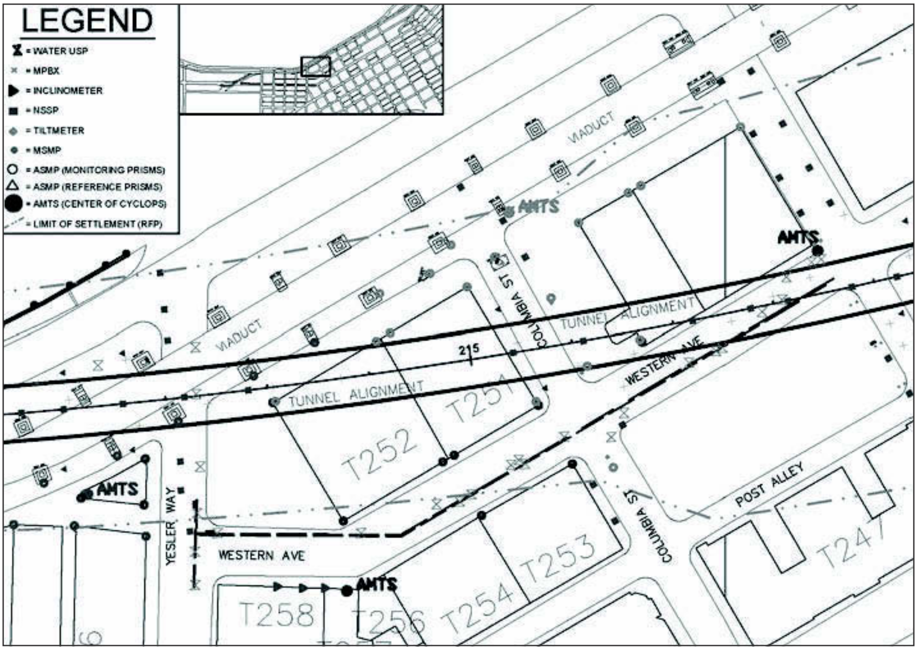

Figure 4. Final instrumentation approach—tunnel alignment from Yesler to Marion Streets

DATA MANAGEMENT

While the tunnel alignment is relatively short, the density of instrumentation makes it difficult to have a comprehensive view of the whole monitoring program. In addition to the design constraints, the timeline for acceptance of installation in the historical areas on landmark buildings (see Third Parties Section) required dividing the monitoring into six different areas or sections. Two sections deal specifically with the portals (Launch pit, or South Portal, and the North Portal), while the remaining four sections divide the tunneling drive (Figure 4). This division was also dictated by the RFP requirement to take instrument readings at least six months (baseline period) before tunneling. During the monitoring period, especially on a tunnel project of this size, decisions have to be made in a timely manner to allow the tunneling excavation to progress safely in accordance with the construction schedule. As sometimes going too "fast" could be the enemy of going "safe," all the project participants need to be involved in the decision making process.

During tunneling, a designated Construction Monitoring Task Force (CMTF) meets daily to review the current monitoring data. The CMTF involves members from WSDOT, STP and SolData, as well as other key affected third parties depending on the location of the excavation front. Even if "nothing" is happening, it is important for all of the affected parties to be continuously informed on the progress of the project with full transparency.

The extensive amount of data collected and ultimately shared with the various parties cannot be fulfilled with paper reports. SolData is using a comprehensive data management system and monitoring database (GEOSCOPE software) to collect, process, manage and display this information in "near real time." The software requires specific skills and continuous maintenance to avoid any system crash, contiguous protection as a significant amount of data flow streams continuously through the internet, and a dedicated server and associated back up to store all of the monitoring data. The monitoring data is available at any time, which can be counterproductive if not managed carefully. While the database capabilities can accommodate any project size, an individual's capability to process the data is much more limited. The best database is not the largest one, but one in which the relevant information reaches the interested decision maker as quickly as possible. One of the challenges with a large and complex monitoring program is to precisely answer the questions: What parameter(s) is critical? Where? When? And, for whom? That is the reason why the CMTF reviews the current alert levels and decides on the actions to be taken to manage all the alerts.

CONCLUSION

As of June 2013, the Alaskan Way Bored Tunnel project is about to commence mining from the south launch pit. All geotechnical instrumentation is in place, with some of the baseline readings extending almost a year in advance of tunneling. Hitachi Zosen's record-size tunnel boring machine is poised to start work and all parties are anxious for this next phase of the project.

The instrumentation approach, first crafted by WSDOT, and subsequently modified and improved upon by Seattle Tunnel Partners and SolData, is further defining the state-of-the-practice for tunneling projects in urban environments. While there have been many challenges to plan, develop, and fully implement the program, the information collected will be invaluable for many reasons. First and foremost, SolData's instrumentation data and Hitachi Zosen's TBM key parameters (grout pressures, face pressures, cutter wear, muck volume, etc.) will be integrated into a single source database and made available to tunneling operators and heading engineers. This is only possible with close cooperation between Hitachi Zosen and SolData so that TBM data is compatible with the existing monitoring data format and uploaded in real time to the central database at predefined time stamps through an internet connection. This data integration provides Seattle Tunnel Partners increased quality control and more timely validation of tunneling parameters and operational performance through the use of both geotechnical instrumentation and monitoring data and TBM operational data.

Owners considering Design-Build should be prepared to offer flexibility in those areas where technology can change dramatically. Geotechnical instrumentation is one such area. As the Alaskan Way Bored Tunnel project has shown, technology can sometimes evolve much more quickly than the contract procurement, award, and project start-up phases. Providing flexibility allows for full incorporation of the latest technologies and improves upon the state-of-the-practice. A qualified instrumentation team, pro-active contractor, and flexible owner are absolutely key to a successful tunneling project.

ACKNOWLEDGMENTS

The bored tunnel project is part of the larger Alaskan Way Viaduct Replacement Program. The authors wish to acknowledge WSDOT's team, lead by Linea Laird, Program Administrator; and Matt Preedy, Deputy Program Administrator. Seattle Tunnel Partners is lead by Project Managers Chris Dixon and Greg Hauser, of Tutor-Perini Corp. and Dragados-USA, respectively. And finally, technical (and moral) support is provided by Tony Stirbys, STP's Geotechnical Manager.

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SUMMARY AND LESSONS LEARNED FROM NEW YORK CITY TUNNELING INSTRUMENTATION

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ABSTRACT

Over the last five years New York City has undergone a boom in tunnel planning, design and construction. Three large tunnel construction projects for subways and regional rail systems are currently underway, with several smaller tunnel projects recently completed. This paper will present a summary of the geotechnical instrumentation programs on the three large projects, namely East Side Access, the No. 7 Line Extension and Second Avenue Subway.

This paper intends to provide insight into the unique specification and contractual practices for the geotechnical instrumentation requirements of each project, with discussions and opinions regarding the various types of instruments, monitoring requirements and Web based data management systems employed. In addition, lessons learned for each program will be discussed, as well as stakeholder's impression of performance of the instrumentation programs. Based on these project experiences, recommendations for geotechnical instrumentation procurement, specifications, instruments, and data management systems are provided for upcoming projects, from the perspective of both the designer and the installer.

INTRODUCTION

Several major tunneling and underground construction projects have taken place in New York City since approximately 2005. These projects were undertaken to address the long pent-up demand for additional commuter rail and subway service within Manhattan and the surrounding boroughs.

The three most notable major projects are East Side Access, the No. 7 Line Extension, and Second Avenue Subway. The New York City area's biggest transportation agency, the Metropolitan Transportation Agency (MTA) is the owner of these three projects. The density of existing infrastructure, the number of stakeholders and access limitations make the installation and monitoring of instruments for these tunnel construction projects particularly challenging.

Each of these projects handled the procurement, monitoring, and installation of instrumentation using unique specification and contractual practices.

Second Avenue Subway

The Second Avenue Subway project is planned along Second Avenue from 125th Street to the Financial District in Lower Manhattan. The project will include an 8.5-mile (13.7 km) long two-track line and 16 stations with average length of 902 feet (275 m) as shown in Figure 1. The tunnels and stations will be up to approximately 100 feet (30 m) below street level. The planning of the subway line is complicated by existing subway tunnels, Amtrak tunnels and the Queens-Midtown Tunnel.

Under the current plan; the project will be built in four phases, with Phase One currently underway. Phase One includes tunnels from 105th Street to 63rd Street, with new stations along Second Avenue at 96th, 86th and 72nd Streets and new connections to

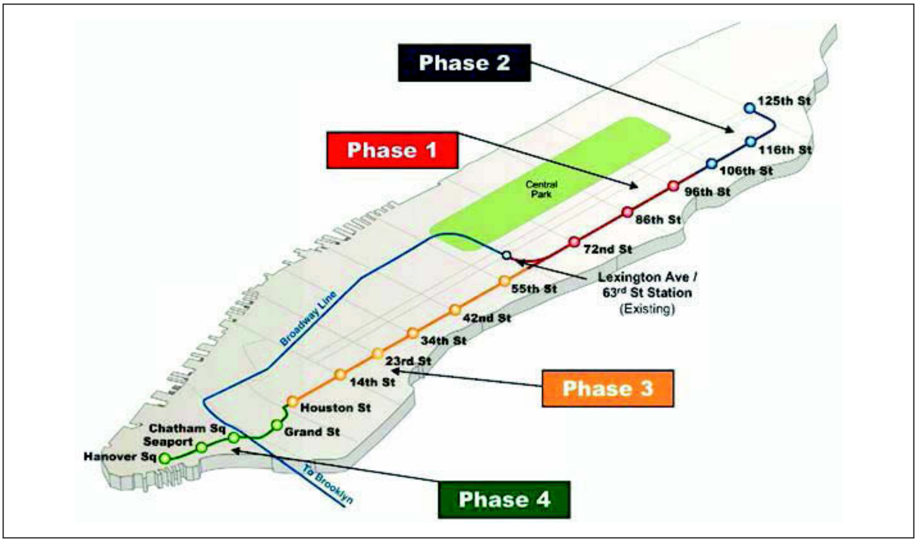

Figure 1. Overall plan view of the Second Avenue project

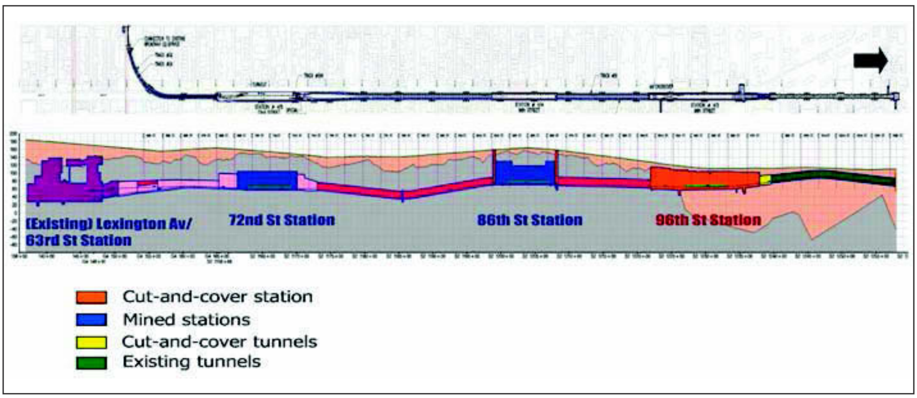

Figure 2. Profile view of the Second Avenue project, Phase 1 alignment

an existing subway line beneath Lexington Ave and 63rd Street. Tunneling between 63rd and 92nd Streets will be done by a TBM, launched from an 814 foot long (248 m) by 75 foot wide (23 m) open cut excavation. This excavated area will ultimately become part of the 96th Street Station.

The geology changes along the subway's length, passing through rock and soft ground, consisting of sands, silts, and clays over Manhattan Schist bedrock, with faults and shear zones as well as fractured zones in the bedrock (Figure 2).

East Side Access

The East Side Access project will connect the Long Island Railroad's (LIRR) Main and Port Washington lines to a new terminal beneath Grand Central Terminal in Manhattan. The East Side Access Project includes two 22-foot (6.7 m) diameter tunnels over 100 feet (30 m) beneath street level in Manhattan and four 22-foot (6.7 m) diameter tunnels 140 feet (43 m) below street level in Queens as shown in Figure 3.

The Manhattan subsurface conditions typically consist of hard rock (Manhattan Schist). Tunnel boring machines (TBMs) have drilled through over 30,000 feet (9,144 m) of rock to create the running tunnels. For the large caverns comprising the LIRR facility beneath Grand Central Terminal, conventional mining techniques (drill and blast and mechanical excavation by road headers) were used.

Grand Central Terminal is the largest train station in the world by number of platforms. The terminal covers an area of 48 acres (195,000 m^2). It has 44 platforms and 67 tracks with 41 tracks on the upper level and 26 on the lower. When the East Side Access project gets completed, Grand Central will offer a total of 75 tracks and 48 platforms (Figure 4).

The one-mile tunnel in Manhattan extends from the end of the previously constructed river tunnels west and southward from 63rd Street and Second Avenue to Park Avenue around 57th Street, continuing beneath Park Avenue past Grand Central Terminal to 37th Street and Park Avenue.

In Queens, the western portion of the tunnels will be excavated using the cut-and-cover method, but the remaining tunneling, such as tunneling under Sunnyside Yard, required minimal surface impact to avoid disrupting LIRR and Amtrak rail operations. The subsurface conditions generally consist of soft ground with a high water table.

In Queens, construction is ongoing for the open cut portion of the project, which will extend the tracks under Northern Boulevard into the Sunnyside Yard and create

Figure 3. Overall plan view of the East Side Access project

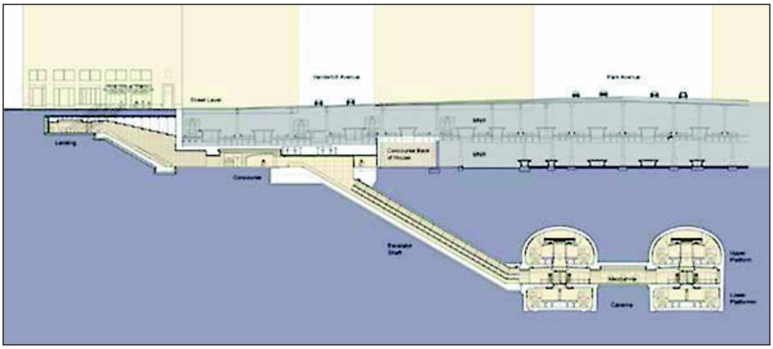

Figure 4. Cross section through the East Side Access project at Grand Central Station

an area that serves as both the launch chamber for the TBMs and an interlocking, emergency exit and vent facility.

No. 7 Line Extension

The No. 7 Line Extension project will extend the existing No. 7 Subway Line west along 41st Street from Times Square to Eleventh Avenue, where it will turn south and run under Eleventh Avenue to 24th Street as shown in Figure 5. Approximately 7,000 feet (2,100 m) of twin-bore tunnels have been driven, primarily in rock, with several underground caverns and shafts for stations, ventilation and service facilities.

The TBMs excavated 7,100 foot (2,200 m) long tunnels northward along Eleventh Avenue to connect with the existing tail tracks of the No. 7 Line beneath 41st Street at Eighth Avenue. The first 300 feet (91 m) of tunneling was complicated by a section of soft ground between 27th and 28th Streets that required ground freezing to reinforce the ground, allowing the machines to pass through the soft ground as if it were hard rock.

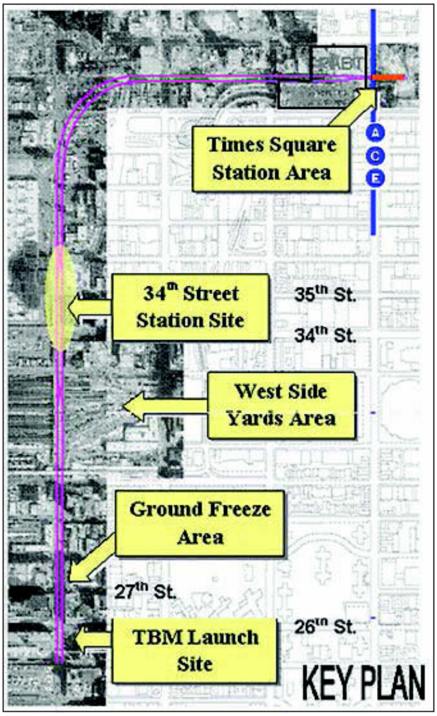

Figure 5. Overall plan view of the No. 7 Line project

Extensive excavation and tunneling were performed beneath the Port Authority Bus Terminal and the Times Square subway station on MTA's Eighth Avenue subway line. The work under these facilities consisted of cut and cover construction (including extensive rock blasting) beneath the lower level of the active bus terminal to create a receiving chamber for the TBMs, demolition of a portion of the unused lower level of the subway station, underpinning of the station, excavation beneath the station, and lowering the grade along several hundred feet of the existing No. 7 Line tunnels. Figure 6 shows a cross section of the TBM launch location.

INSTRUMENTATION PROCUREMENT

Much has been written over the last 20 years regarding the procurement practices for geotechnical instrumentation by many of the current authorities in the field, including papers by Dunnicliff, Powderham, and Koutsoftas. Each of the three projects summarized above used a slightly different delivery method for instrumentation procurement, installation and monitoring.

Second Avenue Subway

Construction procurement for the Second Avenue Subway was a conventional (according to the United States' practices) design, bid, build format and thus the geotechnical instrumentation generally followed this suit. At the time of this writing,, eight construction contracts were released, each requiring separate instrumentation programs. In each of the projects the instrumentation was specified in the contract plans and specifications as the responsibility of the Prime Contractor to procure, install, monitor, analyze

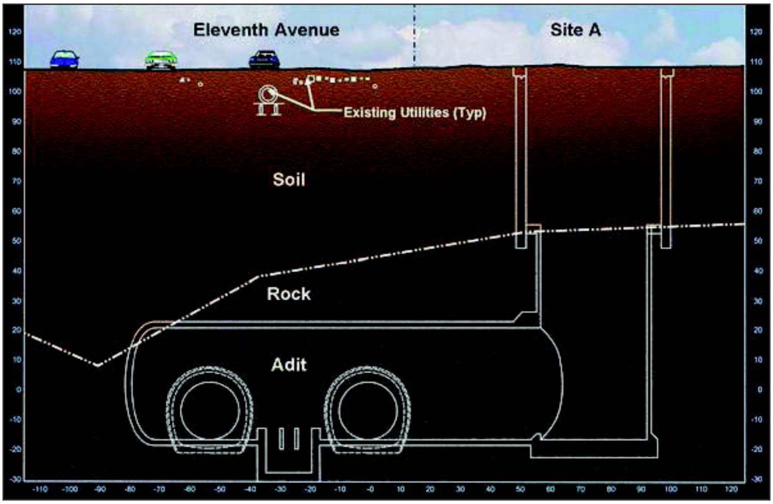

Figure 6. Section on the No. 7 Line Project at the TBM launch location

and report on all instrumentation. In general, the procurement of the contracts spanned approximately five years and specifications were generally not modified based on lessons learned.

The instrumentation section of the specifications required that all instrumentation be performed by a group of Instrumentation Specialists either directly employed by the Prime Contractor or subcontracted. The instrumentation specialists included individuals to provide final design of the AMTS (Automated Motorized Total Stations) systems along with an Instrumentation Engineer with the qualifications of a Professional Engineer in the State of New York or Professional Geologist with six years of relevant experience, a Surveyor, New York State Licensed with three years of relevant experience, and an Instrumentation Supervisor. An AMTS manufacturer's representative was also required during certain times during the installation process.

In all cases on the Second Avenue projects the contractors hired Instrumentation Specialty firms (typically firms that also perform geotechnical engineering). Out of eight contracts to date the projects have been performed by four different instrumentation firms, reflecting the general competitive business environment for this type of work.

The process of specialty instrumentation selection by contractors in New York City has been discussed by others (Volterra) and has generally evolved to a process of several specialty instrumentation firms, some properly qualified and others not, providing lump sum pricing for the entire instrumentation program. This lump sum pricing is typically provided initially by the specialty instrumentation firms providing unsolicited bids to all Prime Contractors prior to the bid. Following the selection of the prime Contractor by the Owner, the Contractor will typically re-scope the instrumentation work, looking at the cost of the individual instrumentation or parts of the instrumentation process that the Contractors may self-perform at a lesser cost. Such items have included the Maintenance of Traffic (MOT), excavation of test pits at the drilled instrumentation location to clear utilities, as-built surveys of installed instruments and/or the provision of lifting equipment to access building instruments.

Following the Contractor's internal process, a revised scope will typically be released and final bids are received and the best value price is selected. The best value may be the lowest price or, depending on the Owners requirements, may be a firm with Disadvantaged or Minority Owned status. Unfortunately this process has not typically been based on a quality or true value. Each of the contracts requires the extensive transfer of the instrumentation upon the completion of individual contracts to the following contracts. Unfortunately, the contract specifications and bid documents could not provide a clear understanding on what instruments were to be left in place, which may have provided a competitive disadvantage to non-incumbent firms.

Like the majority of rail tunneling projects in the United States, the new subway project was designed to underlie the city streets, thus avoiding private property where possible. The majority of the northbound and southbound avenues within Manhattan span a width of 66 feet (20 m). The 2nd Avenue Subway is a long linear project with a concentration of instrumentation on the buildings flanking the subway tunnels with settlement monitoring of the street and some monitoring within large rock caverns.

A partial list of the more interesting or technical instruments on the six contracts included:

- Automated Motorized Total Stations (AMTS) with reflective monitoring prisms for monitoring of movement of buildings on either side of the streets adjacent to the tunnel alignment and access shafts (Figure 7).
- Tilt Beams monitoring movement of buildings adjacent to the tunnel alignment and access shafts.
- Tilt Meters monitoring of movement of buildings adjacent to the tunnel alignment and access shafts.
- Vertical Multi Point Borehole Extensometers to monitor soil and rock above the inverts of running tunnels and caverns.
- Horizontal Multi Point Borehole Extensometers within the caverns.
- Inclinometers both within support of excavation elements (Slurry Walls) and outside of excavations used to excavate vertical access shafts.

Data logging was required on all piezometers, strain gages, load cells, tilt sensors, vertical and horizontal multipoint extensometers, and AMTS systems. Inclinometers were not required to be automated but based on access and specific contract requests several were automated and datalogged. The project specifications required a very strict monitoring frequency, directly tied into the construction schedule.

The early Second Avenue Subway contracts reportedly had the most instrumentation issues to date due to a variety of issues, such as location, lack of understanding of building thermal movement and construction methodologies. For example, these initial contracts were located on the northern portion of the alignment in an area of relatively thick overburden soils above rock. The buildings constructed along the tunnel alignment and at the station/launch box locations were generally turn-of-the-20th century construction, with two- to six-story relatively narrow masonry bearing buildings. These

Figure 7. AMTS on Second Avenue project with power from light pole

Figure 8. Horizontal extensometer on the East Side Access project

buildings were typically founded a single story below grade on shallow foundations bearing on historic fill and/or overburden. Baseline monitoring of these structures was limited based on the project schedule which later caused many questions regarding the accuracy of AMTS monitoring following development of cracks within the buildings.

East Side Access

It can be said that the procurement for the instrument on the East Side Access Project was the most unconventional, but arguably, the best in terms of typical procurement of instrumentation projects in the United States. To date procurement and construction on the East Side Access project has spanned more than 10 years and is currently ongoing.

Very early in the design stages of the project several prominent personnel, including John Dunnicliff, reviewed and influenced the designer's procurement process for the instrumentation program.

These recommendations and influences drove the instrumentation program to be procured such that each individual construction contracts retained specialty instrumentation personnel for the installation and automation of instrumentation with monitoring being undertaken by one or more of the Owners representatives. On one section of soft ground tunneling in Queens that was influenced by more than one contractor, the project wide Construction Management firm solicited and procured the instrumentation installation directly.

The specific instruments required on the project spanned the gamut of the instruments commercially available today. A partial list of the more interesting or technical instruments included:

- Automated Motorized Total Stations (AMTS) with reflective monitoring prisms for monitoring of movement of buildings, retaining walls, tunnels and surface railway structures.
- Tilt Beams monitoring of movement of buildings adjacent to the tunnel alignment and access shafts.
- Tilt Meters monitoring of movement of buildings adjacent to the tunnel alignment and access shafts.
- Vertical Multi Point Borehole Extensometers to monitor soil and rock above the inverts of running tunnels and caverns.

- Liquid Level Systems for monitoring of vertical movement of existing structural columns above the proposed tunnel alignment.
- Horizontal Multi Point Borehole Extensometers within the caverns and shafts (Figure 8).
- Load Cells on support of excavation struts and tie backs.
- Vertical Multi Point Borehole Extensometers above the caverns and running tunnels.
- Inclinometers both within support of excavation elements (Slurry Walls), outside excavations and adjacent to rock caverns.
- Resistance-type strain gages for the monitoring of blasting-related strain on adjacent structures.

Data logging was required on all piezometers, strain gages, load cells, tilt sensors, vertical and horizontal multipoint extensometers, and liquid level systems as well as the AMTS systems. Inclinometers were not required to be automated on all contracts but based on access and on specific contract requests several were automated and data logged.

The Contractors and their Instrumentation Contractors were responsible for the installation, baseline collection and datalogging of all installed instrumentation. Where the East Side Access project differs from other tunneling projects in New York City is that the collection of the manual instrumentation readings, including survey monitoring and vibration data, was the responsibility of the Construction Manager. Following the collection of this data the Construction Manager responsibility included posting, analysis and issuing alerts, if needed, via a web based data management system for all instrument data available to the Construction Manager, Prime Contractor and the Design Team. All conventional survey data was obtained from subcontracted union survey crews.

For the first several years of this work starting in 2003, prior to the release of the running tunnels contract, the Construction Manager used its own internal staff to collect manual readings and analyze the data, placing it into an internal database system with email updates sent to the various Contractors and Designers.

Several years later, after the resolution of some overall project funding issues, the Construction Manager retained an Instrumentation Specialty firm through a competitive bidding process to collect and manage all instrumentation-based data on the project. By this time the web based data management system had been developed to the point that data could be viewed by all parties on a near real time basis.

The East Side Access project utilized several instrumentation types to a larger scale than had previously been used. This was arguably the first use of an AMTS system for monitoring of the internal elements of an active subway tunnel in the United States.

These systems included an extensive network of resistant type strain gages for the measurement of strain as a result of close-in blasting. These strain gages were used extensively on vertical support columns with the Grand Central Station.

The uses of liquid levels systems were also advanced. Within two existing subway tunnels which run over and perpendicular to the running tunnel alignment, a system of pressure transducer-based liquid levels systems was undertaken with limited success.

More notable was the extensive liquid level system utilized within Grand Central Station as shown in Figure 9. This system which consisted of a Geokon's Precision Settlement Monitoring System which is an open channel system, constructed with stainless steel components and filled with a water/antifreeze solution. Each sensor consisted of a cylindrical weight submerged within the liquid and suspended by a vibrating wire force transducer. As the liquid level rises and falls, in this case when a

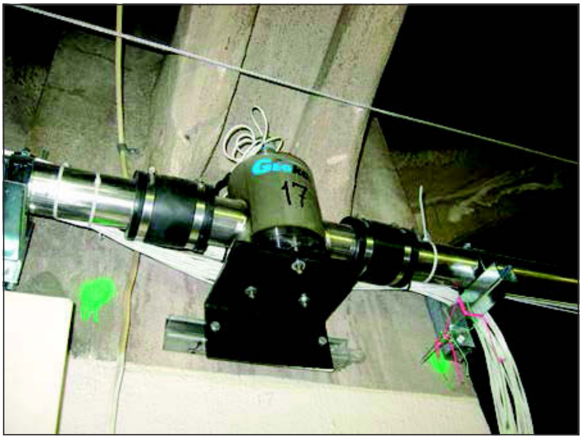

Figure 9. Liquid level systems within Grand Central Station on the East Side Access project

sensor mounted on a vertical column moves downward, the bouncy force of the weight will change, thus altering the tension on the vibrating wire and resident frequency. In total more than 200 sensors were utilized, anchored to concrete covered vertical steel beams which support the station and overlying building structure. The sensors were connected using 3-inch (7.6 cm) diameter stainless steel tubing with datum sensors outside the expected area of influence. The entire system was datalogged and set up for automated monitoring.

It is interesting to note that earlier instrumentation designs for the station level at Grand Central Station included an extensive network of AMTS systems, but these were removed in later contracts due to concerns regarding high temperature within the active train station.

An AMTS system was first utilized on the project in 2003 as the result of a value engineering proposal presented by the Prime Contractor and Instrumentation Specialist to the Owner. The original design called for daily survey reading of more than 50 points within an active, five track subway tunnels which ran adjacent to a section of supported excavation in Queens. Although the cost of the daily survey was of concern, the greater cost was the Prime Contractor and Owner need to provide safety flagging as required by the New York City Subway system, which runs on a 24 hour, 7 day per week basis. The resulting AMTS system consisted of robotic total stations and reflective mini prism survey targets, which were installed on tunnel walls. The robotic total stations were monitoring the X, Y and Z coordinates of the survey targets. One Leica total station was installed in each of four tunnels.

No. 7 Line Extension

The No. 7 Line Extension project is interesting not only in how the work was procured and undertaken, but also in how it differed from similar contracts also in planning and underway at the same time. This timing also impacted the award of the Prime Contractor.

Originally sent out for bidding in 2007 to the Prime Contractors, the MTA only received a single responsive bid in 2008 after more than 30 addenda to the bidding package. The overall price for the single project was just over one billion US dollars in 2008.

Although the instrumentation program was designed by the same firm that designed the East Side Access project, the responsibilities assigned to the instrumentation contractors were slightly different.

The alignment of the project was in a less densely populated part of New York City, and generally in hard rock, thus fewer buildings were adjacent to the project. Instead, the construction of a large station cavern and the location of the adjacent heavy rail tunnels, historic buildings, overlying viaduct and cut and cover construction (including rock blasting) within the lower level of the active bus terminal to create a receiving chamber for the tunnel boring machines called for several different instrumentation approaches.

The Owner considered the cavern excavation design/build, with other portions of the project the more conventional design/bid/build. The design of the instrumentation was a mix of fully defined instruments and monitoring points and "directive notes." These directive notes requiring monitoring for vertical and horizontal movement as well as tilt on selected buildings but without specification of the actual type, number or location of instruments to take the readings.

Similar to the Second Avenue Subway project, the Prime Contractor's instrumentation Specialist was responsible for all data collection, monitoring, interpretations and web based data management. The web based system was not required to provide TBM location information, as has been required on several projects recently, but did require a remote backup web server be used to mirror all data.

The project required the following instruments:

- 23 borehole inclinometers,
- 16 observation wells,
- 81 multi-point borehole extensometers,
- A single nearly-horizontal, 230 foot (70 m) long borehole inclinometer at the TBM receiving chamber under the Port Authority Bus Terminal,
- 10 Automated Motorized Total Stations monitoring hundreds of reflective survey prisms on structures, in Amtrak train tunnels, the Port Authority Bus Terminal, Eighth Avenue Subway Station, and in the existing No. 7 Line tunnels near Times Square,
- Liquid Level Settlement Sensors in the Port Authority Bus Terminal and in the existing No. 7 Line tunnels near Times Square, and
- Strain gauges and load cells monitoring in real time at the Eighth Avenue Subway Station during critical load transfer activities.

Settlement monitoring via manual optical survey of Lincoln Tunnel tubes, columns in Jacob Javits Convention Center and columns in the Eighth Avenue Subway Station was also undertaken.

In areas where directive notes required monitoring of buildings, a combination of AMTS, and tiltmeters were utilized along with manual survey of AMTS prisms (Figure 10). This combination was used to meet the requirement for a tiered monitoring frequency, where the structures within 200 feet (61 m) of active excavation, including access shafts, caverns or running tunnels, required monitoring on a daily basis, seven days a week. Because limited hard wire power was available, most of the AMTS locations were powered using arrays of solar panels with deep cycle batteries. After installation and baseline readings and passing the 200 foot (61 m) zone, monitoring was required on a weekly basis for a period of four weeks and on a monthly basis until the final lining on all underground structures was complete. Given these requirements, the prisms were installed on the required buildings and located both with AMTS installed on a nearby building facade and by manual optical survey to allow for later survey and the preparation of as-built drawings. The tilt meters were battery powered uniaxial units with radio communication allowing for continuous communications via a nearby

Figure 10. Combination prism and tilt meter on No. 7 Line project

data logger with a cellular modem which was also used to control and collect data from the AMTS systems. Using this system the prisms were monitored via AMTS during the critical excavation process on a daily basis and the tilt meter data was used to confirm and/or troubleshoot any thermal variation shown with the AMTS data. One month beyond the completion of active excavation and lack of any significant measured movement, the AMTS was removed from its mounting bracket and moved to another location along the running tunnel alignment. The AMTS data logger was typically left in place to allow for continued data transfer of the tiltmeters. Reflective prisms were read using manual optical survey from the adjacent streets. Where implemented, this system worked very well and was cost effective. In one area where minor building settlement was noted, both the AMTS and manual monitoring was undertaken for more than one year allowing for direct comparisons of the x, y, and z data using both techniques.

Other systems of interest on the project included 81 multi-point extensometers, installed at depths varying between 20 to 120 feet (6 to 37 m) below the ground surface, over the tunnels and caverns (Figure 11). Although not required to be automated, the locations in center lanes of busy New York City streets made the automation of these extensometers the most efficient and safest way to collect the data. Generally these extensometers were widely spaced along the alignment and cellular modems were utilized as the surrounding buildings made radio communication infeasible. Of particular interest was a series of extensometers installed within a shallow tunnel used by buses to access the Port Authority Bus Terminal. The twin TBM run alignment passed longitudinally under this roadway with as little as 15 feet (5 m) of weathered bedrock cover. Ten cellular modems controlled double anchor borehole extensometers installed through this roadway surface to a depth within 3 feet (1 m) of the tunnel crowns. Given the low rock cover, the Owner requested a very short reading/reporting cycle of five minutes. With several modifications to the web-based data management system and frequent battery changes, the five minute frequency was accommodated.

LESSONS LEARNED AS THE INSTRUMENTATION SPECIALIST

Rarely does a city experience the boom in tunneling projects on more than one project within a single decade that New York City has. Through these projects much knowledge has been gained, and many lessons learned.

Despite the almost commonplace nature of geotechnical instrumentation work in large tunnel projects, the industry is plagued with poor instrumentation planning and instrumentation specifications. As a result, asking questions prior to bidding regarding vague or confusing specifications in a formal manner to the procurement agency is very important to completely understand the knowledge of the Designer and the Owner.

Additional recommendations based on lessons learned and the Authors experiences include:

- Understand the Prime Contractor’s union agreements that may impact your work schedule and costs. It is common during the drilling phase of the project that multiple unions will claim work typically done by the union drilling crews.

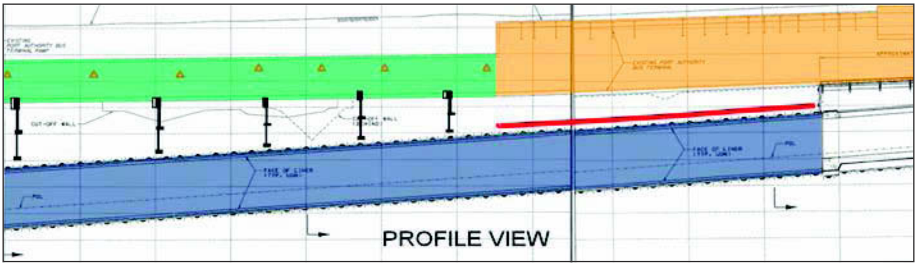

Figure 11. Profile view of tunnels under bus ramp with MPBX's on the No. 7 Line extension, twin TBM runs

- Clearly understand who will be responsible for contacting and making agreements with property owners for the inclusion of instruments mounted within or on buildings or structures.
- Labor hours if the Owner requires the Instrumentation Specialist/Prime Contractor to undertake this task.
- Similarly any interface with 3rd party agencies such as other railroads, subway lines, and or City transportation authorities can be cumbersome and can lead to a loss of schedule and budget.
- The Owners representative in the field, whether it is the Construction Manager or direct Owners representative, will likely have little or no knowledge of instrumentation and especially on how data management must be undertaken.
- Owners and designers may have little understanding for the custom (not off the shelf) nature of most instrumentation systems. A complete automated instrumentation system on a large tunneling project is not something that can be expected to work flawlessly from day one. Concepts of zero drift, data averaging or manual data confirmation are not typically understood by the Owners. Dialogue to this effect with all interested parties early in the project should be undertaken so expectations are clear.

RECOMMENDATIONS TO DESIGNERS AND OWNERS

Based on the author's direct involvement with acting both as the Project Designer and Prime Contractor's Instrumentation Specialist, the following recommendations should be considered for future project development.

- The project Design team should discuss internally and then with the Owner the intent of the instrumentation and the ultimate goals they expect to achieve. Scattering instruments along a tunnel alignment without regard to the building types, subsurface conditions and what problems may occur only adds undue cost to the project.
- The designer should fully understand the instruments specified and their limitations. It is almost commonplace to find project specifications including long obsolete instrument types or incorrect instruments.
- Clearly indicate the reading and reporting requirements of each instrument. Should the instruments be read seven days per week if the construction excavation is underway only five days a week?
- If an instrumentation database management system is required, only specify those commercially available systems that have a demonstrated availability and can handle the types and amounts of data required.

- Carefully consider who and how access will be gained to buildings and other structures during the project implementation. Clearly, it is in the Owner's best interest to self implement negotiation and access agreements to structures that require monitoring, but it is often given to the Prime Contractor who has little political clout or willingness to enter into access agreements.
- Who provides engineering interpretation, analyses and deformation limiting criteria of instrumentation data on these projects is a key consideration and can lead to many meetings and discussions. The author strongly recommends that this task should be part of the Project Designer's responsibility for any existing structure. Who else has studied the impacts of the tunnel on the adjacent structures and knows best why instrumentation was specified for those structures in the first place. The Instrumentation Specialist should be responsible for reviewing their data for errors or thermal change only.
- Clearly define the terms "Continuous" or "Real Time" data collection if they are used.
- Automated data collection and web based data management with automated alarming will cause issues as filtering of erroneous reading is not always possible. Any real time monitoring requirements and expectations should be clearly stated.
- The Project Designer and Owner should discuss and develop plans for reacting to alarms during non-business hours. On one of the projects noted above the Owners Construction Management team who was responsible for overseeing the Instrument was not provided with the proper cell phones or computer access to allow them to see alarms triggered overnight or on weekends!

These are just a few of the issues that should be considered. Regardless of how they have been implemented and procured, these three New York City projects have been a great success and have advanced the state of the art of the instrumentation industry throughout the United States. It will likely be a few decades or more before we see this level of tunneling activity in the area and we can hope that the industry has taken all the lessons learned for improvement in other parts of the country and the world.

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PART 8

Ground Stabilization

Chairs

Paul McDermott
Kenny Construction

Steve Maggipinto
Schiavone Construction Co., LLC

GROUND INVESTIGATION CHALLENGES AT THE PORT OF MIAMI TUNNEL PROJECT, MIAMI, FLORIDA

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ABSTRACT

The Port of Miami Tunnel will connect PortMiami directly to the surrounding highways, alleviating traffic congestion. The project comprises two 11.3-m-internal-diameter, 1.27-km-long tunnels bored under the main shipping channel in Biscayne Bay. These are the first large-diameter tunnels constructed through Florida's challenging carbonate geology, and because of the heterogeneous ground conditions, the baseline investigation left many geotechnical uncertainties. To reduce these uncertainties, a phased complementary ground investigation was performed, adopting numerous techniques, the integrated interpretation of which allowed development of a realistic ground model. The ground investigation, which proved to be one of Florida's most comprehensive, had a major impact on the tunneling methods that were adopted.

INTRODUCTION

For over 20 years, the Florida Department of Transportation (FDOT) studied the feasibility of constructing a tunnel to connect Miami's cruise and cargo port, which is one of the busiest in the U.S. and is Miami-Dade County's second largest economic generator, directly to the well-developed local highway system. By 2006, advances in pressurized tunneling technology enabled FDOT to move forward with this challenging tunnel project.

The Port of Miami Tunnel (POMT) project is a Public Private Partnership between the Miami Access Tunnel Concessionaire, FDOT, Miami-Dade County and the City of Miami and is being constructed by Bouygues Civil Works Florida (BCWF). Together with BCWF, design consultants Jacobs Engineering Group and Langan Engineering & Environmental Services make up the project design-build team.

The project comprises construction of two 11.3-m-internal-diameter, 1.27-km-long, two-lane road tunnels between Watson and Dodge islands and under the main shipping channel in Biscayne Bay, known as Government Cut. The project also includes extensive road improvements and widening of a major over-water bridge.

The local carbonate geology, proximity to the coast, low ground surface elevation, and the high groundwater levels have historically made major underground construction projects in Florida difficult. Prior to POMT, tunneling in Florida was limited to small diameter (typically 2-m or less) utility micro-tunnels and a few short and shallow cut-cover/immersed tube type transportation tunnels.

The POMT is a unique project that has embodied many technical firsts, and at 11.3-m-internal-diameter, these large-diameter tunnels are the first to be constructed through Florida's challenging sedimentary carbonate strata. Site conditions required mining with a Tunnel Boring Machine (TBM) and for excavation to begin above the natural ground surface with the completed tunnels having minimal cover (less than one tunnel diameter under the shipping channel) and limited horizontal separation between the tunnels (0.3 to 1.25 tunnel diameters). The project required one of Florida's most comprehensive ground investigations, using conventional and unconventional techniques to understand the highly heterogeneous conditions.

The project is on schedule with an anticipated completion in May 2014. Once complete, the tunnel will meet major socioeconomic needs by connecting the Port to the surrounding highway system, alleviating traffic congestion in Downtown Miami, and improving the quality of life while keeping the Port competitive with the rest of the world.

GEOLOGICAL SETTING

Geological conditions at the project site consist primarily of carbonate sediments resulting from climatic fluctuations and changes in sea level that occurred between 10,000 and 2 million years ago. As a result of the changing depositional environment, five geologic formations are present in the project area; Miami Limestone Formation, Fort Thompson Formation, Anastasia Formation, Key Largo Formation and Tamiami Formation. Numerous changes in sea level over a relatively short geologic timescale have resulted in a heterogeneous profile consisting of interbedded sedimentary rocks, loose sediments and intermediate geomaterials, all of which vary both vertically and laterally.

Conventional and geophysical ground exploration techniques utilized in these variable conditions have historically been adequate for surface-based projects (bridges, buildings, etc.); however, they proved to be limited in properly developing realistic ground models for a major underground project.

BASELINE CONDITIONS AND TENDER GROUND MODEL

The ground investigations performed before the tender included 87 borings, comprising primarily conventional Standard Penetration Test (SPT) borings, rotary core borings, borehole permeability tests and limited 100-mm-diameter sonic cores. Geophysical surveys, including seismic profiling at selected borehole locations for measurement of compression and shear-wave velocities, and seismic profiling through the shipping channel using seismic reflection and refraction techniques were also performed. The Geotechnical Baseline Report (GBR) defined three subsurface strata, based on general geologic terms, and established a maximum baseline permeability of 1-cm/sec (Parsons Brinkerhoff 2006).

Recognizing the challenges that would be involved with construction of a large diameter tunnel in such highly variable and challenging ground conditions, the design-build team developed an expanded ground model during the tender period in order to better understand the subsurface conditions and associated ground risks. The result was expansion of the GBR's three subsurface strata into an eight-strata ground model based on specific geotechnical properties, material composition and detailed geologic origin. The developed ground model is summarized in Table 1.

Particular concern was raised regarding a "less competent zone," which was indicated by the GBR as falling within the Fort Thompson Formation. This zone was defined as Layer S7 in the design-build team's tender-phase ground model. The top of this layer was identified at about elevation –25-m and was interpreted to be in the order of 7-m thick.

Based upon information provided in the bid documents, the design-build team's assessment, at the time of tender, was that Layer S7 consisted primarily of sand with interbedded zones of limestone and sandstone. However, because, on average, 74% of the material in Layer S7 was not recovered in the investigations, further post-award investigation would be required to properly understand the ground conditions along the tunnel alignment, in particular the composition and mechanical properties of Layer S7

Table 1. Expanded ground model developed during tender period

GBR Subdivision	Design-Build Team Layer No.	Approximate Top of Layer Elevation (Design-Build Team)	Strata Description (Design-Build Team)
Fill	S1	Ground Surface	Fill
	S2	–1m	Silty sand/silt (Marine Bay Sediments)
Miami Formation	S3	–3m	Weak Limestone (Miami Formation)
	S4	–6m	Sand with some limestone (Transition Zone)
Fort Thompson Formation	S5	–10m	Limestone with some sand
	S6	–15m	Cemented sand and shell
	S7	–25m	Sand with interbedded zones of limestone (GBR "Less Competent Zone")
	S8	–32m	Limestone/Sandstone with interbedded zones of sand

Note: Elevations are in meters below mean sea level (NGVD 1929).

and the appropriate ground treatment required to ensure that this layer did not detrimentally impact the tunnel excavation works.

The ground model identified mixed-face, highly permeable conditions along the entire tunnel alignment, with Layer S7 being the predominant layer under the crossing of the shipping channel and the most critical section of the tunnel alignment. The design-build team selected an Earth Pressure Balance (EPB) TBM to be the most appropriate based on the team's evaluation of the ground conditions at tender but it was essential for the design of the tunnel and the confirmation of the tunneling methods for the project that this layer be properly defined.

COMPLEMENTARY GROUND INVESTIGATION

After award of the contract a Complementary Ground Investigation (CGI) was performed in stages with the specific targets of confirming the tender-phase ground model, developing the geotechnical parameters necessary for design, understanding the zones of uncertainty, and defining the areas requiring ground treatment. Of primary concern was the selection of ground investigation techniques which would allow for realistic characterization of all subsurface layers critical to tunnel construction. This proved to be a particular challenge due to the difficulties in sampling the variably cemented soft sedimentary carbonate formations present.

The CGI began in December 2009 with conventional SPT borings and mud-rotary core borings. SPT borings have been widely used in South Florida and are effective in sampling the upper soils, soft limestone and siliceous sands. To maximize rock-core recovery, 100-mm-diameter conventional double-tube core barrels were specified, as well as 90-mm-diameter double- and triple-tube wireline coring systems, with the wireline coring systems proving to be the most effective of the mud-rotary coring techniques. These mud-rotary coring techniques were more successful in the better-cemented materials of the design-build team's ground model Layers S5, S6 and S8 (later confirmed as the Fort Thompson, Anastasia and Tamiami Formations, respectively) where typically higher recoveries were achieved. However, core recoveries remained low in Layer S7, with average recoveries remaining around 26%. Anomalously, very low SPT N values corresponded with low SPT spoon-sample recoveries in zones interpreted to comprise loose sand. However, the fact that these materials could not be recovered raised significant questions among the design-build team regarding the initial tender-phase ground model, and in the worst case, the feasibility of performing the tunneling

Figure 1. Sonic cores from Layer S7

in the event that the anticipated finer grading portion (i.e., sand and silt) was absent along the critical section of the alignment. There was no surface exposure of Layer S7 in the vicinity of the project site, so understanding the actual nature of this material found approximately 30-m below ground level without representative samples was a significant project challenge. Further detailed investigation was essential to address this gap in understanding the ground conditions.

Initially the CGI included Cone Penetration Test (CPT) probes for the investigation of the shallow upper Layers S1 through S4 and to evaluate the need for ground improvement in the areas of reduced tunnel ground cover and horizontal separation. Following the continued poor core-sample recovery from Layer S7 and the remaining significant concerns regarding the definition of this layer in terms of design and tunneling process, deep CPTs were performed to obtain continuous geotechnical data over the entire depth of the layer.

The CPTs were performed using a 15-cm^2, 18-tonne electric piezocone, which provided continuous measurement of the tip resistance, q_c, sleeve friction, f_s, and pore water pressure, U_2. Boreholes were drilled through the upper rock layers, and the CPTs were then advanced through Layer S7 using heavy-duty extension rods and a modified drill rig as a reaction. When refusal to cone penetration was encountered (typically at about 55-MPa), the cone was removed and the hole was advanced by drilling for approximately 15-cm and then cone penetration was resumed. This process was continued until full penetration through Layer S7 was achieved. The CPTs were effective in obtaining continuous data and the measured q_c values ranged from zero to 55-MPa, averaging about 8-MPa, indicating Layer S7 to be highly variable with significant zones of very weak and potentially unstable voided material.

The use of CPT is widely accepted as a soils investigation technique and its penetration through Layer S7 was somewhat unexpected; however, with the success of the initial probes in obtaining an almost continuous "profile" through Layer S7, additional deep CPTs were performed on a systematic grid along the length of the tunnels.

The sonic coring technique reliably obtains full sample recovery through most geomaterials, although the core samples can be disturbed by the sampling process. In an attempt to improve on the core recovery, 150-mm-diameter sonic cores were performed for the CGI and provided complete material recovery from all the investigated layers. Although disturbed by the drilling process, these cores gave the first indication that Layer S7 comprised variably cemented, highly dissolved, collapsible/metastable coralline limestone (coral heads in a variable calcarenite matrix) with high porosity (20% to 80%) and low fines content (i.e., sands and silt). Typical sonic-core samples from Layer S7 are shown in Figure 1. Larger 200-mm-diameter and 300-mm-diameter sonic cores, introduced at a later stage of the CGI, were effective in reducing disturbance/fracturing

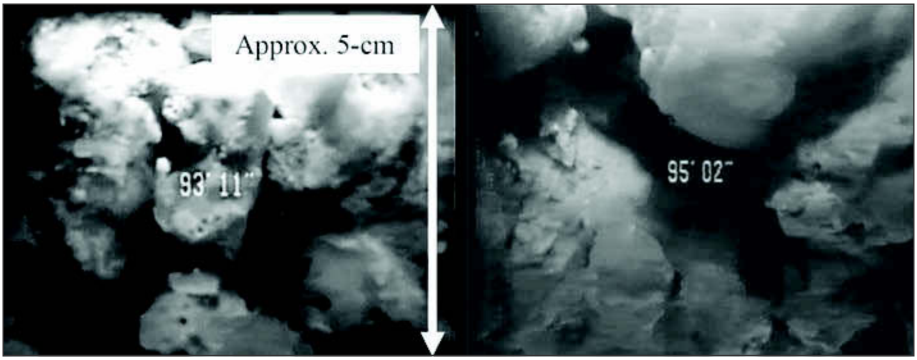

Figure 2. Extract from video of large-diameter exploratory shafts

of the core samples and confirmed the highly variable, porous and dissolved condition throughout the coralline limestone of Layer S7.

The recovery of the coral pieces within the calcarenite matrix led to the understanding that Layer S7 represented the Key Largo Formation. Literature indicated that in the Miami area the Key Largo Formation generally interfingers the Fort Thompson Formation. The design-build team visited the nearest surface exposure of the Key Largo Formation (located in a former rock quarry in the Florida Keys approximately 130-km from the site) which was observed to consist of a rock formation comprising coral fragments encased in a well-cemented calcarenite matrix. However, the unexpected thickness of the coral deposits encountered at the tunnel site, and the fact that the layer could be penetrated easily by SPT and CPT, were not consistent with the conditions observed in the Florida Keys, indicating that the Key Largo Formation of Layer S7 had undergone significant alteration in geological time. From the evaluation of the CGI data, it was inferred that the coralline Key Largo limestones within Layer S7 had undergone significant dissolution, which was likely caused by the flow of freshwater.

With the developing understanding that Layer S7 had a low “fines content” with a high potential for being unstable during tunneling the design-build team had significant concerns regarding the feasibility of effectively performing EPB tunneling through this material, and further investigation was considered essential to finding a solution.

Exploratory test shafts, 2,100-mm in diameter, were excavated on both Watson and Dodge islands to confirm the geological and geotechnical conditions inferred by the interpretation of the sonic borings, cores and CPTs, and to obtain bulk samples for TBM-spoil conditioning tests. These shafts were advanced with casing to an elevation –20-m (mid Layer S6) and drilled uncased to termination at elevation –30-m (top of Layer S8). Shaft excavation was followed by radar profiling (Sonicaliper) and video survey of the uncased portions of the holes. The Sonicaliper revealed that the uncased portions of the shaft excavation in Layer S7 experienced instability and significant enlargement of the hole (over-break), as opposed to excavation through Layer S6, which remained stable during uncased excavation. Video survey of the shafts confirmed that Layer S7 consisted of a very porous stratum composed of variably cemented coral heads and limestone fragments in a variable calcarenite matrix, as shown in Figure 2. Examination of the excavated spoil recovered from the shafts revealed that the cemented materials from Layers S5 and S6 would break into fragments, which were considered representative of how the material could break up when excavated by the TBM. However, the excavated materials from Layer S7 were generally very coarse and contained considerable amount of boulder-sized coral and limestone fragments. The recovered material was used to perform an extensive suite of spoil conditioning trials to further investigate the feasibility of effectively forming the tunnel using the EPB process through Layer S7 (Merritt et al. 2013).

The CGI also included other targeted investigations to address specific project challenges, including exploratory boreholes and in situ nondestructive testing to confirm the depth of existing pile foundations for structures located along the tunnel alignment and a hydrophysical investigation to confirm ground permeability and evaluate groundwater flow velocity.

The nondestructive testing performed targeted tip elevation of the sheet-pile bulkhead wall and pile foundations for port structures in the proposed bored-tunnel alignment area. The borehole testing included magnetometer testing, parallel seismic testing and borehole ground-penetrating radar testing. In addition, pulse echo testing was performed from the ground surface. The borehole test results indicated the tip of the bulkhead sheet pile and pile foundations in the subject area to be above the tunnel envelope. The pulse echo testing was inconclusive possibly because of a lack of contrast between the pile and the ground.

The hydrophysical investigation was performed in a deep observation well and included the measurement of the velocity of suspended particles using a colloidal boroscope and the measurement of changes in fluid electrical conductivity following replacement of the fluid column with deionized water. The hydrophysical work revealed hydraulic conductivities in the order of 0.01- to 0.1-cm/sec, and groundwater flow velocities on the order of 6- to 100-m/day at the interface of Layers S6 and S7.

The CGI lasted approximately 22 months and ended being one of the most comprehensive ground investigations performed in South Florida. One hundred and fifty-two additional investigation holes were completed along the tunnel section both on land and offshore, equating to approximately one investigation hole for every ten linear meters of tunnel. An additional 126 investigation holes were performed for the other project components (roads, bridges, etc.).

INTERPRETATION OF CPT DATA

Established correlations between CPT data and soil behavior types (sand, silt, clay, gravels, etc.) were clearly not going to be applicable to soft sedimentary carbonate rocks and their use would result in misleading interpretations. The development of a site-specific correlation between the CPT data and the likely behavior of Layer S7 during tunneling was necessary. The correlation relied on the integration with the local geology of the findings from the different investigations: the SPTs, the sonic and conventional coring, the large diameter exploratory holes and videos of the borehole walls, and the geophysics.

The SPT results and samples distinguished the following two main coexisting zones in the porous coralline limestone of Layer S7:

- *Well-cemented zones* characterized by a low recovery (and penetration) with high SPT-N values;
- *Weakly cemented and very high porosity zones* characterized by a low recovery (over a full length of test) with low to very low SPT-N values.

A third zone was identified by the SPT method in a relatively isolated area where very soft lime silts were characterized by a nearly full recovery and very low SPT-N values.

These zones were confirmed by the sonic core investigation and by the notion of "*apparent recovery,*" which was defined as the ratio of the compressed length of the fully recovered sample to the theoretical core run length. Measuring the depth of the hole after every run confirmed that 100% of the in situ material was recovered. Apparent recoveries as low as 25% resulted from the compression of the honeycomb

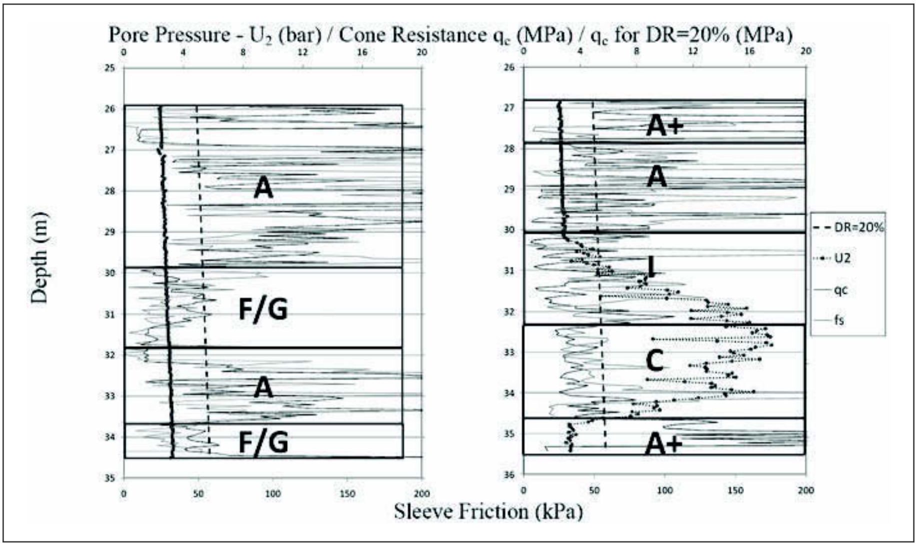

Figure 3. Typical CPT response in Layer S7

structure in the weakly cemented and very high porosity zones within Layer S7. Much higher apparent recoveries applied in the well-cemented zones.

The three zones identified on the basis of the SPTs and sonic cores were also identified in the CPT profiles, typical examples of which are shown in Figure 3. Interpretation of the CPT data enabled the identification of the presence of distinct units within Layer S7. Units A and A+, with high values of q_c, were well-cemented but with large pores that gave rise to the rapid fluctuations in q_c and f_s. Units F/G, with extremely low q_c and f_s values, had a weakly cemented honeycomb structure and were considered to have a high risk of instability during tunneling because of their very high porosity and extremely weak cementation. Unit C, with low q_c and relatively high values of penetration pore pressures, formed the zone of very soft lime silts, which presented a risk of liquefaction during tunneling operations. Other variants of these units were also identified.

The systematic analysis of the CPT results and the correlation with nearby sonic cores led to the detailed understanding of the geological/geotechnical model of the in situ Layer S7/Key Largo Formation highlighting zones where instability could occur during tunneling. Identification of these zones was aided by adding to Figure 3 a line representing the value of q_c that would be expected when penetrating a normally consolidated sand at these depths with a relative density of 20%. Several zones in Layer S7 have q_c values well below this line. Figure 4 shows examples of sonic core samples of different units identified on the basis of the CPT profiles.

GROUND TREATMENT TRIALS

Based upon the design-build team's understanding of the subsurface conditions developed during the tender studies, ground treatment was considered necessary to improve zones of very loose sand identified within the "less competent zone" and which could have a significant impact on the ability to steer the TBM. Ground-treatment trials were a contractual requirement to define appropriate means and methods to efficiently improve the ground, taking into account the high porosity of the ground and the historical tendency for grout to flow away from the target area. Based on the tender-phase ground model, an initial ground treatment trial was implemented concurrently with the

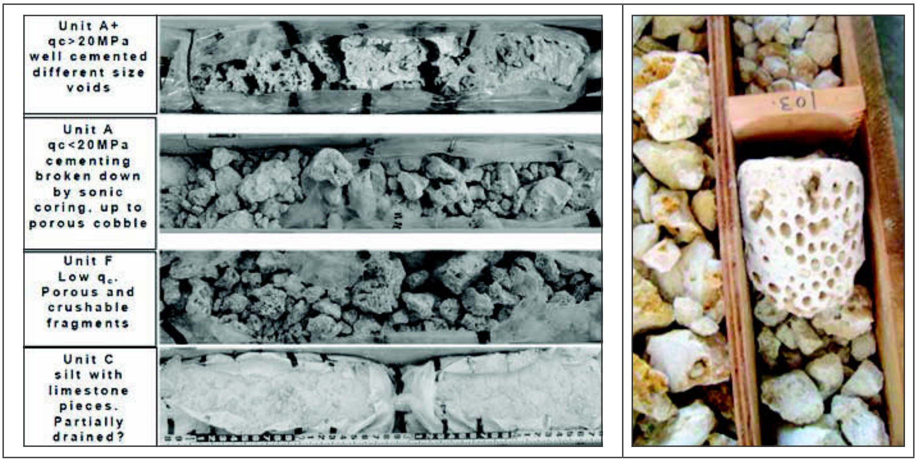

Figure 4. Sonic cores of different units of Key Largo limestone in Layer S7 identified on the basis of CPT profiles

early phases of the CGI. This first trial was based on using a very stiff, dry, low-mobility grout to target compaction of the interpreted very loose sand (SPT N value of less than 10, occasionally zero).

Analysis of the grouting parameters from the first trial indicated the ground behavior to be inconsistent with loose sand. This outcome added further to the understanding that the "less competent zone" or Layer S7 was a complex geological layer with particular properties that needed to be clearly understood before tunneling.

The results of the first ground treatment trial and the understanding from the CGI that Layer S7 was comprised of highly porous and dissolved Key Largo limestone substantially devoid of fines, raised serious concerns about the ability of efficiently performing EPB tunneling in this layer. The high potential for instability during tunneling (metastable/contracting behavior when sheared) and the inability of effectively performing the EPB process required the investigation and evaluation of alternative means to ensure tunnel excavation stability in Layer S7.

To prevent the collapse of the porous structure ahead of and around the advancing TBM, partial filling of the void space with low mobility grout was foreseen to be an effective solution. However, further investigation was required and nine additional full-scale field grouting trials were performed to satisfy two defined goals:

- to identify stable grout formulations that were capable of effectively penetrating the porosity of Layer S7 and which had rheological properties sufficient to manage the risk of "runaway" takes or washout. Equally, such mixes had to have low strength, compatible with the ground and be amenable to the TBM excavation.
- to determine and optimize operational parameters relating to the placement of the grout, such as efficient injection hole spacing and delivery method (e.g., upstage versus downstage).

The results of the trials confirmed the robustness of the developing geological/ geotechnical ground model, namely the highly heterogeneous nature of Layer S7 in terms of instability, strength and lithology, and the generally very high porosity and permeability of the layer. Based upon the findings of the field trials, a unique low mobility, thixotropic grout was developed, suited to the site specific conditions, and optimal injection hole spacing was defined to achieve the formation grouting program's target

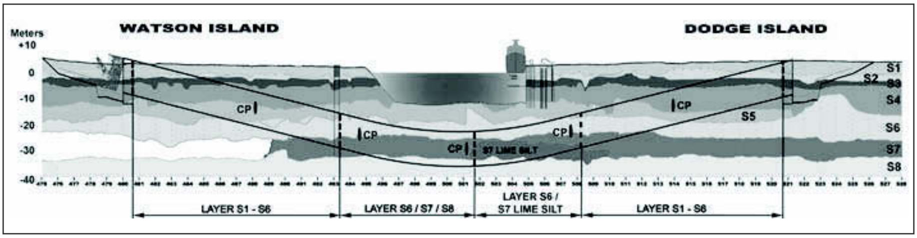

Figure 5. Port of Miami tunnel: geological section (Eastbound tunnel)

of controlling groundwater and stabilizing the metastable Layer S7 along the tunnel alignment.

FINAL GROUND MODEL AND IMPACT ON TUNNEL CONSTRUCTION

The result of the CGI was the development of a refined and detailed ground model that was consistent with all available data and that was sufficiently detailed to allow assessments to be made of the response of the ground to TBM tunneling and the need for and design of ground treatment. The CGI confirmed the preliminary assessment of Layers S1 through S6 and S8. However, the CGI revealed that Layer S7 was significantly different from the preliminary assessment at the time of tender. The final ground model is presented in Figure 5 and summarized in Table 2.

As opposed to the interpretation at tender, which indicated Layer S7 to consist of sand with zones of limestone, the CGI determined Layer S7 at the site to consist of an unusually thick deposit of the Key Largo Formation, comprised primarily of coral heads and limestone fragments loosely bound in a variable calcarenite matrix. The CGI further revealed the coralline limestone layer to be weak, highly dissolved and porous, with zones that would be potentially unstable during tunneling.

The CGI found soil deposits within the predominately coralline Layer S7. However, these soil deposits were found to consist of a localized zone of loose sands at the Watson Island entry/exit shaft but outside the vertical tunnel alignment, and a localized zone of silt present along the eastbound tunnel alignment between the south end of the channel and Dodge Island.

The final ground model had a direct impact on the project, resulting primarily from the presence of the weak, highly porous Key Largo coralline limestone in Layer S7. Some of the impacts associated with the ground conditions were as follows:

- Tunnel excavation required the use of a pressurized face TBM. The high permeability and the potential for slurry loss led to the selection of an EPB TBM as the only feasible solution. After the CGI and development of the ground model, the design-build team determined that EPB tunneling was feasible only in the upper soils and better cemented rock layers (Layers S1 through S6); however, serious concerns arose regarding the possibility of effectively and robustly conditioning the excavated spoils and maintaining an effective EPB process where the tunnel bores encountered Layer S7. The conclusion of the spoil conditioning tests indicated that EPB tunneling would only be feasible in approximately two-thirds of the length of the tunnel bores, and that EPB tunneling could not be performed for the most critical portion of the tunnel drives—the crossing of the shipping channel.
- To maintain face and ground stability along the sections of the tunnel bores where EPB tunneling was not possible, an extensive formation grouting campaign was implemented. As previously discussed, the final formation grouting program was developed over nine full-scale trials to find the optimal grout

Table 2. POMT project ground model

Soil Layer	Geological Description	Strata Description
Layer S1	Man-Made Deposits	Reclamation/dredged limestone and sand fill
Layer S2	Coastal Sediments	Sand, silty sand and silt
Layer S3	Miami Limestone	Weakly cemented limestone with fine sand
Layer S4	Transition Zone	Siliceous sand, limestone/cemented sand layers
Layer S5	Fort Thompson Formation	Moderately to strongly cemented, fine to medium-grained sandy Limestone (UCS 1.5–35.5MPa)
Layer S6	Anastasia Formation	Cemented shell (Coquina)/cemented sand (UCS 2.4–24.2MPa)
Layer S7	Key Largo Formation	Coralline limestone, heavily dissolved and highly porous (coral and limestone fragments weakly to very weakly cemented with calcarenite with zones of uncemented fragments and sand lenses)
Layer S7 SILT		Lime silt with varying amounts of limestone fragments
Layer S8	Tamiami Formation	Limestone and sandstone with interbedded lenses of cemented sand, cemented shell and sand (UCS 0.9–35.9MPa)

mixture and injection pattern. The formation grouting campaign involved more than 1,000 grout holes and the injection of approximately 50,000-m^3 of grout, and was performed in advance of the TBM progress both on land and offshore from barges. Grouting operations in the shipping channel were performed without impacting the busy cruise-ship schedule, and typically limited to working one to three days per week.

- The inability to perform EPB tunneling in Layer S7 led to a series of late and significant modifications to the TBM. An innovative Water Controlled Process (WCP) tunneling method was designed to overcome the challenges associated with the conditions in Layer S7. For operation in WCP mode, a hydraulic circuit similar to that in slurry TBMs was connected to the end of the screw conveyor with an inline crusher (Storry et al. 2013). The WCP mode allowed for balancing the high hydrostatic pressures at the face of the TBM and allowing for the controlled removal of the excavated spoils.

Monitoring of the excavated spoils and TBM parameters during the excavation of the tunnels confirmed that the ground conditions encountered along the tunnel alignments were consistent with the final ground model interpreted by the overall ground investigations.

CONCLUSIONS

The experience at POMT confirmed that for large and complex underground construction projects in difficult and challenging ground conditions to be successful, up-front comprehensive ground investigations and detailed method studies are essential to the understanding and mitigation of potential risks. The adoption of a phased and systematic approach to the ground investigation is essential in identifying the most effective exploration methods for difficult geologic conditions and helps maximizing effort and resources in a demanding project schedule.

Complex geologic conditions, such as young soft carbonate sedimentary rock formations and intermediate geomaterials, will sometimes require the use of unconventional and out-of-the-box ground investigation techniques when classical methods do not provide an adequate representation of the in-situ conditions. Development of site-specific correlations of the ground investigation data must be performed when classical methodologies are not applicable to the geologic conditions at the project.

The cost of the CGI performed by the design-build team was approximately 1.5% of the total construction cost. Although substantial, such an investment in ground investigation paid dividends in the long run by enabling significant risks to be fully identified, understood and appropriately mitigated.

ACKNOWLEDGMENTS

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GROUND FREEZING FOR THE EAST SIDE ACCESS NORTHERN BOULEVARD CROSSING—BALANCING SETTLEMENT AND HEAVE CONTROL

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Frank Pepe ▪ Parsons Brinckerhoff

ABSTRACT

One of the most challenging elements of the MTA Capital Construction Project's East Side Access Project was the Northern Boulevard Crossing which required an SEM tunnel between two 85-ft deep access shafts which extended to as deep as 55 ft below the water table. Tunneling was accomplished beneath a frozen ground arch connecting the two deep excavations, and within very close proximity of the overlying, active subway and elevated transit lines. The work required significant measures to permit installation of freeze pipes and grout pipes through highly variable subsurface conditions below the water table, as well as two critical ground improvement requirements: settlement control and heave control. This paper discusses the pipe installation, the pre-grouting, settlement and heave control, monitoring and instrumentation, and their integration with the overall program.

PROJECT BACKGROUND

The MTA Capital Construction East Side Access (ESA) project, connecting the Long Island Rail Road to Grand Central Terminal in Manhattan, required several tunnel contracts between Sunnyside railyard in Queens into Grand Central Station in Manhattan. One of the most challenging elements of the overall project was the Northern Boulevard Crossing, a 125-ft long sequentially excavated (SEM) tunnel between two 85-ft deep access shafts. The tunnel was situated approximately 55 ft below the water table and would be mined through glacial deposits including silts, sands, boulders, till and bedrock. The tunnel alignment also crossed beneath a pile-supported, elevated railway line and a below-grade subway structure.

Earth support and groundwater control were critical to the tunneling operation. However, project constraints precluded vertical drilling both from ground level and from within the existing subway structure itself. Lowering of the groundwater table was also prohibited because of contaminant plumes in the vicinity. The method ultimately selected was the creation of a horizontally-installed protective frozen arch above the tunnel alignment, extending to bedrock for complete groundwater cut-off, as illustrated in Figure 1. This was determined to be a viable option that would support the excavation without disrupting the highway and rail traffic above while at the same time meeting the groundwater control requirement.

The design of the ground freezing system was an aggressive approach for a number of reasons:

1. The horizontal orientation of the pipes would require drilling from below the water table through the silty sands and sensitive silts known locally as bulls

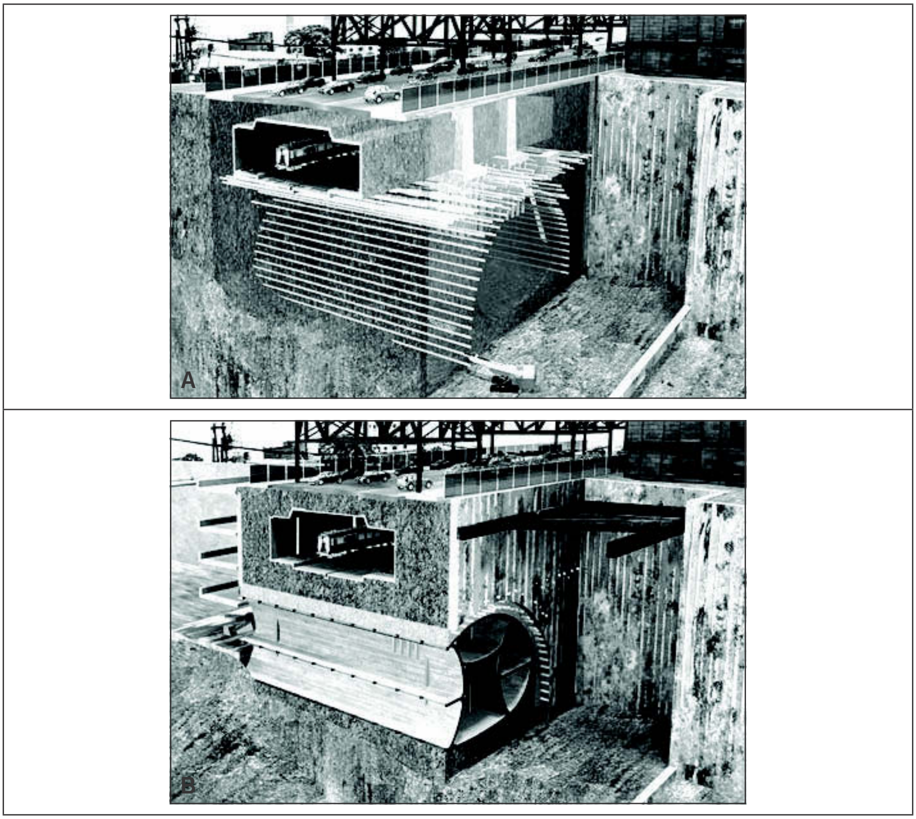

Figure 1. Creation of a protective frozen arch above the tunnel alignment by means of horizontally-installed freeze pipes (a) was selected to provide excavation support and complete groundwater cut-off during SEM tunneling (b)

liver soils. Some loss of ground would be likely, which would result in settlements and possibly movement of the installed pipes.

2. An appreciable thickness of sensitive bulls liver soils (rapidly dilatant silt) was present through much of the tunnel cross-section. This material would be susceptible to the formation of ice lenses and heave. Ice lenses typically exert forces in the direction of the temperature gradient which is normal to the orientation of the freeze pipes and the freezing plane. For vertical freezes, the forces are lateral, counteracted by greater lateral earth pressures at depth, and are rarely an issue, but for horizontal freezes they are vertical, which would exacerbate heave of the structure, particularly at shallower depths.
3. Under the best scenario, with minimal deviation of the drilled freeze pipes, there would still be only several feet between the top of the frozen arch and the bottom of the existing subway structure. It was from this thin layer of ground that the growth and heave of the freeze had to be controlled.
4. As with any freeze, placement of the freeze pipes within the design tolerance is important. Several significant conditions would make alignment of the pipes very challenging. Four clusters of 16 concrete-filled pipe piles had to be

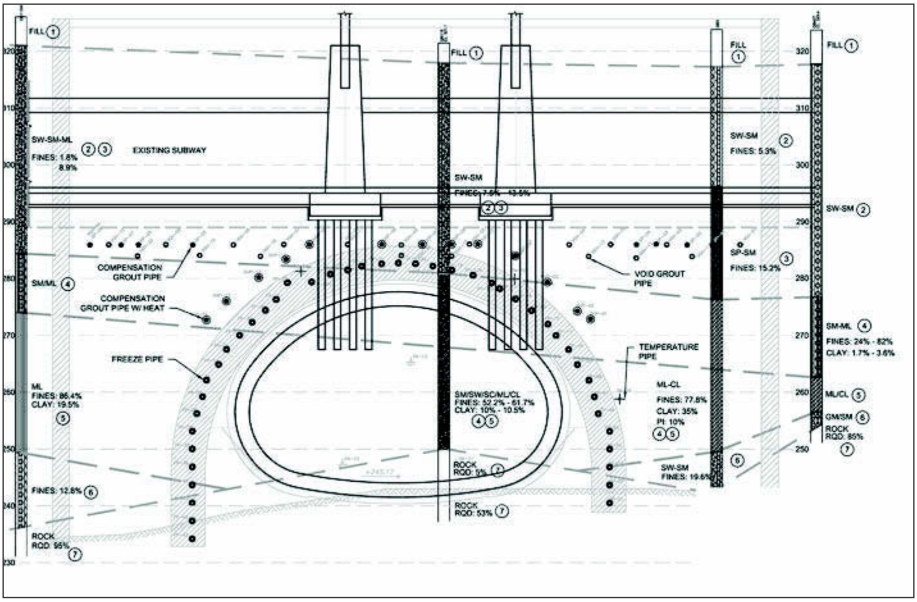

Figure 2. Various drilled holes and geologic profile. Stratum 2 consists of silty sand, the sensitive silty sandy and silt is stratum 4, and stratum 5 is till above the rock.

penetrated (without damaging the piles), as well as cobbles and boulders, and an undulating rock surface.

5. The surrounding groundwater regime is heavily influenced by the presence of pre-existing subsurface structures and cut-off walls. Excessive groundwater gradients, if they exist, could cause difficulties for the formation of the freeze.

The geologic profile is presented in Figure 2.

CONDITIONS SPECIFIC TO THE GROUND FREEZING SOLUTION

A number of factors had to be addressed prior to, during, and following the ground freezing operation. Horizontal drilling for freeze pipe installation would be accomplished below the groundwater table from within what was known as the Early Access Chamber, a specially created work area bounded by a structural slurry wall earth support system. The construction of the crossing required tunneling and installation of freeze pipes below the water table and below settlement- and heave-sensitive structures, as discussed above and illustrated in Figure 1. Therefore it was necessary to implement controls to mitigate the potential risk of and correct for these potential ground deformations. This included the potential ground loss and settlement during freeze pipe installation. A significant portion of the frozen arch would be formed in soils susceptible to frost heave, so a heave control mechanism also needed to be in place. During thawing of the frozen ground following completion of the tunnel, some settlement was anticipated and this, too, had to be addressed.

The mechanisms developed by the design team and the contractor retained for the ground freezing work were as follows (listed in chronological order of their performance):

- *Void filling* of the soils beneath the subway box prior to freeze pipe installation. This was originally intended fill any open, water-filled, zones beneath the structure, but it also provided some pre-conditioning of the ground necessary to control settlements if they occurred.

- *Compensation grouting* through pre-installed grout pipes to mitigate settlement of the overlying structures during installation of the freeze pipes.
- *Soil extraction* from the soil zone between the frozen arch and the base of the subway box to counteract heave of the structure during freeze formation and freeze maintenance.
- *Heat Pipes* around the top of the arch to control the outward growth of the freeze.
- *Compensation grouting* through the pre-installed grout pipes to mitigate settlement during the thaw of the frozen arch upon completion of the work.

GROUT DESIGN

Since pre-conditioning/void filling and compensation grouting with conventional cement-based grouts could render the soils unworkable for subsequent soil extraction, a specially formulated non-cementitious grout was developed by the geotechnical contractor that essentially mimicked the strength and consistency of the in situ soils (Figure 3).

Figure 3. A sample of the non-cementitious grout field-tested before the trial program. Note the decreased volume of the sample due to intended pressure filtration.

The grout mix was composed of a fine sand/coarse silt base, a viscosity-modifying additive, water, and super-plasticizer, and formulated to provide a minimum strength bulk fill material that had sufficient stability (bleed and pressure filtration characteristics) to permit pumping and injection through Tube à Manchette (TAM) pipes. An intentionally high pressure filtration coefficient allowed for the weight of the overburden to essentially squeeze the water out of the in-place grout so that the grout left in place would exhibit characteristics more similar to soil than a fluid.

FIELD TRIAL—COMPENSATION GROUTING

Compensation grouting involves the intentional systematic hydro-fracturing of the soil beneath a structure by the controlled, uniform, high-pressure injection of grout. It is typically performed through Tube à Manchette (TAM) pipes in order to induce heave and thus compensate for actual settlements, or to mitigate anticipated settlements. Grouting is typically performed in several phases with repeated, consistent-volume injections at each port to ensure the formation of multiple fractures through the soil.

The compensation grout pipe array (see Figure 2) was designed to accommodate the space limitations. Grout pipes followed the arch of the freeze in the narrow space between the subway structure and the top of the frozen arch. These pipes were also equipped with heat cables to limit growth of the freeze. Above the shoulders of the arch, the pipes were located ten feet away from the structure and even further from the freeze arch so as not to minimize the concentrated point loading to the structure and frozen arch. The pipes were extended to the north and south of the area to cover the whole influence zone above the arch.

An on-site trial was performed on three adjacent production grout pipes to evaluate grout pipe installation procedures; hole protection methods (blow-out preventers, stuffing boxes, etc.); mixability, pumpability, consistency and behavior of the grout; the ability of the grouting to induce sustained heave of the subway box structure; and

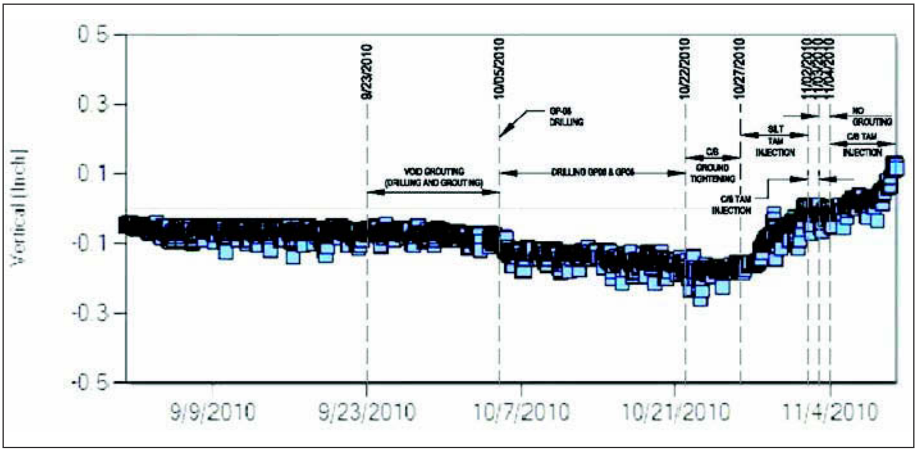

Figure 4. Elevation data from a monitoring point within the subway box immediately above the compensation grouting trial section. Data reflects some settlement during pipe installation (first horizontal pipes drilled on site) and then a steady rise with the injection of grout.

coordination between acquisition and delivery of the settlement/heave data and the grout injection.

Rotary flush drilling was used to advance 5.5-inch diameter casing. In order to prevent loss of ground through the drill tools, the casing was fitted with and an internal check valve and a slightly oversized bit to permit drilling by positive flush methods. At design depth, the TAM pipe, with a minimum diameter of two inches to permit surveying, was inserted in the casing and the drill bit and valve assembly pushed off the end and left in place. This installation method had the advantage of not exposing the TAM ports to abrasion from the drilling process.

The far end of the TAM pipe was, for these initial holes, equipped with a spring basket anchor to ensure it remained in place during casing retraction and also fitted with several “K Packer” wipers to provide a seal between the casing and the TAM itself and prevent the backflow of sand and groundwater. The annular space between the TAM pipe and casing was filled with cement-bentonite grout as the casing was extracted.

The trial was coordinated with real-time monitoring from a total monitoring station inside the subway structure and the grouting process was accurately recorded using a real-time grouting data monitoring system. As the trial was considered complete once a sustained lifting of the structure was achieved, the pre-conditioning, “tightening” phase and lifting phases of the work were rolled into one continuous operation. Figure 4 shows the data from one of the overlying monitors installed within the subway box. Slight settlement can be seen during the installation of the pipes, followed by a pronounced lifting trend. Sustained and controllable lifting of the structure was achieved and the trial deemed a success. However, a significant volume of grout was injected in order to achieve this and with a corresponding significant amount of movement of the grout pipes. One of the three pipes was rendered unusable mid-way during the trial grouting.

VOID GROUTING/PRE-CONDITIONING

The first production phase of the grouting and ground improvement process required to protect the adjacent existing structures was to fill any potential voids present below the subway structure resulting from earlier construction activities at the site and pre-condition or “tighten” the existing ground. The intent of the program was to

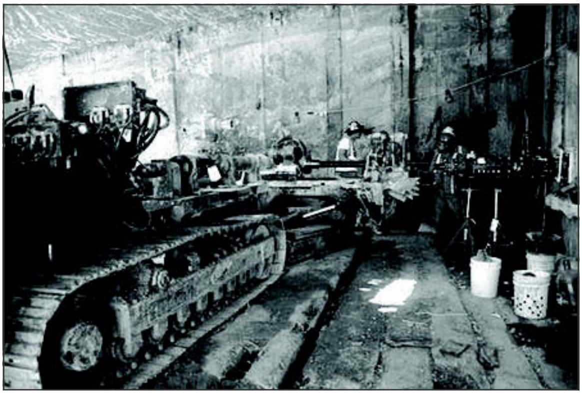

Figure 5. Drilling through a blowout preventer

prepare the ground so that future settlement, if it occurred, could be corrected quickly with controlled grout injections.

Given the large volume of void grouting anticipated, end-of-casing grouting was selected. In this method, the grout is injected directly through the drill casing. As injection proceeds, the casing is slowly rotated and retracted in short (one foot or less) increments as the friction from the grout on the casing is felt with the rig. To limit the potential for uncontrolled lifting of the structure, the maximum allowable grouting pressure utilized at any stage was dependent on the total volume injected into the hole at that point.

All drilling and grouting was performed through groundwater control devices (blowout preventers) (Figure 5). At each location, grouting was accomplished through 3.5 in. diameter casing on a nominal 10-foot spacing and continued until pressure refusal or a total grout volume per hole of approximately 6,300 gallons was reached. Production grout was field-tested three times daily, with the key parameters of specific gravity, viscosity and pressure filtration closely monitored. During grouting, the structure was monitored continuously for movement.

Grouting was initially performed through 10 holes, but extended to an additional 10 holes after grouting of the initial holes was generally terminated upon a pre-determined volume refusal rather than a pressure refusal or movement of the subway, indicating that the intent of filling voids and "tightening" the ground was not yet achieved.

A total of 58,000 gallons of grout (Figure 5) was pumped beneath the subway structure during the void grouting phase. This high volume of grout reflects the extremely loose condition of the ground beneath the structure initially, a situation that was fortunately rectified before continuation of the work. Stiffening up the ground was also effective in pre-conditioning for subsequent compensation grouting. The improvement of the ground additionally resulted in greater controllability of drill deviation (Figure 6).

FREEZE PIPE AND GROUT PIPE INSTALLATION

Freeze pipe design is based on a specific distance between pipes, so pipe deviation is always of concern. Although less critical, grout pipe alignment was of concern to permit grouting where it was needed and maintain safe distances away from the frozen arch and the overlying structure. Maintaining drill alignment while penetrating through old sheeting, concrete-filled pipe pile clusters, cobbles and boulders, and an undulating rock surface was difficult enough. Doing that from below the water table, for the most part through very sensitive silty soils, compounded the complexity of the work as well as significantly restricted drilling methodologies. Significant measures were therefore

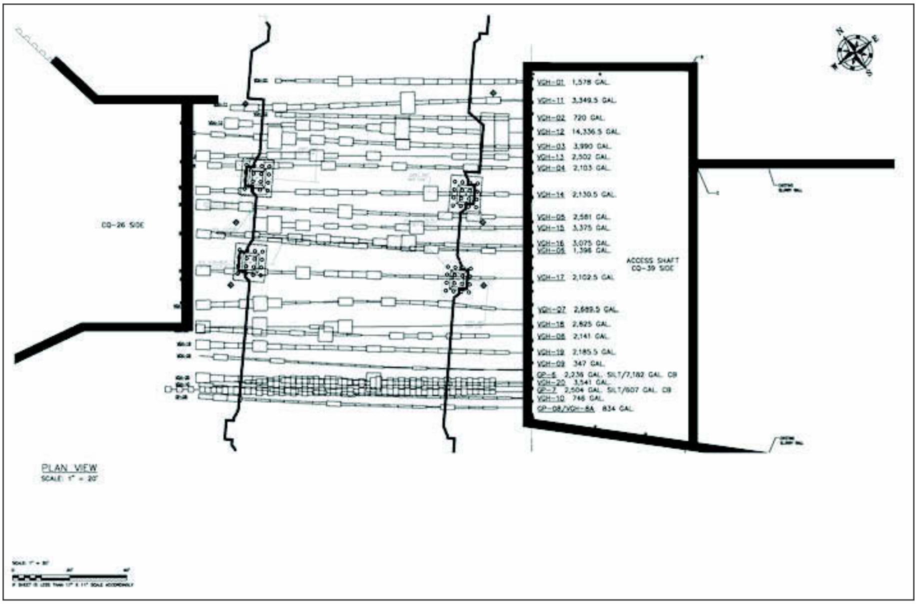

Figure 6. Void grout holes shown on their as-built alignment. Boxes shown are a proportional representation of the void grout volumes injected.

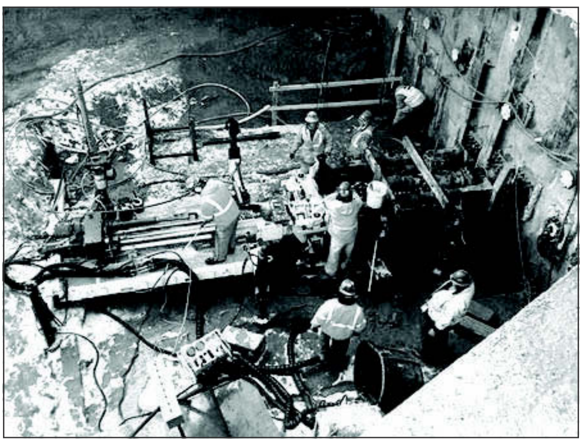

Figure 7. Coring drill used in areas of anticipated obstructions

put into place by the ground freezing contractor to install the pipes within permissible tolerances while preventing ground loss.

Drilling was accomplished in stages as excavation progressed down to final elevation. Skid-mounted core drilling with the capability to advance multiple casings was the primary drilling method selected where obstructions were anticipated (Figures 7 and 8). Because of the higher rotation speeds, this method is typically better suited for drilling straighter holes through obstructions. The core drilling tools also allowed for multiple reductions in casing size (telescoping), if necessary to permit casing and bit changes to continue drilling through obstructions. In areas anticipated to be free of obstructions, pipes were advanced by cased hole, positive flush methods. A back-up provision was

Figure 8. Drilling of the lowermost freeze holes from a deep pit excavated in rock. The lowermost pipes had to be installed full length through competent rock.

made for sonic drilling to be utilized should any borehole meet a refusal condition during core drilling. However, this was not required.

The TAM grout pipes were installed as discussed above for the trial grouting program with positive flush drilling methods, a lost bit, and wipers to prevent an inrush of ground as the casing was being pulled. Freeze pipes were installed similarly, but welded in sections as the pipe was installed.

Groundwater control devices (blow-out preventers) with redundant design features virtually eliminated the possibility of soil and groundwater washing out during the drilling. Significantly larger devices were installed for core drilling operations to enable telescoping of cases through, while smaller, more easily handled devices were installed for duplex drilling. The groundwater control device required a standpipe inserted and grouted into a pre-drilled hole in the slurry wall and secured to a face plate secured to the slurry wall by rock anchors. A large gate valve was bolted to a flange on each standpipe.

The as-built freeze pipe array and freeze formation schedule was verified by computer modeling. Accuracy of compensation pipe installation was also critical since the target zone between the top of the frozen arch and the underside of the subway box structure was very limited. All pipes were surveyed with a gyroscope immediately upon completion.

HEAVE CONTROL

Only on a handful of projects has soil been intentionally removed beneath or alongside a structure in order to correct for differential settlements. It has been referred to as "under-excavation" in the few previous instances. The best known structure to benefit from under-excavation is the Leaning Tower of Pisa in Italy. In 2003, the normally high water table was lowered by the installation of a new drainage system and, over a 2-year period, 70 tons of soil was very gradually removed from beneath the northern side of the tower, away from the lean, allowing gravity to restore the tower to a safe angle while still maintaining the historic tilt.

Under-excavation has also been performed similarly in Mexico City and at Boston's Big Dig. In Boston, on contract 9A4, which involved the jacking of the tunnel boxes beneath the railroad tracks outside of south station, soil extraction was required because of the pressure exerted on the tunnel headwall due to the heave (expansion) of the mass-frozen Boston blue clay. While soil extraction has never previously been attempted (intentionally) from below the water table, previous experience with drilling and working in the sensitive silty sand and silts below the water table, including ground loss observed during tieback drilling performed on the adjacent East Side Access contract section, suggested that soil extraction may actually be readily achieved at this site if needed.

Where performed previously, the under-excavation or soil extraction has been performed with a counter-rotating outer casing and inner continuous flight auger. Because this work at the East Side Access site would have to be implemented horizontally, with a drill situated below the water table, methods had to be modified. A specially-designed inner tool would be operated from within a casing advanced to the specified location. The tool was fitted with several vanes and flutes, that when advanced out beyond the end of the casing, would regulate the volume and particle size of material entering the casing. Soil extraction would cease when the tool was pulled back into the casing. The groundwater pressure and natural tendency for the material to run would be relied upon for the removal of soil. Multi-fluid jet grout rods would permit flushing of the soil from the casing as needed.

MEASURES IMPLEMENTED TO DATE

Ultimately, soil extraction during freezing was not required since only an inch of actual heave occurred. At the time of writing, mining, installation of waterproofing (Figure 9), and installation of structural ring girders is complete. Concreting of the tunnel is underway. Because proper precautions were taken with the design and use of blowout preventers, compensation grouting was not required during the system installation. At the time of this paper, work is still underway and the freeze has not been thawed. Compensation grouting will be performed then, but not to the extent that was anticipated because the heave observed was not to the extent that was anticipated.

CONCLUSIONS

The proactive design and implementation of a set of ground movement control measures and ground improvement techniques were developed to reduce the risk of

Figure 9. Completed tunnel with waterproofing installed. Note freeze visible around the edge.

adverse impacts due to settlement and heave during SEM tunneling under Northern Boulevard and the active transit systems. The rigorous controls implemented during drilling generally limited the magnitude of any ground loss during grout and freeze pipe installation and therefore eliminated the need for an initial phase of compensation grouting. Actual heave observed was also less than anticipated and therefore the heave mitigation procedures were not required The field trial program was successfully performed for the compensation grouting design developed for the site. It is currently anticipated that compensation grouting will be performed as needed during the thawing process and the ground has been pre-conditioned and grout pipes are in place and available. However the amount of grouting required is uncertain at this time' since the heave observed was generally less than anticipated.

COMPENSATION GROUTING AT FLORENCE HSR TUNNEL

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ABSTRACT

Florence HSR tunnel underpasses the densely urbanized area of Florence in Italy. TBM/EPB excavation is executed in soft soil and, in particular next to the southern portal, under a shallow cover: 6m to 8m of sand and clay separate the tunnel from the foundations of 2 masonry buildings and a steel bridge. Compensation grouting resulted as the best approach to preserve structures serviceability. The paper describes compensation grouting activities performed, from design to execution. The monitoring and data processing system is also detailed. Together with appropriate real-time post-processing strategies, it allowed a punctual control and guidance of the grouting activities.

FLORENCE HSR PROJECT

The tunnel to be executed is part of the European high speed train network towards Rome. The underground works consist of: 6.5 km double tunnel excavated with an EPB TBM; a northern portal in Riffredi area; a southern portal at Campo di Marte (which is also the TBM launching pit); and a new underground central station in Belfiore area (Figure 1). Excavations techniques employed comprise mechanized and conventional tunnel excavation, cut and cover and deep excavation (Italferr 2012). Florence soil is characterized by soft clays and sands. The water table lays above the tunnel crown for most of the track. The tunnel under passes more than 150 buildings, many of which can be regarded as historical buildings, with a cover between 6 m to 20 m.

Passive and active protection measures have been designed to guard existing buildings, bridges and rails. In particular, compensation grouting has been foreseen in the southern part of the tunnel, area Ponte al Pino, where two buildings are going to be under passed with a cover between 6 m to 8 m (Figure 1), and also at about 3 km in the north of the southern portal to protect the ancient Fortezza Da Basso.

Compensation grouting consists in grouting a controlled amount of mixture at a controlled pressure. The technique allows to consistently reduce or completely avoid settlements induced by excavation on superstructures. Grouting is performed by means of Tube A Manchettes (TAMs) also named Sleeved Port Grout Pipes (SPGP) located at a suitable distance from superstructure and tunnel face. Compensation grouting activities are subdivided in two principal phases: a “pre-treatment” performed prior to tunneling to permeate the area and avoid loss of time and energy during the next phase; “concurrent grouting” performed during excavation advance when relevant settlements are measured. To perform compensation grouting, a suitable monitoring system has to be installed on the superstructure to read structure’s settlements (Henn Raymond 1996).

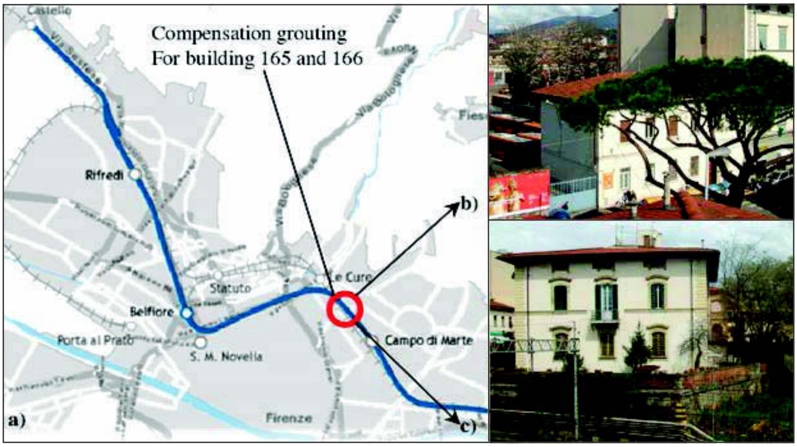

Figure 1. Florence project highlights: (a) plan view; (b) building 165; (c) building 166

Table 1. Relevant building risk classes according to Boscarding and Cording 1989

Class	Description	Cracks width
3–Moderate	Cutting out and patching might be required, doors and windows sticking, possible damage to utility services, water tightness possibly impaired	5 to 15 mm
4–Severe	Removal and replacement of sections of wall might be required, doors and windows frames distorted, floor slopes, walls lean or bulge noticeably, utility service disrupted	15 to 25 mm

Compensation Grouting Activities at Ponte al Pino Area

The area is located at the southern end of the tunnel and is the scenario of the compensation grouting activities described in the paper. The area is named after the bridge ("Ponte") crossing the existing railway in proximity of an ancient pine tree (Pino). Two buildings in this area require special mitigation activities, i.e., compensation grouting, hereafter named building 165 and building 166. The buildings are particularly sensitive for two main reasons: excavation cover is particularly small; and excavation volume control is particularly difficult to optimize at such a short distance from the TBM/EPB launching pit. Buildings 165 and 166 are 2 stories masonry buildings with strips and pads footings at –1 m and –3 m from ground level, respectively. Building risk assessment assigned risk class 3 and 4 (according to Boscarding and Cording 1989, Table 1) to building 165 and 166, respectively, given an excavation volume loss of 1%.

Compensation grouting is performed from two shafts, shaft 5 and shaft 6 within an active area having less than 50m radius. Grouting is executed in a fluvial clay deposit composed by a lower layer (thickness variable from 3 to 5 m) of clay with silt, sand and gravel with a permeability of 10^{-4} m/s, and an upper level (about 4 m thick) of claily silt with a limited permeability of about 10^{-8} m/s.

Grouting mixes, monitoring system and the overall operability have been tested on a test field located southern of shaft 5 and composed by two concrete plates (plate 1 6 m × 6 m × 0.5 m and plate 2 8 m × 8 m × 0.5 m) realized at ground level. An overall layout of the entire area is shown in Figure 2.

The monitoring system installed on the buildings and on the test field comprises: 3D targets installed on the facades of the buildings automatically read by a robotic total station; hydrostatic level circuits installed at ground level and in the basement of the buildings. Hydrostatic cells layout is shown in Figure 3.

DESIGN

The local settlements induced by excavation and the effects of compensation grouting have been in-depth studied by means of 3D FEM software (MIDAS GTS 2008). The numerical study aimed at showing the effects which some key features have on compensation procedures. The study reproduced excavation advance simulating the EPB shield, surface settlement and ground reaction to compensation grouting.

The results obtained from the numeric model represented a valid reference to esteem, for the simulated boundary condition, the influence of grouting on superficial subsidence field. Furthermore, the study defined an analytic procedure to design compensation grouting with regard to mixture quantity and quality, procedures and sequences to be applied for execution and defined a meaningful performance parameter to be used for design.

The analyses investigated the behavior with and without compensation grouting and considering different grouting strategies.

In the analyses without compensation grouting, settlement domain has been studied analytically using Peck formula (Peck 1969). 2D and 3D fem models have been tuned on those results varying soil constitutive models to assess numerical models reliability. Furthermore, 3D analyses modeled in detail excavation advance, simulating each advance step (1.5 m) and the overpressure on the cutter-head to be able to capture the dynamicity of the process. An example of 2D and 3D models and settlements comparison is shown in Figure 4.

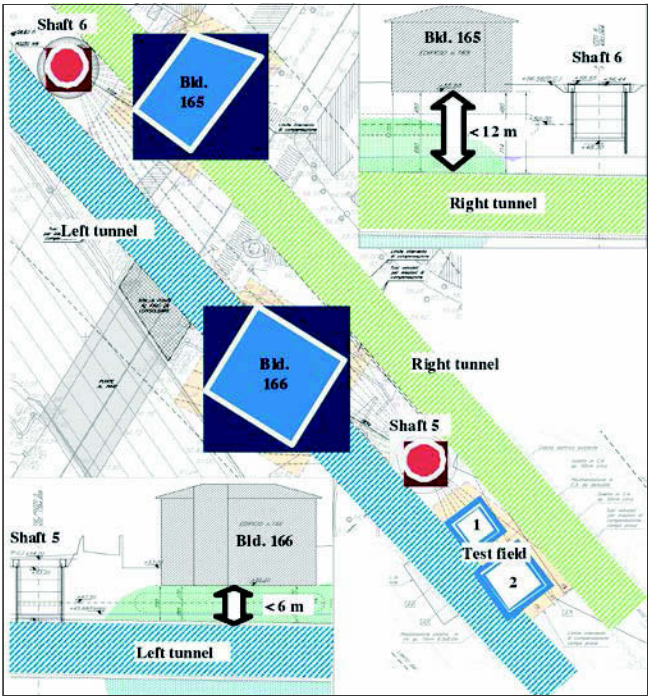

Figure 2. Buildings and compensation grouting shafts in Ponte al Pino area

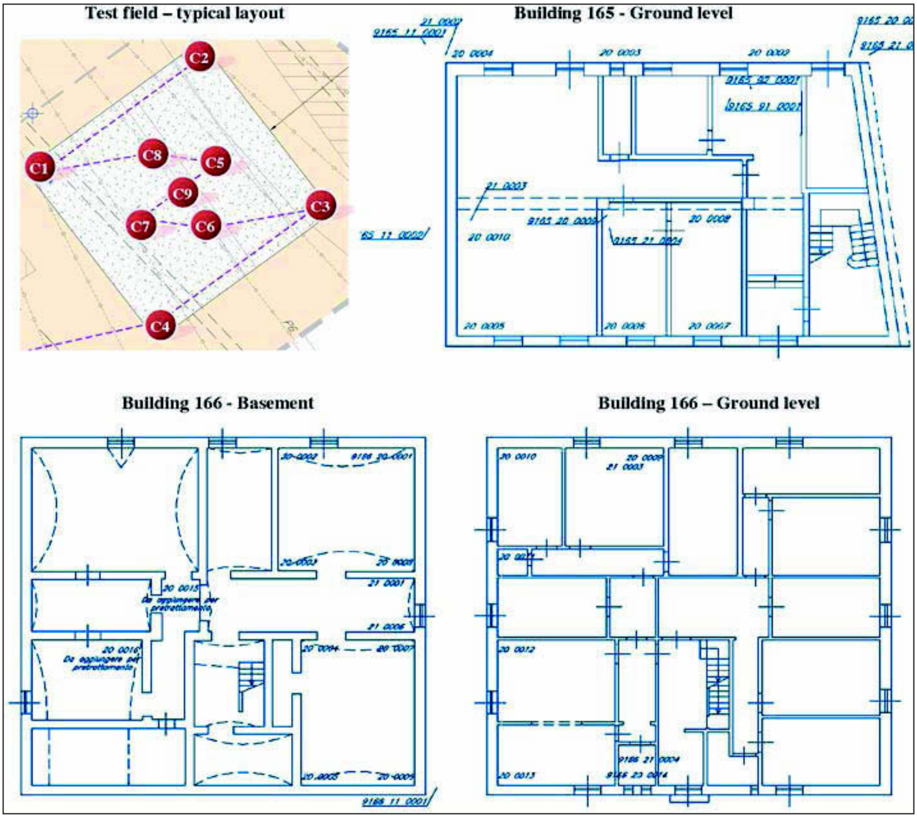

Figure 3. Monitoring system: hydrostatic levels on building 165 and 166 and typical test field layout

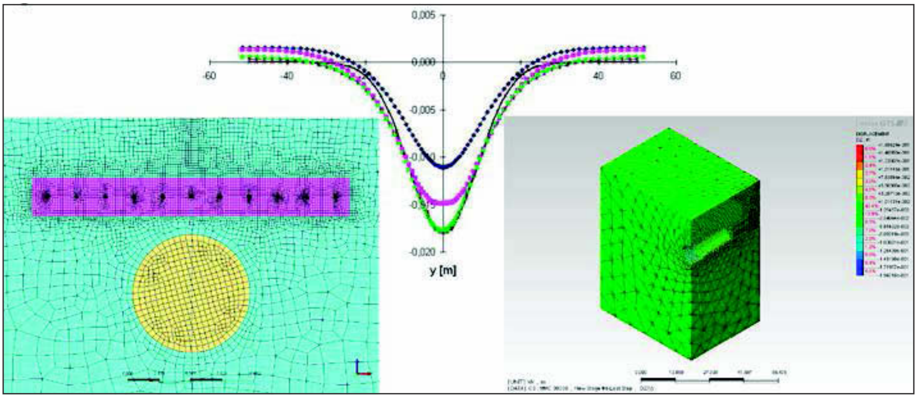

Figure 4. Numerical models geometries and 2D settlement curves comparison

Compensation grouting effect has been afterwards analyzed considering the case with pre-treatment of the area and the case without pre-treatment. In case of pre-treatment, the analysis has been performed assuming different Young modulus of the pre-treated area in order to verify the effect on the final settlement of the quality of the pre-treatment material (expressed in terms of stiffness of the pre-treated area). The simulation also considered several cases with a different grouting strategy, varying the number and the order of grouting ports simultaneously activated. A comparison of deformed shape with and without compensation grouting is shown in Figure 5.

Numerical simulations pointed out that pre-treatment material shall not require relevant stiffness characteristics. Grouting execution strategy effect proved that the larger the number of simultaneous injections, the higher the compensation grouting performance. Figure 6 compares the results obtained for building 166 injecting all grouting ports at a time, 1 injection at a time from central grouting port outwards and 2 simultaneous injections from central grouting ports outwards. Finally, 3D modeling allowed to verify the influence of the compensation strategy in the longitudinal direction, taking into consideration the effect of the non-injection area located at the rear of the TBM shield (not to damage concrete lining). Compensation grouting efficiency resulted slightly higher in the 3D model (Figure 6).

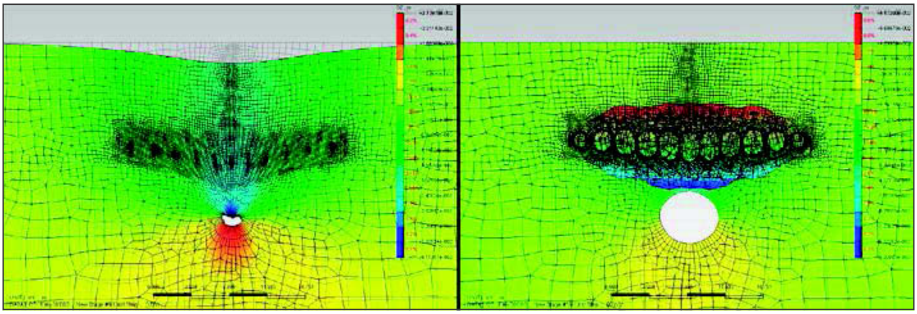

Figure 5. Comparison of settlements with and without compensation grouting

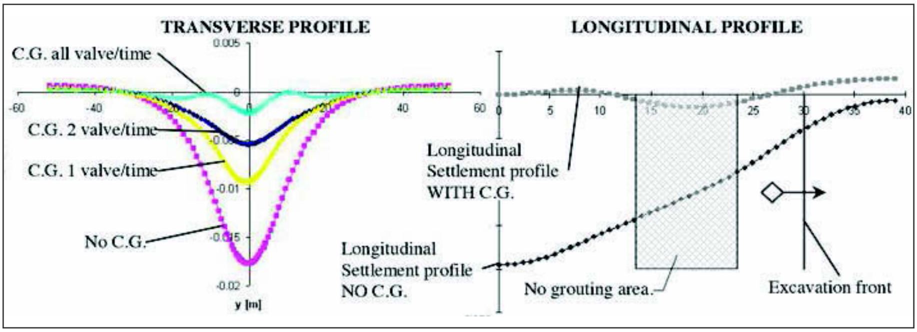

Figure 6. Influence of grouting strategy and 3D effects in compensation grouting

SOFTWARE TOOLS

Compensation grouting activities have been performed taking advantage of dedicated software based on the latest web technologies. In Florence the software architecture is composed by a monitoring platform dealing with monitoring instruments, a software controlling grouting equipment and a compensation grouting suite mastering the data flow and assessing the grouting volume and location.

Monitoring Platform

Monitoring platform principal tasks are:

- Gather data from monitoring equipment
- Provide a convenient representation of structure settlements/heave and distortions
- Store monitoring data

A peculiarity of the monitoring platform developed by the authors for Florence project is its ability to calculate in real time important parameters derived from monitoring readings. Parameters like distortions and maximum deflection ratio are of vital importance for buildings' structural health (Burland et all 2001). They provide a direct measure of the potential shear and bending effects acting on relevant portions of the structure. These parameters are not point-wise information but derive from a set of settlements read along a building structural alignment and, therefore, cannot directly be read with a monitoring instrument.

The monitoring platform gathers settlements data along structural alignments as soon as they are forwarded by the instruments and calculates distortions and maximum deflection ratio making them available in real time to the compensation grouting suite allowing for a direct supervision of both settlements and distortions.

Compensation Grouting Suite

Compensation grouting suite is the hearth of compensation grouting activities. The writers developed the suite to provide the required support for the activities on site. Its principal tasks are:

- Real time data retrieval from the monitoring platform
- Real time evaluation of structural health (check on settlements and distortions)
- Definition of which grouting port has to be activated and grouting volumes for each port (Grouting strategy)
- Automatic communication of the grouting strategy to grouting control system
- Retrieval of actual grouting volumes, pressures and grouting ports as executed on site
- Compensation grouting efficiency update
- Relevant parameters storage

The peculiarity of the compensation grouting suite developed by the authors lays in its ability to provide a real time support for defining the compensation grouting strategy when the team requires it. When threshold values are exceeded, a series of non-uniform grouting injections have to be performed on a precise number of grouting ports to restore building's structural health (in principle restoring building original layout). Which grouting ports have to be activated and the amount of grout for each port has to be defined very quickly for an effective compensation grouting. The compensation grouting suite does it automatically, communicating the strategy directly to the grouting station (the strategy is checked and approved by engineers supervising the activities).

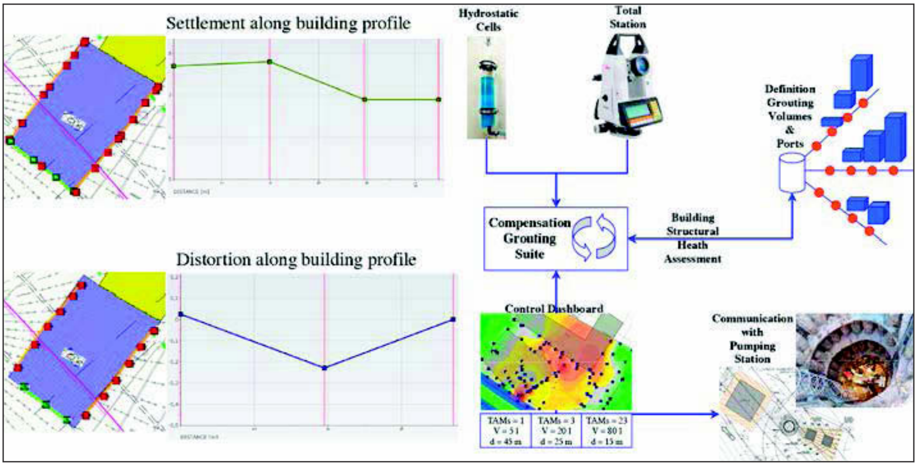

Figure 7. Settlement and distortion representation and compensation grouting suite workflow

The suite localizes the settlements retrieved from the monitoring system and activates the grouting ports belonging to that particular area. Monitoring points and grouting ports layout do not coincide on a one-to-one basis as monitoring points are significantly fewer than grouting ports. Therefore, the suite extrapolates the reference settlement value for each grouting port determining the grouting volume required to heave its effective area back to its original level. The grout volume is then calculated considering the Ground Efficiency Factor (GEF) applicable in that area. The GEF is the ratio between the grouted volume and the soil volume increased measured by the monitoring system in a particular area. GEF values for the entire area are continuously updated by the suite at each loop to provide the engineering supervising and confirming the suggested GEF values with the latest information available (Figure 7).

APPLICATIONS

Compensation grouting activities performed so far in Florence HSR project comprise test field pre-treatment and concurrent grouting simulation, and buildings 165 and 166 pre-treatment.

Test Field

Pre-treatment goal was to uniformly heave the test field plates within a range of 3 to 5 mm. Furthermore, a concurrent grouting step has been simulated to check the capacity of the system to actuate a non-uniform displacement field within the largest plate. The non-uniform displacement field was calculated on the basis of the Peck curve for a 1.5 m excavation advance (Figure 10 left).

Pre-treatment strategy was to firstly grout external grouting ports in order to create a confinement perimeter, and then to grout inner grouting ports. Pre-treatment was executed injecting one port at a time. Pre-treatment successful execution is shown in Figure 8 where practically all monitored points present an heave within the target (The hatched areas represent the goal-heave). Only control point 3, in the eastern corner of the plate presents an heave slightly less than 3 mm.

The results of concurrent grouting simulation is shown in Figure 9 and Figure 10 . Goal-heave is given for each control point in the legend and represented in the graph with the hatched area. Grouting strategy was to inject outer TAMs first, proceeding

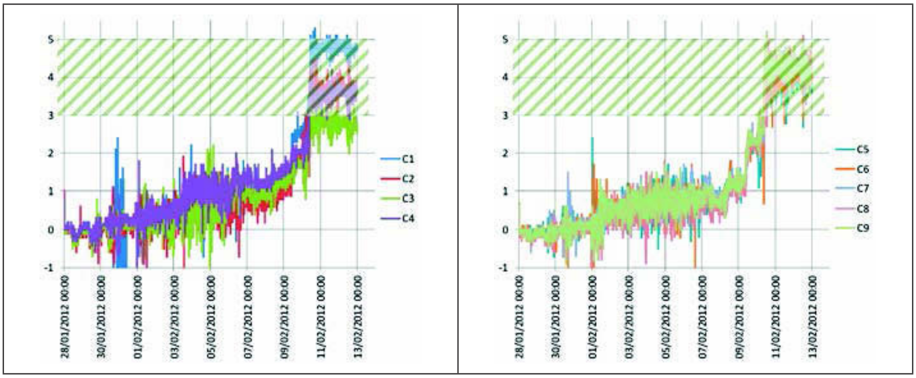

Figure 8. Pre-treatment results for plate 2 on test field

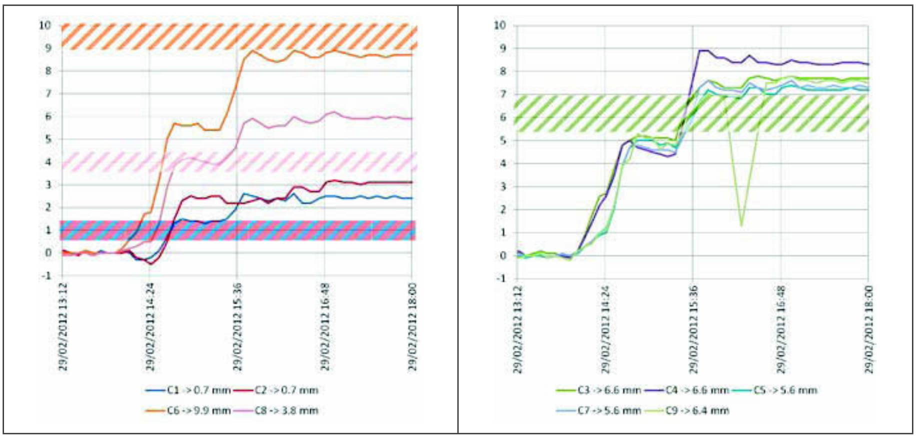

Figure 9. Concurrent grouting simulation: heave measured at control points

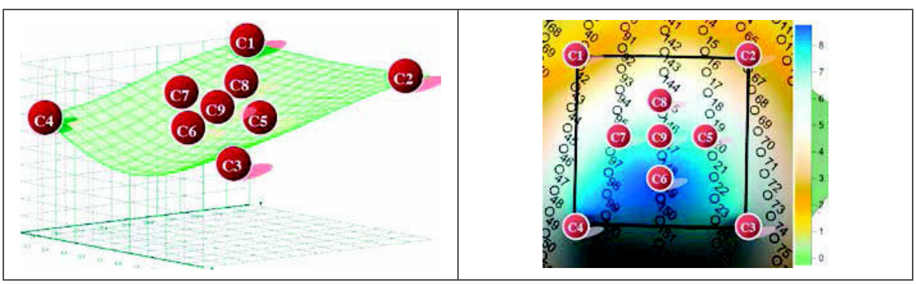

Figure 10. Concurrent grouting simulation: 3D view of the heave measured on plate 2

inwards and injecting 4 ports at a time. The resulting deformed shape obtained is flatter than the goal-shape. In particular northern corners present a higher heave than desired, while peak heave at control point 6 has been practically reached. This is due to the high relative stiffness of the plate (50 cm thick, 8 m × 8 m), which resulted in a flatter deformed shape. Nevertheless, the results obtained can be regarded as successful in reproducing a Peck curve for the simulated advance.

Building 165

Considering the position of building 165 with regard to the tunnel axis, pre-treatment target was a non-uniform heave varying from 7 mm to less than 1 mm. Excavation effects are expected to be larger on the western areas of the building and the pre-treatment strategy has therefore been adjusted to recreate on a smaller, positive scale excavation settlement domain. Pre-treatment strategy employed 4 grouting ports at a time, injecting first the outer area to create a confinement perimeter and proceeding inwards. Control point layout on building 165 are shown in Figure 11. Measured heave is shown in Figure 12 where desired values are hatched in green. An larger heave has been achieved on control point 1, nevertheless pre-treatment activities has been considered successful.

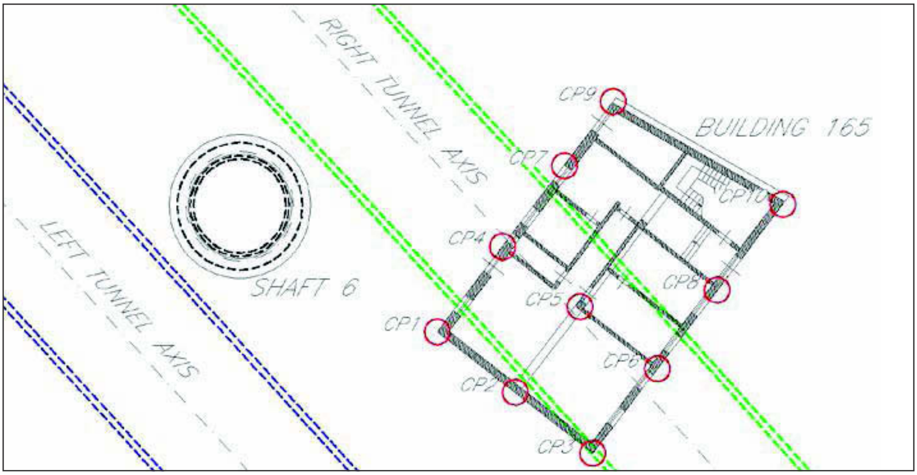

Figure 11. Building 165: control points layout

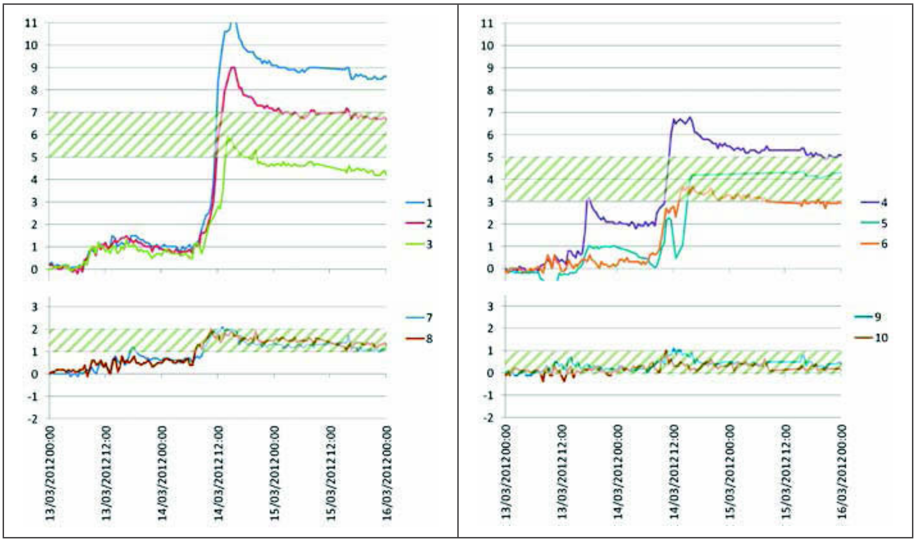

Figure 12. Building 165: pre-treatment results at control points

Building 166

As per the test field, pre-treatment target was to achieve a uniform heave within 3 to 5 mm. Pre-treatment strategy employed 4 grouting ports at a time, injecting first the outer area to create a confinement perimeter and proceeding inwards. Control points layout is shown in Figure 13. Measurements are shown in Figure 14. Slightly smaller heave (2.5 mm) is measured on control point 4 and 13. Nevertheless, the results have been considered sufficient to prove the good efficiency of compensation grouting system in that area.

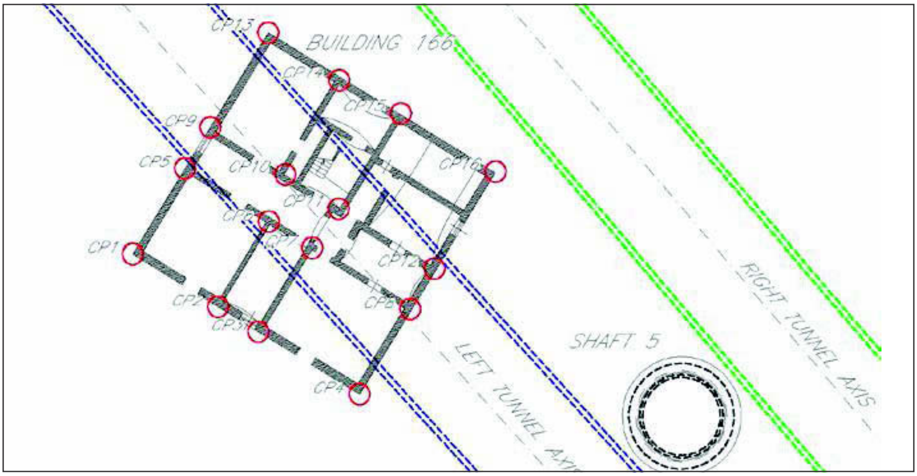

Figure 13. Building 166: control points layout

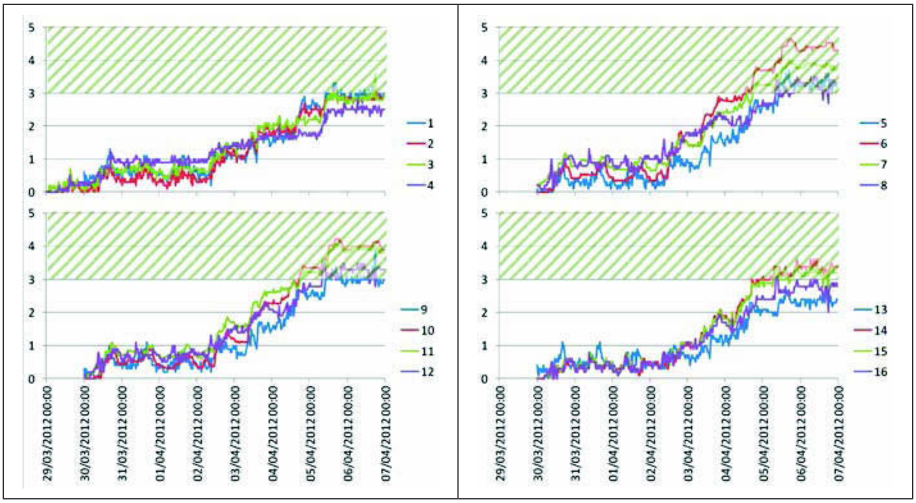

Figure 14. Building 166: pre-treatment results at control points

CONCLUSIONS

The paper described compensation grouting activities performed so far in Florence HSR project. Compensation grouting resulted as the best solution to ensure structural health of two masonry building laying at short distance from TBM launching pit and having a net cover varying from 6 to 12 m. The paper described the in-depth numerical simulations performed to assess compensation grouting efficiency and to define the influence of compensation grouting key factors. The paper described the results of compensation grouting activities performed in a test field area where the monitoring system, the compensation grouting suite, the grouting mixtures and the overall efficiency of the system have been intensively proven. After pre-treatment, a settlement field was reproduced, corresponding to the Peck formula displacement field obtained for a 1.5 m excavation advance. Results obtained were successful and the paper described the results of the following pre-treatment activities performed for the two masonry buildings.

The success of the compensation grouting activities performed could not be obtained without the support of particular software tools developed by the authors which enabled a real time access to monitoring data, simultaneous building risk assessment and direct calculations of grouting volumes and locations which were directly transferred to the grouting control system and injection team.

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DOES THE END JUSTIFY THE MEANS—THE COST OF GROUTING AND THE BENEFIT TO OWNERS

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Jack Raymer ▪ Jacobs Engineering

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ABSTRACT

Long term groundwater inflow into a finished hard rock tunnel represents a permanent, fixed cost for the Owner. This cost continues long after the Contractor and Construction Management teams have left and moved on to other projects. These long term costs associated with operating the tunnel are often neglected during design and construction. Modified contact grouting has been used in the Atlanta area to substantially reduce groundwater inflow into hard rock tunnels. This paper looks at the unit rate method of payment for grouting and a typical cost per gallon of reduction associated with lowering the overall resultant groundwater inflow in to the tunnel.

TUNNELS LEAK

Hard rock sewer tunnels constructed below the water table are going to leak. They are going to leak because there is typically a head difference between the outside and the inside of the tunnel, the ground is permeable, and because the tunnel lining cannot be made absolutely impermeable. The question is how much the Owner is willing to spend to minimize the leakage. Owners, with their tunnel design engineers, should establish reasonable, cost-effective criteria regarding the allowable rate of leakage.

Issues influencing the Owner's decisions regarding leakage include the ongoing cost to manage infiltrating groundwater and the effect on local groundwater hydrology. While it is tempting to say "no leakage allowed," the goal of "no leakage" can be quite expensive and is probably unattainable. As long as there is a substantial head difference between the inside of the tunnel and the outside, groundwater will tend to find its way through even "impermeable membranes." The leakage will occur at the joints, seams and other imperfections. These imperfections are inevitable over the length of a long tunnel.

Modified contact grouting has been used successfully in the Atlanta area for the past eight years to minimize leakage into hard-rock sewer tunnels and meet the Owners' infiltration criteria. The technical details of modified contact grouting were described in Bedell and others (2011). In summary, modified contact grouting combines traditional contact grouting with consolidation grouting in a single program that, ideally, makes one pass through the tunnel. Modified contact grouting is cost-effective and can be bid in a manner that rewards both production efficiency and quality while allowing the Owner to control costs. The first purpose of this paper is to discuss the costs of modified contact grouting relative to the tangible benefits of water control. The second purpose is to discuss an effective, proven strategy for bidding this work and controlling costs.

THE OLD WAYS

Sewer tunnels with cast-in-place concrete linings typically received two separate rounds of grouting. The first round was traditional contact grouting, and the second was consolidation grouting. In many cases, the consolidation grouting was done as an afterthought, once it was found that the tunnel lining, as installed, leaked a lot more than a ring of perfect concrete should leak in an ideal world.

The goal of contact grouting was to fill the annular void between the concrete lining and the excavated rock wall; the purpose was to distribute structural loads evenly around the lining and to fill up any areas where the concrete was thin. Pressures were typically kept low so as not to damage the concrete lining. Groundwater control was not a serious consideration, because the pressures were too low to push back groundwater and plug water-bearing voids. Payment for traditional contact grouting was often included in the price of the lining, since it was reasoned that the time and materials needed for grouting would be lower if the concrete were placed carefully and annular voids were small. Likewise, the contractor would be penalized with higher grouting costs if the concrete was sloppy and there were large voids.

The purpose of consolidation grouting was to reduce groundwater infiltration through the tunnel lining. The method was to drill through the lining and into the water-bearing zones in the rock mass, and then pump in high-pressure grout to push the water away from the tunnel and plug the pore spaces, which are typically open joints in the rock mass. This approach is difficult, however, because the water-bearing fractures are covered by the lining and cannot be seen, and because the low-pressure contact grouting has made the flow system around the tunnel concrete very complex. The point where water finds a crack in the concrete and leaks into the tunnel is not necessarily close to the point where the rock fracture leaks water into the annulus. As a result, the grouting crew would have to rely on geotechnical map reports made during mining to give them clues to where the water-bearing joints might be. More often, the grouting crew would simply drill a pattern of holes and hope, by luck, to intercept some joints.

The time and materials required for consolidation grouting were typically considered a function of the ground conditions and, as such, were seen as the full responsibility of the Owner to bear. This leads to time and materials payments that can become a hemorrhage of cash over which the Owner has little control, except to capitulate. The programs tend to be long in duration without predefined completion dates or reasonable quantity estimates. The programs are considered complete when either the inflow criterion is met, or more typically, when either the schedule or money is exhausted. Since consolidation grouting typically occurs at the end of the project, it may be the only thing keeping the project from going online. All of these factors can put great pressure on the Owners, forcing them to choose between more and more spending or accepting an inferior project.

UNIT RATE

The unit rate method of payment has been discussed many times in the grouting and tunneling industry. While there has been agreement in the principle of shifting towards a unit rate method of payment, there have remained numerous questions. When the authors decided to move towards a unit rate method of payment involving grouting, two primary goals were identified. The first was to reward the Contractor for being efficient; the second was to keep the Owner in control of the grouting program, since it is the Owner and no one else who has to live with the results over the long term.

The unit-rate structure for modified contact grouting, as it has been recently implemented, includes five bid items. The Engineer estimates the expected quantities during the design, but the actual quantities can vary. All costs for modified contact grouting must be covered in these five items.

1. *Grout Holes.* The Contractor bids a unit rate for each grout hole drilled during the grouting program. The cost of the grout holes includes all labor and equipment necessary to drill the holes, make and break connections, install and recover the packers, and patch the holes. The number of grout holes is moderately variable, depending on the amount and distribution of water encountered in the tunnel.
2. *Bags.* The Contractor bids a unit rate for every 94-lb. bag of grout pumped into the ground. Bulk cement, if used, is converted to 94-lb. bag equivalents. Additives and water are incidental and paid for separately. The cost per bag includes all labor and equipment necessary to handle and mix the grout. The number of bags is highly variable depending on the sizes of the annular voids, the amount of panning, and the sizes and quantities of the rock joints.
3. *Grouting Hours*. The Contractor is paid for every hour the grout pump is running, to the nearest minute. This is the primary incentive for the Contractor to be efficient in their grouting operations. If the operation is not well organized, then the grout pump does not run, grouting is not performed, and the Contractor is not paid under this item. This item is moderately variable and depends, in part, on the apertures of the annulus and the rock joints.
4. *Invert Drain Plugs.* This item is particular to modified contact grouting, as described in Bedell and others (2011). The invert plugs are used to isolate sections of the invert drain. This item has low variability, depending mainly on the length of the tunnel.
5. *Set-ups.* The Contractor is paid every time the equipment is moved down the tunnel and set up in a new location. Setup locations must be approved by the grouting inspector. This has low variability and depends mainly on the length of the tunnel.

The unit-rate method is easier to manage and rewards contractors for being efficient. There are no arguments over whether or not hours were legitimate. The only hours in the contract are when grout is actually being pumped into the ground. The other quantities are all tangible and can be directly counted.

Grout is paid by the dry weight of cement, not by the volume of the mix. This eliminates the temptation to pump large volumes of watery mix in order to run up the grout quantities. If the mix is made too thick, however, the holes will refuse early and the grouting hours will be too low. The contractor is rewarded for putting as much cement as possible into the ground, which is the goal of the consolidation grouting.

The high pressures used in modified contact grouting severely test the structural soundness of the concrete lining. If the lining is thin and has large voids, then the grouting pressures will cause the lining to fail (a blowout is typical) and the contractor will have to repair it. The cost of repairing a failed section of lining vastly outweighs any additional money the contractor might make by pumping more grout to fill those voids. Modified contact grouting places a strong incentive on the contractor to pour high-quality concrete the first time around.

BENEFIT

Grouting tunnels is expensive but the benefits are well worth the cost. One tangible benefit is the money saved by not having to run infiltrated water through a wastewater treatment plant. For the three Atlanta projects considered, the net annual savings from the grouting program, based on treatment costs alone, ranges from $141,000 per year to $273,000 per year (see Table 1).

The values in Table 1 are based on the following assumptions and qualifications. (1) Inflows were rigorously measured using weirs, except that the inflow at the end of

Table 1. Summary of cost (treatment and grouting) for three Atlanta tunnel projects

	Units	Nancy Creek	South Cobb	South River
Year completed		2005	2013 (proj.)	2011
Length of tunnel	ft	43,700	29,100	9,076
Target inflow criterion	gpm	238	252	46
Inflow at the end of mining	gpm	1700	1200	600
Inflow at the end of concrete	gpm	1190	840	420
Inflow at the end of MCG	gpm	230	152	23
Gallons reduced by MCG	gpm	960	688	397
Annual reduction in water vol.	MG	504.6	361.6	208.7
Cost per gpm of wastewater reduced		$ 525.6	$ 525.6	$ 525.6
Cost to treat wastewater	/MG	$ 1,000	$ 1,000	$ 1,000
Annual treatment savings	/yr	$ 504,576	$ 361,613	$ 208,663
Capital cost of MCG program		$(4,000,000)	$(2,800,000)	$(1,800,000)
Annualized cost of MCG program	/yr	$ 231,320	$ 161,924	$ 104,094
Annual realized savings from MCG	/yr	$ 273,256	$ 199,689	$ 104,569
Annual return on investment		118.13%	123.32%	100.46%

MCG = modified contact grouting

concrete lining was taken as 70 percent of the inflow at the end of mining, based on experience. (2) The cost to treat wastewater is typical of the operational costs for the Atlanta metropolitan area. (3) Annual treatment savings is the annual reduction in water volume times the cost to treat wastewater. (4) The annualized cost of the MCG program is based on the annuity of the capital cost, given a 30 year period and an interest rate of 4 percent. The break-even interest rates range from 12.3 to 13.3 percent. (5) The annual savings is the difference between the annual treatment savings and the annualized cost of the MCG program. (6) The annual return on investment was calculated as the annual savings divided by the annualized cost.

The capital costs for the Nancy Creek tunnel were higher for a few reasons. First, the tunnel was longer. Second, due to the construction and consent decree schedules, the Nancy Creek tunnel required multiple shifts and six total crews. Third, grouting was paid on a time and materials basis, as it was the first time that modified contact grouting had been used on a project and unit prices were not sought in the bid process. The cost per gpm reduced was comparable on all three projects. While the costs were comparable, the unit-rate method afforded the Contractor the opportunity to earn a good profit by doing the work well.

CONCLUSION

Modified contact grouting has been used in Metro Atlanta for the past eight years to reduce the amount of groundwater infiltration into hard rock tunnels. We have moved from the time and material basis of payment to a unit rate method of payment. The unit-rate basis is easier to manage, produces excellent results, and reward the Contractor for being efficient.

The annual return on investment from MCG is quite large, and is over 100 percent for a 30-year payout period and today's low interest rates. There is no question that high-quality grouting is worth the cost and that MCG is the best way to get that quality accomplished.

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SUPPORTING MEASURES FOR URBAN TUNNELING

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Daniel Rendell ▪ Keller Getec

INTRODUCTION

The improvement of urban infrastructure, particularly the building of efficient links for public commuter transportation, can often only be achieved by means of tunnels due to the lack of available space due to urban development. The following paper shows how intelligent use of software linked with monitoring, tunneling and compensation grouting along with geotechnical instrumentation provides the stakeholders with confidence to proceed with complicated underground construction activities in an urban environment.

Frequently, sensitive buildings are located within the zone of influence of tunnels, spray concrete lined caverns and large excavations, which have to be monitored for movement during the construction process. The interaction between the existing buildings, subsurface construction processes and existing below ground infrastructure are extremely complex. Definitive methods of measuring and reporting in a clear and precise manner are now a standard within the Instrumentation and Monitoring industry.

Process-integrated supporting measures are designed to prevent damage occurring to buildings or to compensate for removal of materials at depth during tunnel construction. The examples in this paper relate to compensation grouting.

Settlement measurement systems can be accomplished nowadays with a multitude of sensors reporting data in real and near real time. This enables the end user to be well informed of the construction process and to able to plan, control and efficiently manage the works with confidence.

Should a suitable settlement control variable be lacking in the selected TBM driving technology, due to technical or ground conditions then the use of other measures to complement the process is recommended. Since the 1980s compensation grouting has been classed as an autonomous supporting measure. The controlled variables for compensation grouting to attain movements or heave in the ground are: grout volume and grouting pressure. They are controlled in precise ways using advanced technology that to work needs the integration of measured data on site.

Efficient systems are available for face supporting during tunnel construction and compensation measures in areas where there are more complicated subsurface works. These systems permit a precise analysis on the effects of building influenced by the drive in real time through systematic storage, evaluation and engineering presentation of measurement values and controlled variables. The safety and risk reduction of a tunnel drive and the associated auxiliary structures has increased significantly through the application of these intelligent and adaptable control mechanisms.

METHODS

Sensitive buildings are frequently located within the zone of influence of tunnels and spray concrete lined caverns, which have to be monitored for movement during the tunnel construction. This is usually because of the Stakeholders commitments to the public or sensitive structures identified during structural surveys. The interactions between the existing building, subsurface construction processes and groundwater are extremely complex and require a high degree of design input during the planning stage.

The predicted deformations of buildings are assessed by simulating foundations to the excavation induced ground movements that are likely to occur during tunnelling (not taking into account the influence of the structure's stiffness). This approach reflects the rapid development of settlements that occur in the short term before any adjustment response can be developed within the structure. Burland (1995) and Mair et al. (1996) calculated the tensile strains in the building and also provided direction in ways to interpret these in terms of damage in "degrees of severity" which are expressed in categories ranging from "negligible" to "very severe."

The general, assessments are broken down into three stages and the level of detail or further assessment and protection measures are the result of the exercise. The initial assessment includes for ground surface settlement contours drawn, and if the predicted settlement of the building is less than 10mm or the buildings are not subject to slopes in excess of 1 in 500, the risk of damage is negligible and the process goes no further. The second stage assessment involves the maximum tensile strain in the building being calculated and a damage category assigned. This will be a conservative assessment because the building strains are based on the "green field" ground movement predictions. In reality, the actual movements may be reduced by the stiffness of the building. The final stage is a detailed evaluation and is undertaken when moderate damage has been predicted in the previous stages. The tunnelling sequence and other three dimensional aspects as well as the soil structure relationship is considered. Tunnelling works at depth usually produce smooth settlement trough shapes and the angular distortions occurring at the foundations of structures can be reasonably predicted by the methods discussed above. However consideration should sometimes be given to unusual geological conditions that may result in localised differential settlements and damage to the structure.

It is at this stage when protective measures would be considered for structures in the moderate and higher damaged categories.

Protective measures include the use of controlled operation of the TBM (tunnel boring machine) in sensitive areas or the use of grouting methods to stabilise and heave ground in the area of the works that are affected by the constraints above.

Methods of measuring and reporting in a clear and precise manner are now a prerequisite within the instrumentation and monitoring and grouting industry. As a result, intelligent, adaptive supporting and compensation measures, keeping with the observation method principle are all the more important.

PROCESS-INTEGRATED SUPPORTING MEASURES

Process integrated supporting measures are designed to prevent damage occurring to buildings or to compensate for removal of materials at depth during tunnel construction. The examples in this paper relate to compensation grouting and TBM control measures.

Controlling variables must be undertaken based on measured values, which have to be collated with a continuous process control from a real time measurement system and evaluated and visualised. Real or near time measurement systems can be accomplished nowadays with a multitude of sensors that are installed in ground, on structures or within machinery. This enables the end users to be well informed of the construction process and to be able to plan, control and efficiently manage the works with confidence.

The major items of instrumentation in use on large urban tunneling projects consist of the following:

- Hydrostatic Water Level Systems, (settlement)
- Automatic Total Stations (settlement and tilt)
- Precise level monitoring (settlement & distortion)

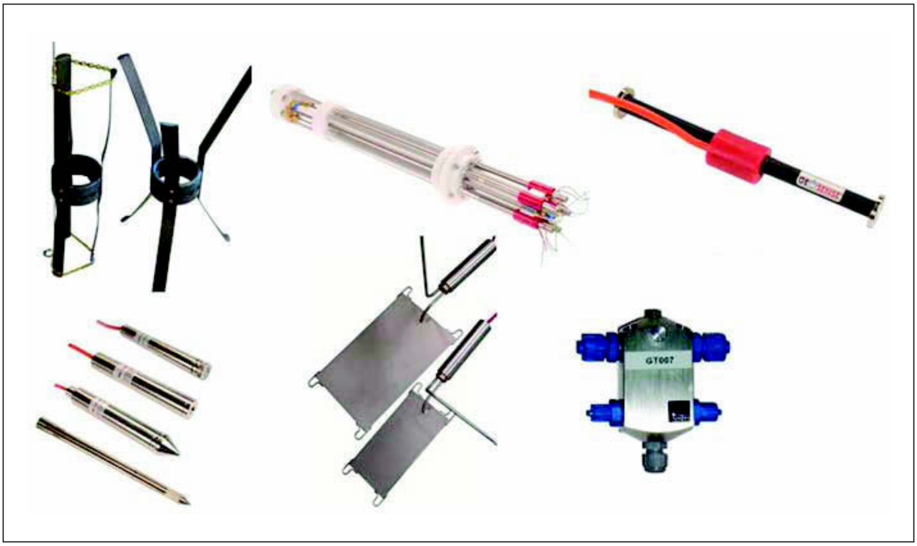

Figure 1. Different sensors in multi channel measurement systems. Left to right magnetic extensometers, multi head rod extensometer, strain gauge, piezometers, pressure cells and hydrostatic level cells. (Images courtesy of Getec and MGS Geosense.)

- Displacement transducers (crack and Sewer monitoring)
- Biaxial tiltmeters (Tilt)
- Strain transducers (structural stresses)
- Radial and Tangential pressure cells (stresses in SCL lining)
- Piezometer (pore water pressures)
- Extensometers (subsurface compression and elongation)
- Horizontal and vertical inclinometers (transversal movements)
- Shape array (movement and vibrations)
- Fibre optics (strain and acceleration)

To enable engineering interpretation of the interactions between the construction and the measured values from the site, the data must be stored, evaluated and visualized using a suitable software system. For mechanised tunnel drives there are various software programmes that provide key TBM parameters to the site teams to evaluate as construction proceeds and to discuss in shift reviews. The key features are:

- Face Pressure
- Grouting volumes and pressure
- Alignment
- Torque values
- Material excavation
- Total advance time and speed

Getec (from many years working in the mining and compensation grouting industry) have developed a monitoring system that links all key items of urban tunneling, which is linked to a database via an open interface that includes the TBM parameters, instrumentation data and compensation grouting data.

All stored values from the three main sources of data are continuously evaluated and displayed in various presentations and process images. The ability to playback data in an archive mode, which can include the various visualistion modes such as contouring, volume loss and positions of excavated tunnel face variables along with all measurement values allows for a data presentation in a more efficient manner.

The spectrum ranges from the complete overview of the tunnel drive by way of the presentation of the current extract for the shield operator right up to detailed presentations and time variation plots for individual measurement values. Visulisation in the form of maps, images and graphs can be provided, which facilitate orientation within the project and the allocation of measured and controlled variables to sensors or affected buildings. For visualisation purposes an automatic 3D-CAD core is used. This CAD core permits a 3D underground model for the entire project to be set up, in which the geological and hydrogeological conditions can be taken into account alongside the geometry.

The position of the TBM is shown real time on the GIS system. The software can display influence zones that reflect the forward and rear monitoring zones and assign these as priority readings to provide a clear understanding of settlement influences. The zone adjusts automatically corresponding to the depth of the TBM. The volume loss calculation is shown automatically from the assigned points. The data from the TBM can be managed to reflect the requirements in shield pressure and back pressure grouting, as well as groundwater levels. The TBM operator has a module that provides the TBM data in a visual format for the required parameters against measured. It also

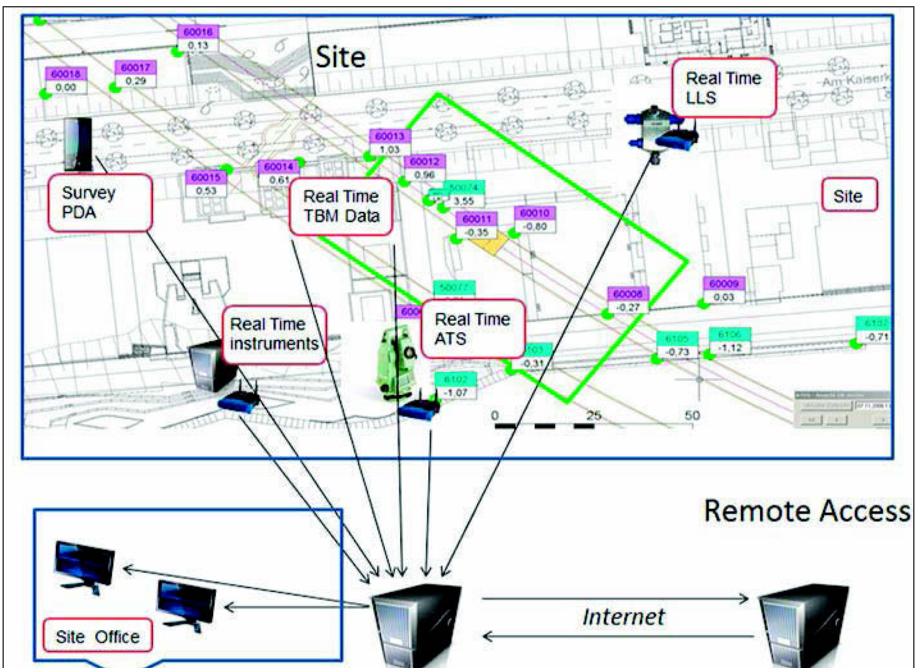

Figure 2. Overview of data flow on site using various geotechnical instruments

features the monitoring information at both the surface and sub surface instrumentation with the same interaction for users. The influence of the TBM and monitoring data is dynamic and real time. This enables the TBM engineers to assess the whole sphere of works immediately.

COMPENSATION GROUTING CONTROL

Should a suitable control variable be lacking in the selected TBM driving technology or large underground caverns are to be constructed using SCL methods, then the use of other supporting measures to complement these processes is recommended.

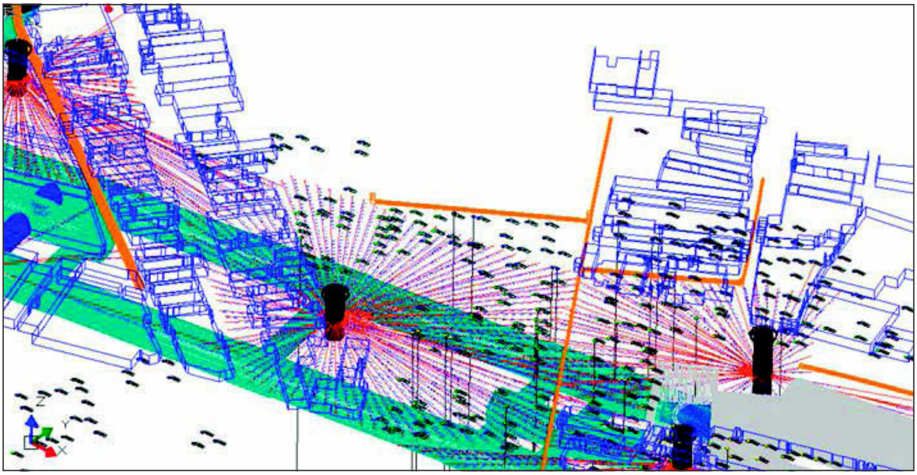

Figure 3. 3D image of grouting arrays and water levels cells in basements in central London

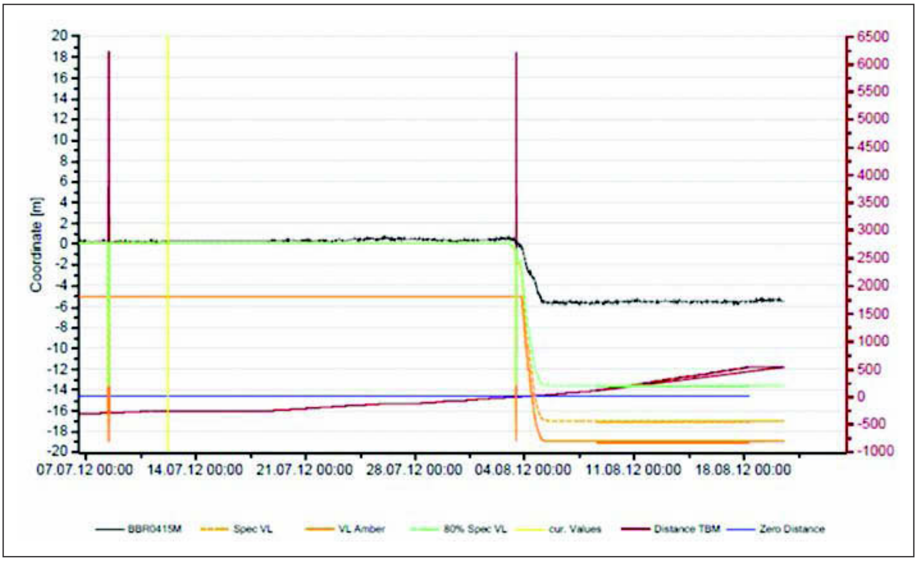

Figure 4. Volume loss curve for TBM works with projected theoretical

Figure 5. TBM operators view of surface data alongside the automatic zone of influence indicator

Compensation grouting is classed as an autonomous supporting measure. The controlled variables for compensation grouting to attain movements or heave in the ground are: grout volume and grouting pressure.

Compensation grouting involves installing horizontal steel tubes (Tube a Manchette or TaM) under structures at depth in clay material. The steel tube is installed in sections and has a series of holes, pre-drilled uniformly at set centres covered with a hard wearing rubber sleeve. The grout is introduced into the clay via an inflatable packer which is pushed down the (TaM) to the required sleeve location. Once in position, the packer is inflated at both ends and the grout is pumped at pressure down the grout line, into the void that is caused between the two inflated packers. The sleeve, with the pressure of the grout, lifts and allows the grout to disperse into the clay. Clay naturally has fissures, and these filled with grout under pressure (causing a jacking wedge which in turn manifests to the surface) that causes heave. The grout is typically the same strength of the surrounding soil and its composition can be changed to suit the site requirements. Repeat injections are required over a long period of time and therefore the grout mix and durability of the steel TaMs need consideration. Once each grouting phase is complete, the system is usually washed out with water.

Compensation grouting works are generally carried out in three phases, first there is the installation phase, which requires the usually horizontal boreholes to be drilled. The depths of compensation grouting boreholes are usually deep enough to avoid utility services, but there are risks of hitting piled foundations. Over large areas of treatment, it is usual to have a series of shafts and grout fans that are designed to interlace, allowing a full coverage.

It is during this phase of works that the settlement rates are calculated due to the effects of drilling. In clay materials, casing is frequently used to advance the borehole to mitigate the settlement and heave. The use of water flush in drilling can lead to heaving of structures without control and the use of uncased auger drilling can cause settlement. These elements can be controlled using monitoring instruments installed within the basements of the structures that are being drilled under. The most convenient method of measurement is a hydrostatic water level cell system.

Once the drilling has been completed, the boreholes are surveyed using a maxibore probe. This device provides the 3D location of the borehole measured at one

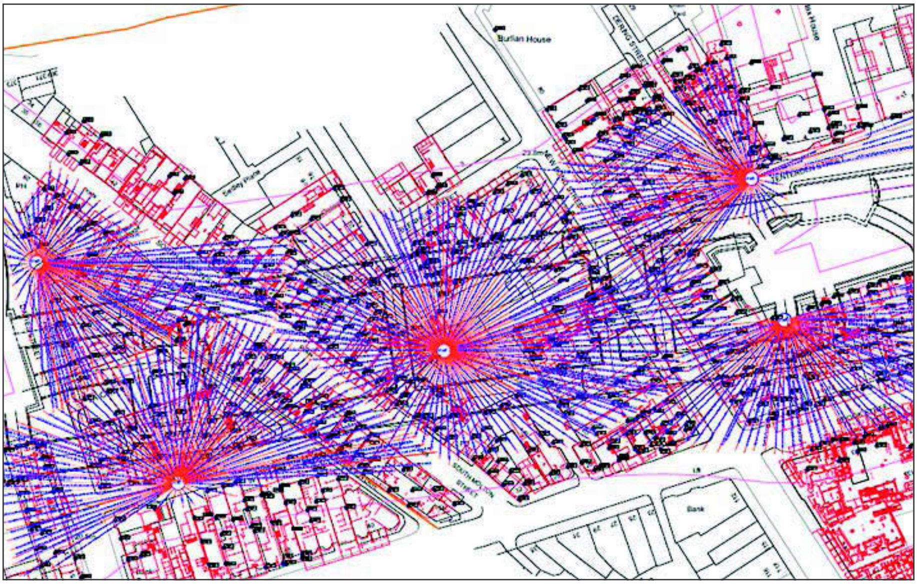

Figure 6. Compensation grouting fan layout in Mayfair London. The treatment area is approx 20,000m^2.

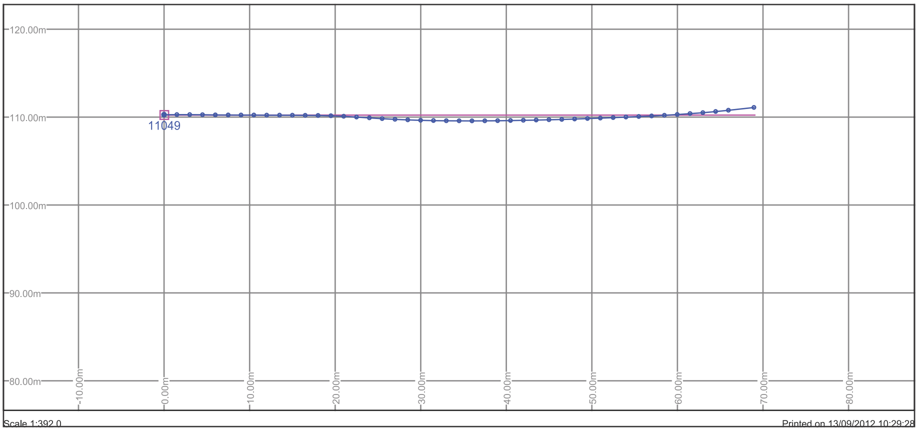

Figure 7. As-built survey of TAM borehole over 70m

metre intervals. The data files are loaded in the grout control software and can be viewed spatially if required, The TaM sleeves are also named and positioned.

Pre-treatment is the second phase where grouting is carried out to cause a degree of heave across the site. The use of real-time monitoring from instruments located in the structures basements are critical for the success of this phase. The grouting instructions are prepared within the grout control software and sent to the grouting containers which are located at the various grout shafts. The grouting containers are self-contained units that houses the agitators, pump and control software. The monitoring data is downloaded to a terminal in the container so that information is available immediately. Once the grouting phase is complete the data is stored and the shift engineers can retrieve this information and compare with monitoring instruments. The data can be presented in various views (including 3D) and can also can be reviewed over periods in a playback facility to analyse the effects of other construction activities.

The third phase of grouting is termed as concurrent grouting. The grouting is strategically placed in areas that are not restricted due to TBM progress or excavation. The monitoring systems employed need to be able to calculate slopes and distortion in a clear and quick manner for assessment by grouting engineers. The grouting is usually carried out in small volumes at very regular intervals rather than allowing settlement to occur and then jack the ground back to original levels.

DATA MANAGEMENT

Managing and visualizing data is key to delivering a successful project. The effectiveness of instrumentation is also a consideration at design stage. The Instrumentation and Monitoring industry is constantly changing, with more accurate and functional products being released constantly. An adaptive software system must be in place to adapt to these changes and data formats.

Monitoring systems are usually placed prior to works commencing and require care in their design, so that additions to the system and layout can be accommodated later. Clear and precise data is required to manage the works correctly.

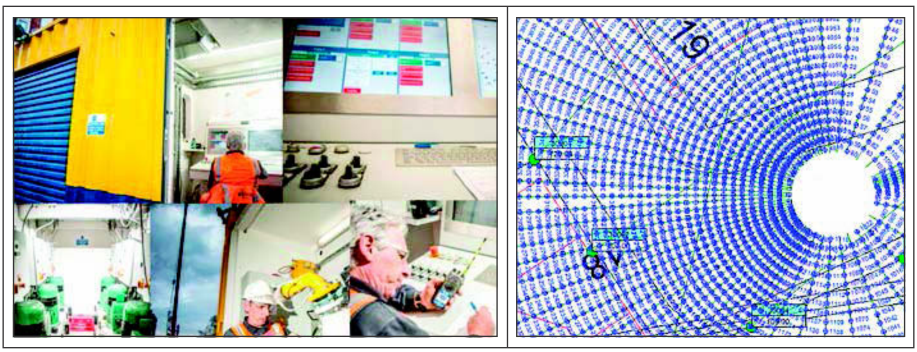

Figure 8. View of layout of grouting container and grouting fans with as built data and sleeve naming

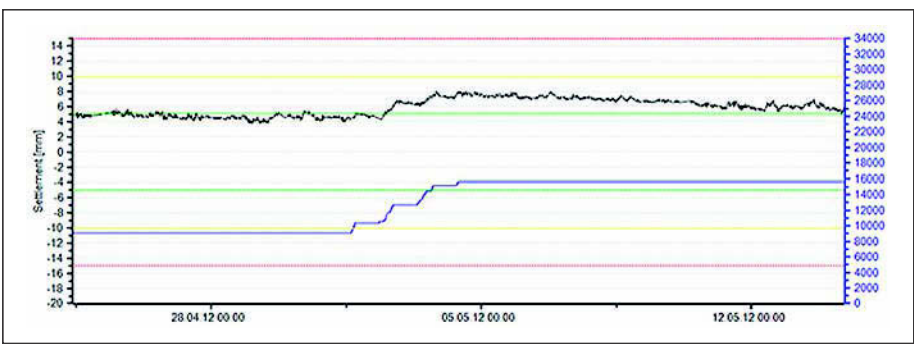

Figure 9. Graph showing the cumulative volume of grout (secondary axis) with controlled heave of structure measured with hydrostatic levelling cell

The monitoring system has to consider these following three items when specified.

- Speed and capacity
- Visualisation
- Functionality

The speed of the monitoring database is paramount when dealing with TBM, monitoring and grouting data. Web based software when dealing with small projects. The use of server based platforms is recommended when total systems users exceed 10 or more. A majority of users will concentrate on viewing their particular work area only and not the whole project, therefore provision to allow partial areas of the site to be available for users is an important consideration. gtcVisual users can split their work areas down to sub locations within the site. The use of preset graphical presentations involving points that are particular to buildings or construction progress can be preset at the beginning of the project to ensure continuity within the site team. This also allows third parties to receive the same data in formats that are easy to read.

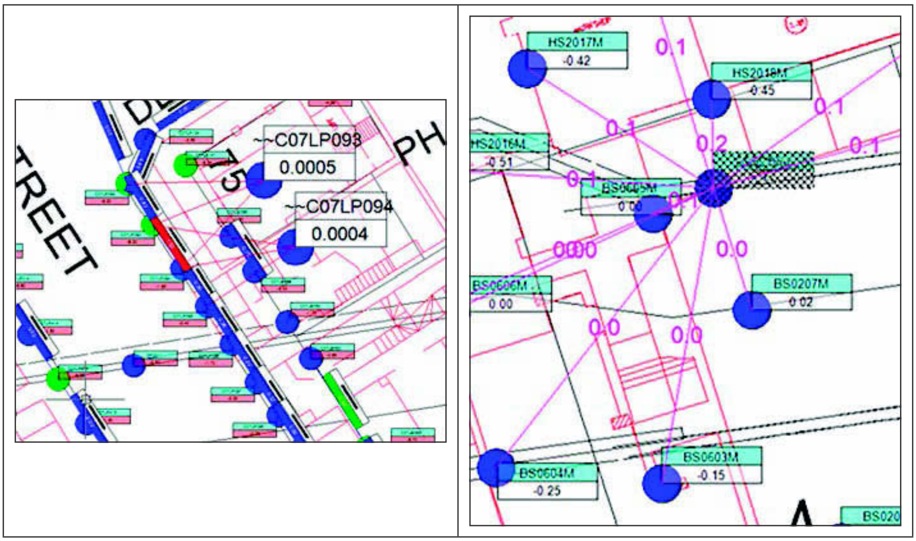

Figure 10. Slopes and deflection ratios shown in real time

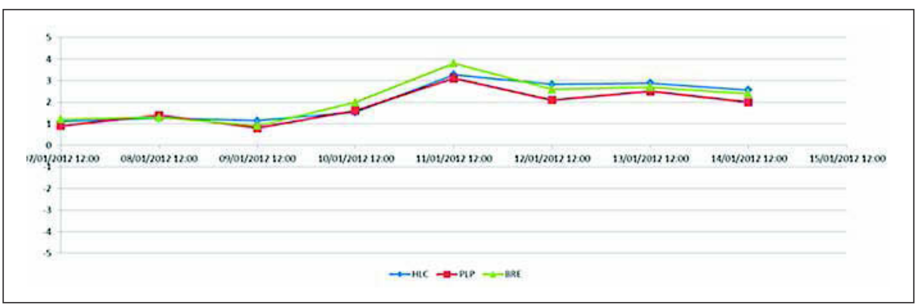

Figure 11. Three separate and different measuring methods show correlation over time with compensation grouting: hydrostatic levels, precise level points, and building points

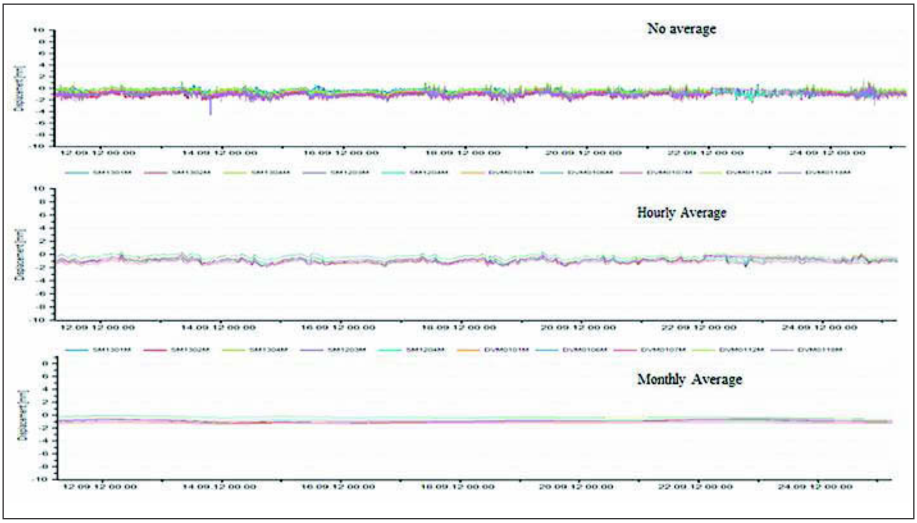

Figure 12. Various data visualisations

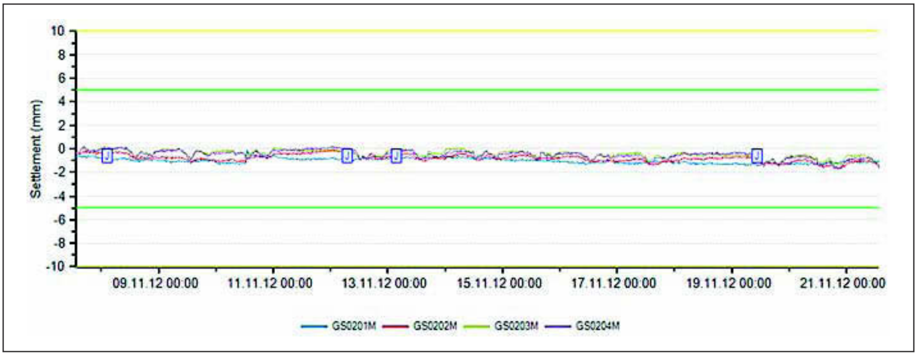

Figure 13. Journal entries visualized on data points for a single measuring point

Loading data can slow down server resources and therefore consideration should be made to provide facilities to show data from various time stamps or data formats on loading, to speed up operations.

The storage capacity of servers and the backup procedures for data are important factors to consider on large projects. Measurement rates increase and decrease during a projects lifecycle as does the grouting requirements. As per the observational method, a contingency plan and actual resources have to be available to cope for the unexpected. Data transmission for large areas of monitoring must also be considered. Signal strength (if using the 3G data networks) and network coverage in peak hours have direct consequences on critical works.

Analysis of data in tunneling works requires the work site reports or an indication of activities associated with data changes to be available on one location. The site report feature should include all elements of work including excavation data of other structures such as station boxes. The use of the observational method requires data to be analyzed within design software and therefore logical download facilities should be included to enable engineers to take raw data from the monitoring database.

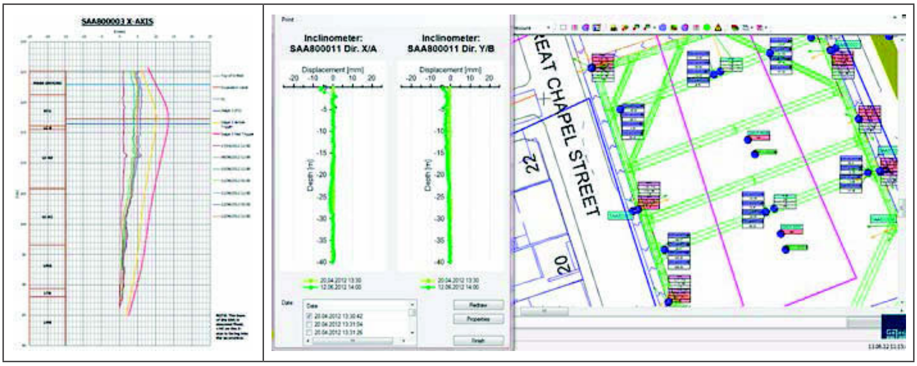

Figure 14. Representation of shape array data and in place inclinometer data in gtcVisual. Note the inclusion of design limit alarms, prop and excavation levels, and soil profiles.

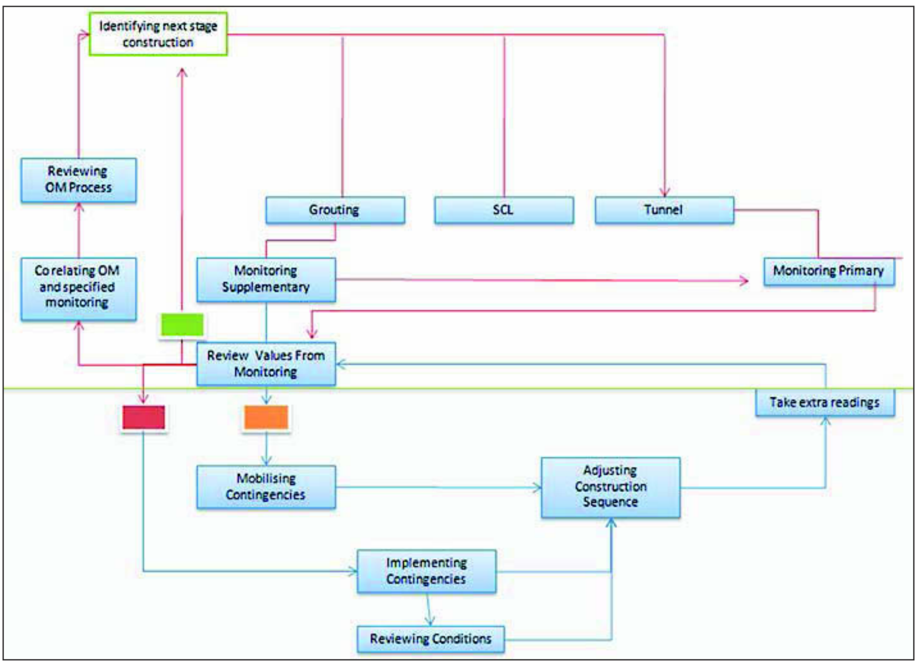

Figure 15. Schematic to show method of site control for tunnelling and compensation grouting works

Alarms are commonly used in monitoring software and can be extremely useful if managed correctly. Monitoring software requires stringent controls to the management of alarms. The alarms should typically include 6 levels of response and the ability to notify the end users of breaches via modern communication methods. There should always be a documented method of reacting to alarms and each breach should trigger members of the designated site teams to react in a prescribed way. Triggers are usually determined by design parameters or stakeholders requirements dependent on the sensitivity of their assets.

SUMMARY

Efficient systems are available for face support during tunnel construction and compensation grouting measures in areas where there are more complicated subsurface works. These systems permit a precise analysis on the effects of buildings influenced by the tunnel or excavation in real time through systematic storage, evaluation and engineering presentation of measurement values and other controlled variables The safety and risk reduction of a tunnel drive and the support of associated auxiliary structures has increased significantly through the application of these intelligent and adaptable control mechanisms.

Communication between tunnelling, grouting and monitoring teams is vital. The real-time feedback to the monitoring software and dynamic review from engineers on site is the key factor in establishing a controlled approach to tunnelling, grouting, monitoring, construction safety and progress.

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INNOVATION IN ANNULAR GROUTING AT THE EUCLID CREEK TUNNEL, CLEVELAND, OHIO

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Jarrett E. Carlson ■ McNally/Kiewit ECT Joint Venture

Brian Garrod ■ Hatch Mott MacDonald, LLC

Doug Gabriel ■ Northeast Ohio Regional Sewer District

ABSTRACT

Void grouting behind concrete segmental tunnel linings is a relatively straightforward task in conjunction with an Earth Pressure Balance or Slurry Tunnel Boring Machine in soft ground. However, there are times when a segmental lining is called for in a rock tunnel, with or without the need for such a TBM. This paper focuses on the Euclid Creek Tunnel in Cleveland, Ohio, and explains: what drove the selection of this lining system; the challenges that exist when using this system in a rock tunnel with an open TBM; and how the challenges were dealt with during construction.

INTRODUCTION

Void grouting behind concrete segmental tunnel linings is a relatively straightforward task in conjunction with an Earth Pressure Balance or Slurry Tunnel Boring Machine in soft ground. However, there are times when a segmental lining is called for in a rock tunnel, with or without the need for such a pressurized face TBM. This paper focuses on the Northeast Ohio Regional Sewer District's (NEORSD's) Euclid Creek Tunnel (ECT) in Cleveland, Ohio, and explains: what drove the selection of this lining system; the challenges that exist when using this system in a rock tunnel with an open TBM; and how the challenges were dealt with during construction.

The ECT design team was led by Hatch Mott MacDonald, LLC, with major sub AECOM providing hydraulic and near-surface structure design. The construction is being undertaken by the McNally/Kiewit Joint Venture. Concrete Systems, Inc. is building the concrete segments for the tunnel lining at a local Hanson plant in Macedonia Ohio.

PROJECT DESCRIPTION AND BACKGROUND

Project Context

Since the early 1970s, the NEORSD has been responsible for wastewater treatment facilities and interceptor sewers in the greater Cleveland Metropolitan Area. Having invested almost one billion dollars on tunnel systems to clean up Lake Erie and its tributary streams since 1972, the NEORSD has now entered into a consent decree with the United States and Ohio Environmental Protection Agencies (OEPA/US EPA) and the United States Department of Justice (USDOJ) in 2011 to implement a roughly $3 billion, 25-year program to further control combined sewer overflows (CSO) in the area. When finished, the annual combined sewer overflows (CSO's) into Lake Erie will have been reduced by approximately 9 billion gallons.

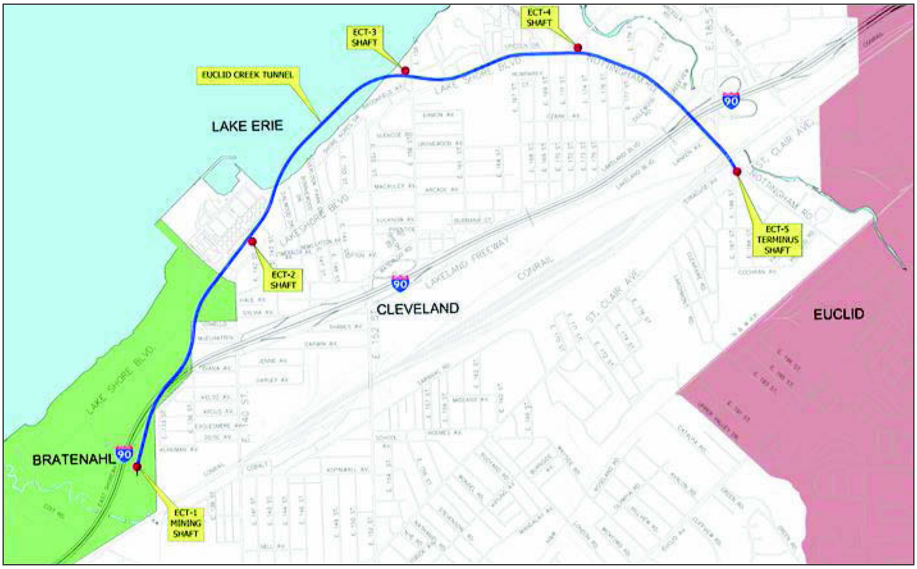

Figure 1. Euclid Creek tunnel (ECT) project layout

The Euclid Creek Storage Tunnel (ECT) Project is the first project under the consent decree, and one of seven large diameter tunnels that will be designed and constructed between now and 2035. Construction of this approximately $200-million-dollar project is being undertaken by the McNally/Kiewit Joint Venture (M/K JV). Work on ECT began in early 2011 and is scheduled to be complete in 2015. At the time of this writing, the tunnel drive is over 20% complete with TBM hole-through expected in October of 2013.

Project Layout

The 24-foot internal-diameter ECT tunnel traverses 18,050 feet at average depths of 200 feet beneath eastern Cleveland and the Village of Bratenahl, as shown on Figure 1. Horizontal alignment selection for the tunnel was a result of balancing the need to pick up flows at several locations (coincident with shafts ECT 2-4 in Figure 1) while staying in public rights-of-way to the greatest extent practical. The ECT-1 mining shaft and the ECT-5 terminus shaft are in line with the tunnel, while the ECT 2, 3 and 4 shafts are off-line, connected to the tunnel with short adits. Baffle-type flow-drop structures will be constructed within all shafts except the ECT-1 mining shaft. That shaft will serve as the junction between the ECT Tunnel and the future Dugway Storage Tunnel (DST) which will continue approximately 3 miles to the south. A low spot exists in the vicinity of shaft ECT-1 with respect to the ECT and DST tunnels. An adit from this low point will connect the tunnel system to the future Tunnel Dewatering Pump Station (not shown) which is presently under construction. From there, flows will be pumped/conveyed to the existing Easterly Wastewater Treatment Plant.

Project Geology

The ECT alignment generally lies along the gently sloping Lake Plain coastal terrace. Topographically, the area consists of a relatively flat to very gently northwesterly-sloping land surface, which has been incised by geologically recent river erosion. The surficial deposits in the vicinity of the ECT alignment vary from about 15 to 100-ft thick and consist primarily of a series of glaciolacustrine deposits and glacial till overlain by localized fill. The project area is underlain by late Devonian-aged (360 to 408 million years before present) shale bedrock which was incised by river erosion and glaciated in the past, and since in-filled with glacial till, glaciolacustrine deposits, alluvial deposits, and surficial fill materials. Sand and gravel layers (outwash/fluvial deposits) are often interbedded with the glacial till and glaciolacustrine deposits.

For the western half of the tunnel alignment the depth to the bedrock, known locally as Chagrin Shale, is about 90 to 100-ft below the existing ground surface. The bedrock surface rises to the east and is only 15-ft deep at the eastern end of the alignment near Shaft ECT-5. The Chagrin Shale is a weak to medium strong, gray, argillaceous shale with strengths typically in the 5,000 to 15,000 psi range. Strengths can be considerably lower, particularly near the soil/rock interface due to weathering and stress relief.

Natural gases, including hydrogen sulfide and methane, are known to be present in the Chagrin Shale. Quantities of methane are often enough to create 'geysers' when drilling exploration holes, and some holes have vented for days or weeks although the latter is relatively rare. One past NEORSD tunnel project experienced a lengthy shutdown due to methane.

TUNNEL BORING MACHINE (TBM) REQUIREMENTS

Regardless of the lining type, a fully shielded TBM was specified by the Owner and the Design Team over a main-beam rock TBM. This would allow for safe installation of the circular lining support within the safety of the shield. Since the TBM would be in sound rock with several diameters of rock cover, and since large, sustained water infiltration is not typical in the Chagrin Shale, there was no need to specify an Earth Pressure Balance-type TBM. The specification was silent on whether a single or double-shielded TBM was required; this was left to the contractor. (There are pros and cons of single vs. double shield TBMs in this formation, but the discussion of that is beyond the scope of this paper.) The M/K JV supplied a single shield Herrenknecht Open Rock TBM.

BENEFITS AND RISK REDUCTION DUE TO ONE PASS LINING IN SHALE

Past NEORSD large-diameter, TBM-bored tunnels in Chagrin Shale have used a two-pass tunnel lining system consisting of steel ribs and lagging (timber and steel mat) followed by cast-in-place concrete. While these tunnels have been installed successfully, several risks were always inherent in the use of such a system related to crown and sidewall overbreak, invert degradation, gas, and the need for labor-intensive cleaning between ribs prior to concrete placement.

The use of a one-pass system had been proposed to mitigate the fore-mentioned risks in the past, but typically the cost of a one-pass system had been shown to be prohibitive when compared to the two-pass. However, the overbreak issue had been getting worse as the NEORSD moved to larger and larger diameter tunnels. In addition, a recently completed tunnel in a similar shale formation at twice the diameter of ECT experienced great difficulty controlling overbreak (See Figure 2); another factor leading NEORSD to re-consider the use of a one-pass system. The ECT designer again reviewed costs for a one-pass vs two-pass system, but this time using steel fiber in lieu of rebar for the one pass lining. For ECT, it was shown that the steel fiber achieves the strength, toughness and nominal bending capacity required for handling and service loads with a lower amount of steel per cubic yard of concrete compared to rebar-reinforced concrete segments, thereby reducing costs. In addition, fiber does not need the labor required to tie and assemble rebar cages, also saving time and money. Essentially, the use of fiber in the one pass lining made this system cost competitive with a comparable two-pass system, while greatly reducing project risk.

Figure 2. Overbreak issues in recent two-pass tunnel in shale

The following list summarizes some of the benefits/risk-reduction features of the one-pass lining for the ECT project:

1. Reduction of risk due to mining stoppages when encountering gas. A recent NEORSD project was troubled by gas seeping into the tunnel, well behind the TBM. Since the segments are gasketed against external hydrostatic pressures and internal surge pressures, they provide a barrier that will reduce gas entry into the tunnel during construction. Once the lining is grouted in place behind the TBM, gas entry into the tunnel is eliminated or reduced to a level that can be handled by the ventilation system. There will still be a potential for gas at the TBM face as always but the risk of long term shutdown will be significantly reduced over a comparable two-pass system.
2. Reduction of risk due to overbreak. Overbreak occurs in the crown of a shale tunnel for a number of reasons, but in general, it is a combination of in-situ stresses, stress concentrations, gravity, bedding and strength of the shale. There are ways to mitigate this using two-pass systems, but in light of the cost comparison and further risk reduction, the NEORSD and the design team agreed to move forward with concrete segments. With this system, the excavated rock is never seen or exposed in the tunnel, but is immediately supported.
3. Reduction of time to construct. A one-pass lining system is completed as it is mined, therefore there is no need to go back and cast a final concrete liner. Since tunnel construction is typically on the critical path for a project, this can result in an overall reduction to the contract construction schedule. It is estimated that for ECT, the one-pass system will result in a total construction schedule savings of 4 months over the two-pass system. This schedule savings can be taken from the overall contract or left in as float to reduce risk of late contract completion. In the case of ECT, it was left in as float.
4. Increase in quality. Since the segments are produced in a factory and steam cured to a high strength, the quality of the concrete exceeds that of a cast-in-place lining. In addition, segment lining thickness is ensured in a factory setting with no risk of losing thickness in the crown from too little concrete or settling concrete, as sometimes happens with cast-in-place concrete in tunnels. Since the lining is grouted into contact with the surrounding rock, there is also no risk of pouring concrete on a weakened invert and no need to clean

between ribs prior to concrete placement, as there would be for a two-pass system

5. Increase in safety. The nature of the one-pass lining installation means that workers are never exposed to the potential for rock fall. The tunnel is also easier to keep clean when compared to a two-pass system due to the continuous concrete invert and, as just mentioned, not having to laboriously clean between ribs. Lastly, the one pass lining does not allow the invert to degrade as happens in a two-pass lining in shale, where stress relief, water and equipment can combine to turn the invert to mud, causing equipment derailment and lost production.
6. Decreased infiltration/improved performance. While groundwater infiltration during or after construction has not typically been problematic in this formation, the segments provide protection against incidental seepage water into the tunnel compared to the typical two-pass construction. Further, the dense nature of the segments, coupled with the gaskets, should reduce the operational infiltration during the service life of the tunnel compared to cast in place concrete.

MAJOR RESIDUAL RISKS FROM ONE PASS LINING IN SHALE

While the one-pass lining eliminated scores of major and minor risks in the project risk register, at least two major risks remained that would have to be accounted for in the design and construction. The first major residual risk was the risk of grout travel to the tunnel face and/or grouting-in of the TBM. Unlike an EPB TBM, there is no pressurized face, or pressurized annulus for that matter, to provide resistance for the forward travel of annular grout (the grout used to fill the annular void between the back of the segments and the excavated rock surface). Fluid grout would have a tendency to flow forward and into the face of the TBM. If not controlled, the lost grout would result in squatting or ‘ovalling’ of the segments (due to lack of grout confinement), wasted grout, and increased cost of contact grouting. In addition, the grout around the TBM could set, making it difficult to move the TBM forward due to the hardened grout and increased skin friction.

The second major residual risk was the inability to see/deal with overbreak in the crown. With an approximate 6-inch void behind the segments, the rock can ravel/expand into the void. Further, if the rock breaks ahead of and above the TBM, it can be ingested in the face resulting in voids over the TBM. The rock above these voids can further ravel by the time the machine has moved forward and the segments are being installed in the affected area.

THE USE OF TWO-PART ACCELERATED GROUT WITH RAPID GEL TIME

The residual risks discussed above were dealt with in two ways. First and foremost, the designers specified the use of a two part, accelerated grout with a rapid gel time. The “Part A” grout component as used in this paper refers to the flowable, un-accelerated cementitious grout (and other admixtures/components). The “Part B” component refers to the accelerator. The combination of the Part A and Part B components is hereinafter referred to as “bi-component grout.” General requirements of the grout can be summarized as follows:

1. The A Component must not gel within 72 hours of mixing (M/K chose 6 due to means and methods of placement).

2. Mixed A and B components must gel within 30 seconds. The 30 second gel time was selected to allow the grout to flow around and encapsulate the segment ring prior to gelling, and to reduce the chance for forward flow.
3. Strength gain in the short term should be enough to sufficiently constrain segments. Minimum compressive strength at 28 days of 500 psi (since relaxed to 150–200 psi). The initial strength of the gelled grout was not as important as the ability to arrest deflection of the ring under the rings self weight and the weight of any rock loadings.
4. Maximum compressive strength at 28 days of 2,000 psi. (Arbitrary maximum, but strength of this magnitude or higher was not needed, especially with the risk of grouting-in the TBM…).
5. Flow characteristics allowing grout to encapsulate the ring prior to gelling.
6. Maximum 2% water bleed by volume.
7. Grout viscosity that will allow the A component to be pumped 18,000 ft with reasonable pump pressures.
8. Segregation limited so that tunnel delivery line does not silt or build up.
9. Sufficient filling/strength to support the weight of the gantry wheel loads.
10. Grout must be delivered through the tail of the TBM as mining progressed; grouting through the segments was strongly discouraged so that the grouting (once up and running) could be as streamlined and repeatable as practical.

A substantial Test Program was specified requiring the Contractor to drill several holes in each of the first 20 rings (or until such time as the method proved to be working). In addition, convergence monitoring was required (the M/K JV supplied dedicated inclinometer rings fastened to the tunnel walls) to measure segment squat/ovalling. By drilling the holes through the segments, the encapsulation and set of the grout could be checked. When compared with deflection and signs of cracking, the efficacy of the system could be proven or disproven, and methods altered as needed.

In addition to the test program, a program of contact grouting/consolidation grouting was required. The GBR stated "…overbreak in the crown…will be unavoidable…(therefore) contact grouting will require drilling 5 feet behind the lining and filling voids that exist as much as 5 feet behind the lining." In this way the potential for loosened rock and/or voids above the segments could be investigated and mitigated during contact grouting.

GROUT PERFORMANCE DURING CONSTRUCTION

Mix Design

At the onset of the mix design process, a list of necessary requirements was developed in order to determine two critical elements; first, what critical performance objectives needed to be quantified through the mix design testing, and second, what tests could be performed in order to measure and quantify the performance of the tested grout against these critical performance objectives.

The performance objectives were developed based on a combination of contract specified requirements and known additional requirements for construction purposes (Table 1).

It is noted that the test described in ASTM D6910 yields a result that is representative of viscosity, but not an actual measurement of viscosity.

Of all the parameters to test for in the mix design development, the contractor wanted laboratory tests that were simple to execute and easily repeatable in both a laboratory and in tunnel operations. In this manner, the tests could be repeated during the tunnel operation in order to troubleshoot and diagnose problems in real time as

Table 1. Grout requirements

Requirement	Source	Test
A component to remain fluid for 72 hours	Contract	Marsh Funnel Viscosity per ASTM C6910
Mixed A and B components gel in 20–30 seconds	Contract/ Contractor	Timing upon mixing in lab and time during full scale TBM testing
Compressive strength within 500 psi to 2,000 psi	Contract	3"×6" Cylinders tested in accordance to ASTM C39
Early strength sufficient to constrain segmental lining	Contract	Penetrometer testing per ASTM C403
Flowability to encapsulate segmental lining	Contract	Timing upon mixing in lab and time during full scale TBM testing
Maximum 2% water bleed by volume	Contract	ASTM C940
A component viscosity to allow pumping 18,000 ft	Contractor	Marsh Funnel Viscosity per ASTM D6910
Limit segregation in tunnel delivery line	Contractor	Full scale test and close examination of the sample in the bleed test for segregation
Sufficient strength to support TBM gantry wheel loads	Contractor	Penetrometer testing per ASTM C403

they occurred. To simplify the data collection for this purpose, the contractor settled on 6 inch compressive cylinders, Marsh cone funnel, and a penetrometer for the majority and focus of both lab and field testing.

Measuring of the gel time in the field is actually considered by the contractor as of secondary significance. In the field, accelerator is increased until the following criteria are met: The rings exhibit squatting at acceptable levels, grout does not appear through the shield or with the muck, and cured grout can be routinely found fully encapsulating the ring at the 12 o'clock position. When these three criteria are met, the gel time is sufficient for construction requirements. Lab testing of gel time is simply used to verify the chemistry is sufficient in order to obtain a wide range of available gel times 'in the bank' and acceptable for use in the field. Of note, lab testing of the gel times indicated that gelling could occur as quickly as 3 seconds with accelerator rates in excess of 10%.

BASF provided mix design development services in their Beachwood, Ohio facility. The proximity of this facility to the construction site was uniquely helpful. After a search of industry literature and a review of other current projects, some initial mix designs were developed. The Euclid Creek Tunnel required a high 8 hour strength so as to support the gantry wheel loads of 40 psi. This paralleled the requirement for solidity of the annular grout immediately behind the tail shield to provide the necessary confining strength for the segmental lining.

Initial mix designs from other Bi-Component grouts on EPB/Slurry style tunnels did not yield the necessary 8 hour strengths required to support the rings and gantry wheel loads. A series of tests continued by increasing cement content until the grout was achieving the desired result. After the required cement content was determined, additional tests were run to find the most economical fraction of Fly Ash as a component of the mix. Ultimately, about 36 different mixes were batched in the laboratory before determining the typical project mix design. The typical mix design on the project is shown in Table 2.

This mix met all of the Project requirements, including viscosity for pumping over three miles to the TBM, shelf life and interim and final strength requirements.

Laboratory Results Compared to Field Results

There is an inherent difficulty in obtaining field samples of accelerated Bi-Component grout for unconfined compressive strength cylinder testing. The accelerator is added to the grout through an injection port in the tail shield, less than four feet from the end of the tail shield and the annular space. See Figure 3. This injection port is generally obstructed by the last segmental ring constructed, save for the very end of an advance. To obtain representative samples, once per day a one inch diameter hole is drilled through the segmental liner near the end of the tail shield at a clock position near a grout injection port. The accelerated Bi-Component grout extrudes out of the hole in a manner similar to soft serve ice cream, albeit not quite as tasty. The three by six inch cylinder molds can then be filled with gelling grout. This procedure was initially complicated because cylinders would have significant voids. Rodding of the cylinders (per the same ASTM concrete cylinder sampling requirements) nearly eliminates unwanted voids. However, this process has to occur quickly, and some samples simply have to be abandoned. The benefit to this process is it provides compressive strength data which is representative of the annular grout behind the segmental lining. (This would not be an advisable process if the tunnel was under any external hydrostatic pressure, for obvious reasons.) Once the samples are collected, the drilled sampling hole is plugged.

Field results from the annular gap sampling program above typically correlate within 10% for the laboratory testing for 7 day strength results. 28 day results of the field samples tend to fall short those from the laboratory by a wider 30-40% margin. BASF and the Contractor suspect this difference is attributable to the change in accelerator from a base chemical of aluminum sulfate during mix development to sodium silicate used on the TBM. It is expected with sodium silicate, that high early strengths are developed and then strength gain is minimal from 7 to 28 days. The current data supports this rule of thumb for bi-component grouts.

Table 2. Typical ECT 2-Part grout mix design

Requirement	Source
Cement ASTM C150 Type II	790 lbs/CY
Fly Ash ASTM C618 Class F	160 lbs/CY
Bentonite	68 lbs/CY
Water	161 gal/CY
Retarder	51 oz/CY
HRWR	71 oz/CY

Figure 3. Accelerator injection/mixing port inside of the tail can

TBM Configuration

Grout is batched on the surface near the launching shaft with a Häny colloidal mixer serviced by three silos and a pair of plunger style pumps to deliver to the TBM. The grout is piped through a 2½ inch slick line down the shaft and to the rear of the TBM.

Grout is discharged from the slick line to the agitator tanks on the fourth gantry of the trailing gear. These tanks hold sufficient Part A grout for one advance. The tanks are agitated and are discharged from the bottom to each of four progressive cavity pumps. These pumps are controlled by a programmable logic controller (PLC) through both pressure and volume. Each of these pumps has a dedicated delivery line up to the shield. The tail shield incorporates eight each built in delivery lines embedded in the tail shield.

Also, the third gantry contains a tank for the accelerator Part B component. This tank also feeds four progressive cavity pumps, controlled by the same PLC as the grout. Each of these four pumps has a dedicated delivery line up to the shield. The four pairs of two component grout lines are combined through individual ports in the tail shield.

For Bi-Component grouting in the Euclid Creek Tunnel, again with no hydrostatic pressures or confinement provided by the shale, it has been found most efficient to grout through the highest four available grout ports. Pumping at the springline or below in the invert tends to generate high grout pressures in the lower ports. This makes intuitive sense, for what amounts to nothing more than atmospheric pressure behind the lining, the lower grout ports wind up working against the pressure of the fluid grout column created behind the TBM. Also, this grout column is also quickly turning into a gel. Pumping to lower ports causes the pumps to have to overcome static pressures and the gelling action. These lower grout pumps will trip off at pressure, but the grout continues to be placed from the upper ports. Grout from the upper ports has a tendency to find the lower port and proceed to gel/cure inside the lower port. Cleaning is then required.

This approach of pumping to the four available top ports has the advantage of providing the operator of the grout system better information. By using the top ports at lower pressures, it is easier to spot trouble with clogging grout lines by monitoring for increases in grout line pressures beyond the normal pumping pressures. When one starts to show higher pressures, it is starting to clog, it can then be cleaned out during a ring build before the clogging gets worse.

As a final note on the TBM and associated grout hardware, it is critically important to clean the system thoroughly each day. Lines in the tail shield will tend to plug, especially if the volume is over-pumped or overbreak has encroached into the excavation line. This second point is important in the Chagrin Shale, as overbreak and stress relief frequently results in shale resting against the tail shield or the precast concrete segmental lining. When this occurs, the grout finds the path of least resistance, not up into the overbreak void, but forward past the tail shield and seals, or into other grout lines. In short, grout from one port will, over time, start to find its way to another port, and if that port is not flowing then accelerated grout can work its way up static grout ports. A routine program of daily cleaning of the tail shield grout ports has been found sufficient to maintain tunnel progress. As with any grout system, the agitators, pumps and lines need a thorough flushing daily. During longer, more infrequent shutdowns, the agitators are opened up and cleaned. To date the delivery line into the tunnel has not needed any special cleaning beyond a daily flushing with water.

Commissioning and Start Up

Tunneling commenced on 10 August 2012 and went slowly as the TBM continued to be assembled and supporting infrastructure installed. However, by the first week of September, it was noticed from the confirmation drilling through the segments that the grout was not gelling or solidifying and remaining liquid far in excess of the 30 seconds desired. Shortly after this, half a dozen segments in sequential rings exhibited full structural cracking near the crown of the tunnel, usually in the center of one of the larger universal segments. After some diagnosis, it was determined that the accelerator system was not operating properly. The acidity of the aluminum sulfate-based accelerator was not compatible with the stators and rotors of the progressive cavity pumps supplied. The components of the accelerator pumps were replaced, the accelerator system purged and cleaned and the contractor switched to a sodium silicate-based accelerator. Since this change was effected, the grout system has been functioning as needed.

With the chemical compatibility issues sorted and the TBM and batch plant fully commissioned, it is now a rare event that tunneling progress is restricted due to the placement or need for grout. In short, it is now a matter of fact that Bi-Component grout can be used as the sole source to provide confinement on a large diameter segmental lined tunnel. This has significant advantages in that grout ports through the segment are not required. This saves considerably on several issues: First, grouting through the segments requires labor to maintain and monitor the grouting connections as the TBM advances, and second, grout ports are an effective method of ensuring that the tunnel leaks, as they are another flow path for water through the segments.

Monitoring

A drill is mounted on the bridge six rings back from the tail shield. This drill is used to provide inspection holes at a minimum of every ten rings. A video equipped borescope is extended up the hole, and typically full encapsulation of grout up to the 12 o'clock position is routinely discovered. On occasion a void is discovered, a grout line is connected to the finished hole with a packer and the void is addressed during the ring build. This grout line is connected to a single pair of the eight total bi-component grout pumps, one Part A and one Part B. In this manner, contact grouting is addressed as the TBM advances. We have found that the fluid nature of the grout lends itself well to contact grouting, although the accelerator must be reduced if long flows and permeation are needed.

Also, a bespoke system developed by VMT, GmbH has been utilized which includes the attachment of rotational inclinometers to each segment in a ring. This "array" of inclinometers transmits rotational data of the segments and through a series of geometrical calculations, the software yields a graphical representation of ring convergence over time. The arrays are installed periodically and as necessary to monitor the work (more frequently at the beginning, occasionally during normal progress to prove repeatability) and track a ring from inside the tail shield to the first gantry. In typical mining production, with all equipment and hardware functioning, about one half an inch of ring squat is measured. The only time a significant amount of convergence was measured was during the first week of September as described earlier when the accelerator system began to malfunction from accelerator corrosion.

Figure 4 shows the total movement (squat) of a typical ring in the course of the excavation of the next 17 rings. The advantages of the inclinometer system are that it provides real time data for monitoring of ring deformations, and it provides frequent data and recording abilities. The disadvantage is that it takes a fair bit of time and savvy to get an array installed.

CONCLUSIONS

The various technologies discussed in this paper are not new or particularly innovative. For example, bi-component grout and tail-injection have been in use for some time in EPB tunnels in soft ground and rock. What made this project different was that, to the knowledge of the authors, this is the first successful attempt at using tail-injected,

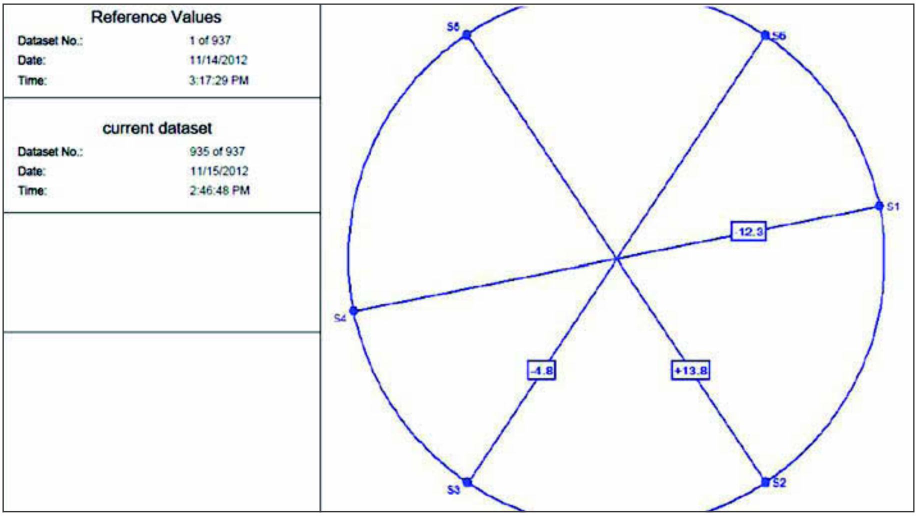

Figure 4. Deformation measurements indicate ring squat during project start up

bi-component grout in rock in an open face machine. The authors would like to note that had significant groundwater infiltration been expected, such a system would not have been specified or attempted (with this type/configuration of TBM) due to the likelihood of grout washout during placement. Fortunately this risk is low in the Chagrin Shale, and not expected to be a significant issue.

The successful implementation of this technology required a lot of collaboration between contractor, machine supplier, grouting specialists and others. Additionally, the procedure is very sensitive to any of a number of changes in mixing method, temperature, etc; all of this at a time when other project start-up issues and TBM commissioning are ongoing. For this reason, the learning curve can be long for all involved, requiring a lot of give-and-take and collaboration between Owner, Engineer and Contractor.

ACKNOWLEDGMENTS

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PART 9

Grouting—Water Control

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PROBING AND GROUTING PREDICTIONS FOR ROCK TUNNELS

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ABSTRACT

The control of groundwater inflows has become a key issue for the environmental impact of new tunnels. As a result, rock tunneling frequently has the requirement to probe ahead, and where the probe-hole inflows exceed a stated trigger, to proceed with a fan of pre-excavation grouted holes. The specification of the probe-hole trigger for pre-excavation grouting is reasonably straightforward. However, predicting how often this probe trigger will be exceeded is very difficult. This paper discusses the methodology for this prediction. The authors have been involved in many of the recent tunnel tenders for the MTR in Hong Kong, and have carried out the detailed design for the probing and grouting in two of the latest rail projects for Hong Kong's South Island Line.

INTRODUCTION

The necessity and methodology for controlling water inflows into hard rock tunnels is well represented in the literature. The collection of papers by the Norwegian Tunnelling Society (2001) provides an introduction to the topic. Once the criteria and methodology have been specified, the cost of meeting the specification must be estimated. This paper discusses ways of calculating some of the quantities which may be involved in this estimating process.

To illustrate the issues and suggested methods of calculation, two projects from Hong Kong's South Island Line are used as example projects.

SOUTH ISLAND LINE (EAST) CONTRACTS 901 AND 904

The MTR's South Island Line (East) is a 7km long medium capacity railway extension connecting the Southern District of Hong Kong Island to the existing MTR network at Admiralty Station (ADM). At ADM Station, the South Island Line (East) (SIL(E)) will connect with the existing Island (ISL) and Tsuen Wan (TWL) Lines plus the future cross harbour section of the Shatin Central Line (SCL).

The SIL(E) includes two stations on Hong Kong Island at Ocean Park (OCP) and Wong Chuck Hang (WCH) plus two more stations on Ap Lei Chau Island at Lei Tung (LET) and South Horizons (SOH). The SIL(E) is constructed in tunnels from ADM to the southside of Hong Kong Island, then viaducts through OCP and WCH station and across to Ap Lei Chau Island then the remaining section back in tunnels through LET and SOH stations. The SIL(E) is being delivered through 5 main construction contracts. The SIL(E) is due for completion by 2015 and the SCL is envisaged to become operational shortly afterwards (Figure 1).

Contract 901

Contract 901 covers the integrated ADM Station including a cut and cover extension of the existing ADM station to accommodate an interchange concourse, circulation areas

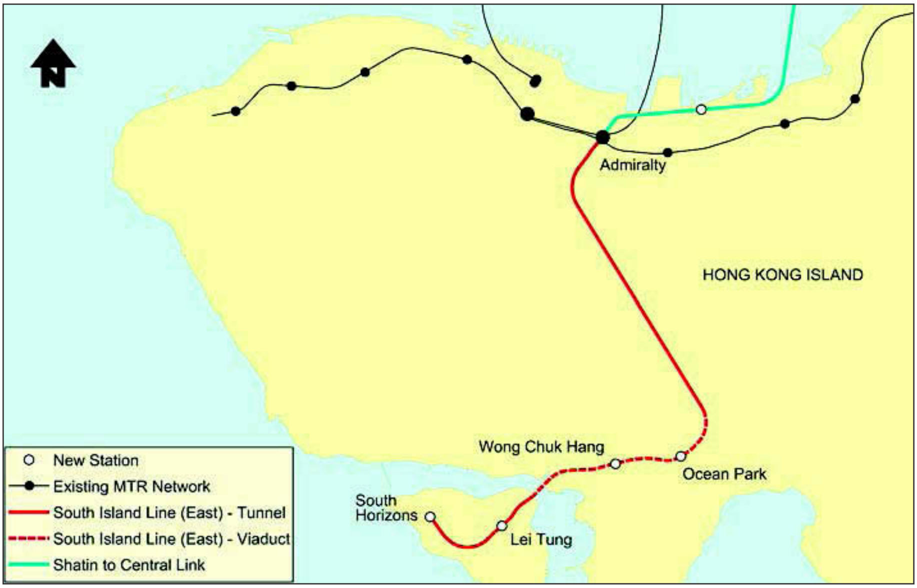

Figure 1. Plan of SIL(E)

and plant rooms plus four new platforms below and adjacent to the existing ADM station; two for the SIL(E) and two for the future SCL. The SIL(E) platforms to be located in a 24m span cavern with the SCL platforms constructed within 10m platform tunnels either side of the cavern. The contract includes 160m long overrun tunnels to the south of the SCL platform tunnels. The SIL(E) running tunnels to the south of the cavern are to be constructed by the adjacent contract.

MTR has awarded Contract 901 to the Kier-Laing O'Rourke-Kaden Joint Venture, who have subsequently engaged Benaim in association with Aurecon to provide all Contractor design inputs.

Contract 901 is located within medium grained granite of Mesozoic age. This granite is referred as Kowloon Granite and it is part of the large body of igneous pluton which has intruded into much older country rock, the fine ash vitric tuff. The majority of the tunnelling is located within the granite with the granite/tuff interface located within the overrun tunnels to the south of the main cavern and platform tunnels.

The tunnels are to be predominately excavated within slightly decomposed to fresh granite. The rockmass consists of 4 main joint sets, one sub-horizontal and the remaining sub-vertical. The rock mass conditions are considered favourable with Q values typically logged between 1 and 8 and rock strengths typically exceeding 75MPa. No major fault zones or other major weaknesses are expected to be encountered through the contract. A minor fault zone of only 2–3m wide is expected within the overrun tunnels. This fault zone expected to contain moderately strong rock with chlorite coated discontinuities (Figure 2).

Contract 904

Contract 904 covers the tunnels and stations on Ap Lei Chau Island. The majority of the contract is constructed in driven tunnel including the LET station, which is to be constructed in a 22m span cavern and includes two associated tunnelled entrance adits. The contract also includes a ventilation adit that joins the running tunnels between LET and SOH stations. The SOH Station is a C&C station.

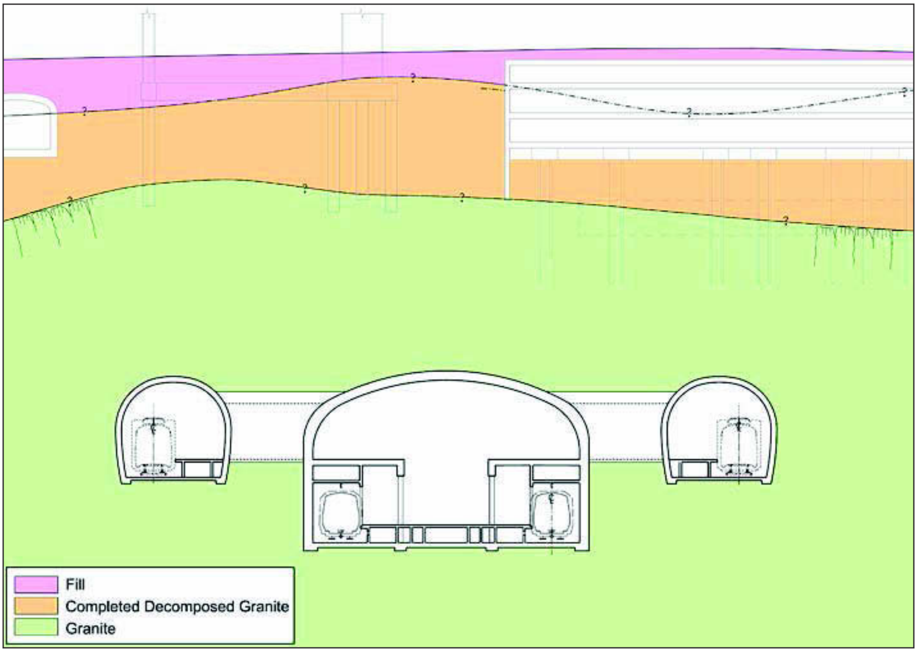

Figure 2. Cross section of Contract 901 cavern and platform tunnels

Figure 3. Plan of Contract 904 tunnels

Table 1. Summary of a typical specification (MTR South Island Line Contract 904)

Item	Summary of Specification Requirements
Probing ahead of the face	■ Three probe holes shall be maintained a minimum of 20m ahead of the advancing face at all times.
Pre-excavation grouting trigger	■ Probe hole inflows are measured 1 hour after completion of the hole. ■ Pre-excavation grouting is required when inflows exceed 0.5 litres/min/m of probe hole or 1.0 litres/min/m of probe hole for a single or multiple probe holes respectively. ■ Pre-excavation grouting is required until the above criteria are met.
Post-excavation grouting trigger	■ Post-excavation grouting is required when inflows exceed 10 litres/min from any individual source on the excavated surface or 50 litres/min over any 50m length of excavated tunnel. ■ Post-excavation grouting is required until the above criteria are met.
Final acceptance	■ Final inflows shall not exceed 3.0 litres/sec for any 1000m of tunnel

MTR has awarded Contract 904 to the Leighton Asia-John Holland Joint Venture, who have engaged Aurecon as its consultant for the design of all tunnelling temporary works.

Contract 904 is located within the Cretaceous age Ap Lei Chau Formation, which predominantly comprises fine ash welded tuff interlayered with distinctive flow bands (Figure 3). The majority of the driven tunnels are located within slightly decomposed tuff and occasionally in moderately decomposed tuff. The tuff is dark grey in its fresh form, but has discoloured to pale grey and pale brown to yellowish brown as the weathering has progressed. Nearer to the portals the tunnels are excavated within completely decomposed tuff, which require soft ground support systems.

The rockmass generally has 3 major joint sets (1 sub-horizontal and 2 sub-vertical) and an additional minor sub-vertical joint set. The rock mass conditions are considered favourable with Q values typically logged between 1 and 15 and rock strengths typically exceeding 100MPa. Two faulting systems are present through the tunnels, trending in the NW-SE and NE-SW directions. These fault zones are expected to contain multiple narrow zones of moderately to highly decomposed tuff as a result of penetrative weathering along the fault planes.

PROBING AND GROUTING SPECIFICATIONS

Typically, the specification for groundwater inflow control will cover the following:

- Probing ahead of the face to identify areas of water inflow,
- When to institute pre-excavation grouting as a function of the results of the probing,
- Requirements for post-excavation grouting to control the water inflows remaining after pre-excavation grouting and excavation,
- Acceptance criteria for water inflows into the completed (and lined) tunnel.

A summary of a typical specification (MTR South Island Line Contract 904) is contained in Table 1.

Responding to such a specification, the tenderers are required to estimate the following:

- Number and length of probe holes: this is an exercise in applying the specification requirements to the contractor's proposed sequence of excavation.
- Number of pre-excavation grout events: this is a probabilistic calculation which is the main subject of this paper.
- Grout take in the rock: the uncertainty in this estimate is very high.

HYDROGEOLOGICAL DATA

During the investigation phase for the project, the hydrogeology will be characterised, assisted with the results of testing. This data is likely to consist of some of the following:

- Groundwater levels; piezometric heads
- Packer tests
- Rising and falling head tests
- Pumping tests
- Groundwater chemistry results

The data should be contained within the broader context of a conceptual hydrogeological model (Stone 1999). This model should be the starting point for the investigation, calculations and specification. The conceptual hydrogeological model should place the project within the relevant regional system of the groundwater cycle and should identify the sensible segmentation of the data.

A CLOSER LOOK AT PACKER TESTS

Conducting a Packer Test

A full explanation of the packer test is given in Houlsby (1990) and Quiñones-Rozo (2010). A maximum pressure is calculated from considerations of depth and confinement. The test length blocked off between inflatable packers. The test is conducted in five stages, measuring the flow over ten minutes for each stage. The pressures are increased and then reduced using 50%, 75%, 100%, 75% and 50% of the maximum pressure. Providing the correct pressure in the test section involves care to account for the difference in head at the surface and the test section, as well as pressure losses due to velocity in the piping. Interpretation is required to differentiate conditions in the test, some of which are related to the transmissive features in the rock, and others to various failures of the test itself. Such conditions can include:

1. Laminar flow, where there is a reasonably linear relationship between flow and pressure,
2. Turbulent flow, where higher velocities cause more resistance than the laminar case,
3. Dilation, where the higher pressures open the discontinuities,
4. Void filling, where the flows from the test clogs the discontinuities,
5. Wash out, where the flows from the test washes out material in the discontinuities,
6. Packer leakage, a common effect that can be easily missed,
7. Poor accuracy (or zero reading) of flow meters at very low flows, and
8. Poor accounting for velocity pressure losses at very high flows.

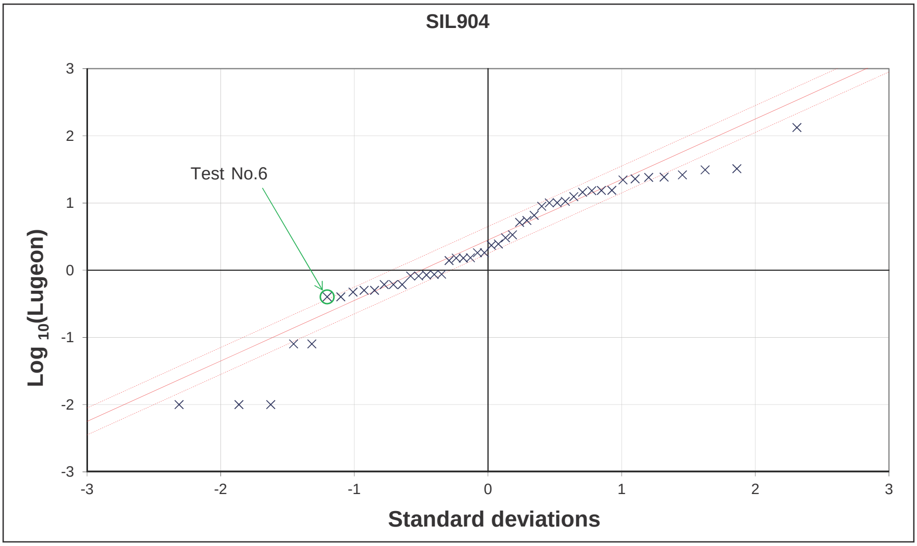

Figure 4. Plot of packer test data set from Contract 904, using the procedure of Raymer (2001, 2005)

Conditions 2 and 3 show a non-linear response to the different pressures. Conditions 4 and 5 show different flows at the same pressures, going up and coming down. Condition 6 is easily confused with 3 or 5. Conditions 7 and 8 show up in the Raymer plot (discussed below).

Analysis of Data

Raymer (2001, 2005) shows an analysis technique based on the assumption that the packer test data is log-normally distributed. The procedure is to consider all of the tests relevant for each data set. It is necessary that this should include all tests, including those where the result is expressed as either "no flow" or "greater than x Lugeon." The tests are sorted into order, and the logarithm (base 10) of the Lugeon value is plotted against the inverse cumulative value of the standard normal distribution for the percentile of the sorted order. Where test results are described as "no flow" a small value is assigned. Similarly for very high (but not accurately measured) results, a high value is assigned.

Figure 4 shows the packer tests from Contract 904, analysed as described. To demonstrate the procedure, test number 6 (in order from lowest to highest) has a test result of 0.4 Lugeon. There are 48 test results. Each result represents 1/48 = 2.08% of the total result set. We assume that each result therefore is located in the middle of its range, and therefore the 6th result is located at a percentile of 11.45% (i.e., 6*2.08% + 0.5*2.08%). Assuming the standard normal distribution, the inverse cumulative value for 0.1145 is –1.202, meaning that this percentile is located 1.202 standard deviations below the standard normal distribution mean of zero. This x axis value of –1.202 is plotted against the y axis value of the log Lugeon result: –0.399 = $\log_{10}(0.4)$. While this sounds complex, in practice it takes very little time using a spreadsheet programme.

Figure 4 shows some commonly seen features. The point on the extreme LHS is a point with no flow. For the purpose of the analysis this has been plotted as a nominal value (of –3). In fact, all of the leftmost points are of doubtful accuracy as the flows measured are below the reasonable accuracy of the flow meter. The points on the right side show a consistent falling off from a linear plot. Investigation of the test reporting

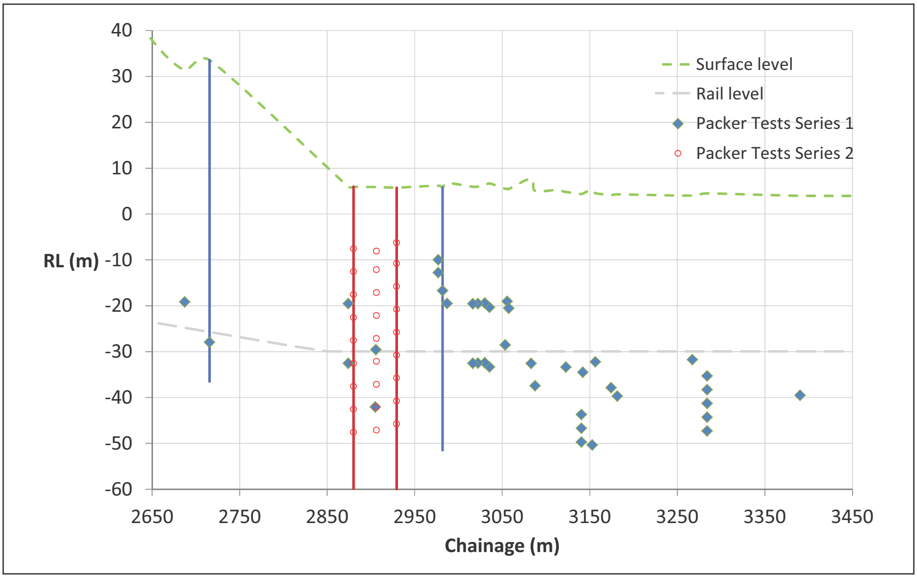

Figure 5. Contract 901 packer test locations by chainage

reveals that no correction has been applied to account for pressure loss due to velocity in the thin pipes feeding the water to the test section. While this correction is negligible for low flows, it leads to an under-reporting of the Lugeon value at high flows. Despite inaccuracy or reporting error at the extremes, a fit is possible to the middle of the data set. The fitted line shows a median value (about $\log_{10}(0.45)$, or 3 Lugeons). The slope of the fitted line (about 0.9) is a measure of the variability of the data, which would be equal to the standard deviation of the data assuming all of the points are on the line of fit.

Selecting the Locations for Packer Tests

It is common practice for geologists to review the core and select packer test locations where there are more discontinuities. Obviously, this practice is not compatible with the statistical analysis described above. An example of this is shown in the case of South Island Line Contract 901.

The first phase of testing adopted the philosophy that packer tests would be taken where the cores showed fracturing. The test lengths varied. In the second phase, the packer tests were set up as a sequence of 5m test lengths only. Figure 5 shows the test locations for both the first and the second series of tests with phase 1 in blue and phase 2 in red.

These test results have been analysed using the method of Raymer. Figure 6 shows the results of this analysis.

It can be seen that the median value of the second series of tests is nearly an order of magnitude less. This result is not surprising, given that the phase 1 tests only foused on fractured zones. However it does demonstrate very well that the statistical analysis of packer test results will be significantly awry if this issue is not understood.

Choice of Test Lengths

There does not seem to be much consistency within the industry with respect to packer test length. The Hong Kong GEOGUIDE recommends from 3m to 6m. We discuss the

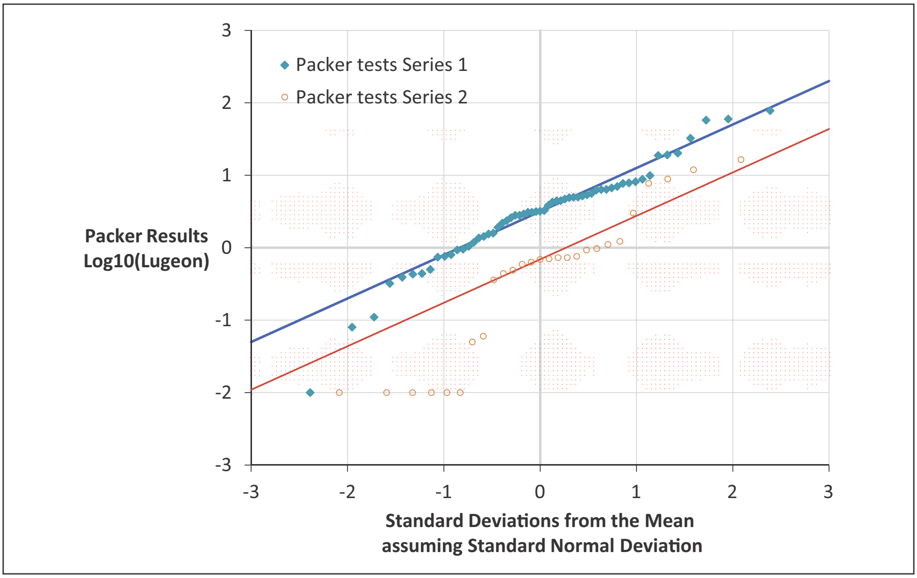

Figure 6. Analysis of Contract 901 packer tests

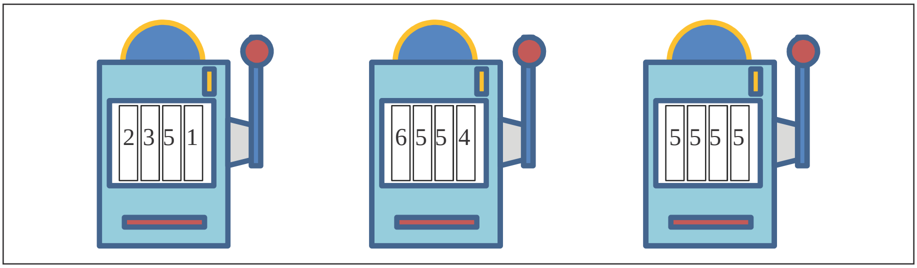

Figure 7. Pokies (a) independent rotors, (b) rotors correlated to their neighbours, (c) rotors linked

issue of varying lengths here. Gustafson (2012) recommends the adoption of exactly 3m as the standard length for packer tests.

Using a longer packer test length (say 20m) is equivalent to taking the average result of multiple shorter packer test lengths (say 4 × 5m) whose length adds up to the longer value. The variability of the longer test length will be different, which is not desirable when undertaking a statistical analysis approach. This can be shown with a simple example.

Consider a poker machine with four rotors and six numbers per rotor. For the purpose of this example, assume that the rotors all start lined up with the number 1111. If the three rotors are independent of each other, there will be 6×6×6×6 equally likely possibilities (Figure 7a), and the likelihood of getting number 5555 will be 1/1296. On the other hand, if the rotors are rigidly linked together and lined up (Figure 7c), then the possibilities are restricted by the linkage and the likelihood of getting number 5555 is 1/6. If the rotors are linked together such that they slip relatively to each other only rarely, then the likelihood of getting number 5555 will be less than 1/6 but not as low as 1/1296. The actual likelihood of getting 5555 will depend on the likelihood of slippage, which is a way of saying that adjacent rotors are correlated.

Table 2. Correlation of nearby tests in Contract 901

Test Spacing	Distance Between Test Centres	Number of Correlation Pairs	Correlation
Adjacent	5m	24	0.345
One test apart	10m	21	0.109
Two tests apart	15m	18	–0.043

Packer tests taken from separate locations in the same rockmass can be expected to be independent. However, as with many geotechnical properties, tests taken close to one another have a tendency to be similar. This is called autocorrelation. Autocorrelation can be calculated for packer tests if there are boreholes available with sequential packer tests. For a borehole with n sequential packer tests, there are n-1 pairs of results which are one packer test spacing apart, n-2 pairs which are two packer test spacings apart, and so on. Correlation is calculated by:

$$\rho_{x,y} = \frac{\sum x_i y_i - \bar{x}\bar{y}}{(N-1)\sigma_x \sigma_y} \tag{1}$$

where x and y are data sets with N objects with mean $\bar{x}$, $\bar{y}$ and standard deviation σ_x, σ_y.

The packer tests in the second series of boreholes in Contract 901 each have 9 tests of 5m length. The correlation between nearby tests is as shown in Table 2, noting that these correlations are calculated for the log(Lugeon) results.

For n tests, which are correlated together with an average correlation of ρ_{av}, the standard deviation of the mean of the n tests is:

$$\sigma_n = \sigma_i \sqrt{\frac{1}{n} + \frac{n-1}{n}\rho_{av}} \tag{2}$$

where σ_i is the standard deviation of the individual tests.

To give a concrete example, the standard deviation of 20m packer tests can be estimated from analysing the results of sequential 5m packer tests. If there is no autocorrelation, the standard deviation of the 20m packer tests is expected to be half of the standard deviation of the 5m packer tests. There are three ways that 5m tests are located adjacent to one another, two ways that the 5m tests are 5m apart, and one way that the 5m tests are 10m apart. The average correlation is therefore:

$$\rho_{av} = \frac{3\rho_1 + 2\rho_2 + \rho_3}{6} \tag{3}$$

where ρ_1 is the correlation of adjacent tests, ρ_2 is the correlation of tests one spacing apart etc.

For the data in Contract 901, the average correlation is 0.202. Assuming a standard deviation of 0.6 (the slope of the line in Figure 6), the expected standard deviation for 20m tests is 0.38 using Equation 2. Gustafson (2012) shows the same effect of reducing variability with increasing packer test length.

PREDICTION OF PROBE INFLOWS FROM PACKER TESTS

Heuer (1995) provides an estimate of tunnel inflows, and also discusses probe hole inflows. Heuer provides a heading inflow factor (HIF), which accounts for the three dimensional effect at the face, and for the fact that higher inflows can be provided by local reservoirs.

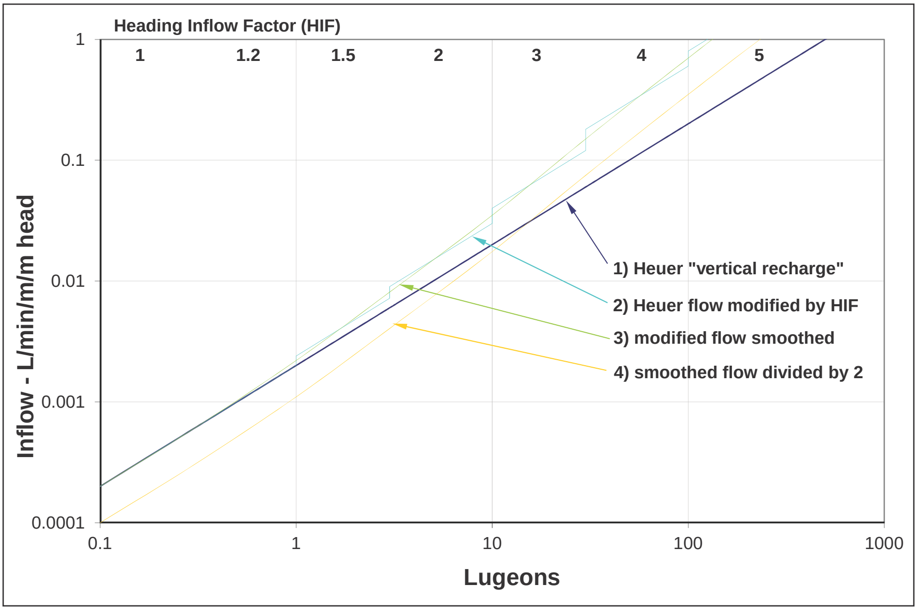

Figure 8. Adjusted Heuer chart

Heuer also proposes that probe hole inflow will be less than the full tunnel inflow. If the assumption is that our probe hole picks up one half of the tunnel inflow, Figure 8 shows the version of the Heuer inflow chart for probe holes.

The probe hole length is specified to be at least 20m. Exactly the same issues discussed above for packer tests lengths apply here. A 20m probe will demonstrate a significantly lower variability than is seen for the short packer tests, and the calculation methodology to adjust for this is as described above.

The result is expressed as a probability versus identification of actual localities. An example calculation is given below.

Adopting the results from Contract 904, we interpret the packer tests to have a median value of 3 Lugeons, and a slope on the Raymer graph of 0.9 (see Figure 4). This latter number is in fact the standard deviation of the log(Lugeon) values of the recovered packer test distribution.

The probe holes are 20m long, and the average packer test lengths approximately 4m. Therefore 5 packer tests will constitute one probe hole. Using the correlation numbers in Table 2, and assuming that the tests are essentially independent when greater than 10m apart, the average correlation for the 5 tests is (4 × 0.345 + 3 × 0.109 + 2 × 0+ 1 × 0)/10 = 0.171. The standard deviation for the probe holes, based on the packer test results is therefore (using Equation 2) 0.58 × 0.9 = 0.52. Therefore we are expecting the distribution of probe hole permeabilities to have a mean of 3 Lugeons, and a standard deviation of 0.52 log(Lugeons).

If the tunnel is (say) 40m deep, the adjusted Heuer chart allows us to predict the permeability which will trigger the specification requirement to pre-excavation grout. Applying this trigger to the standard normal distribution with the parameters estimated above gives a probability that the trigger will be exceeded. If the trigger is expected to be exceeded (say) 9% of the time, an estimate can now be made of the number of pre-excavation grouting events.

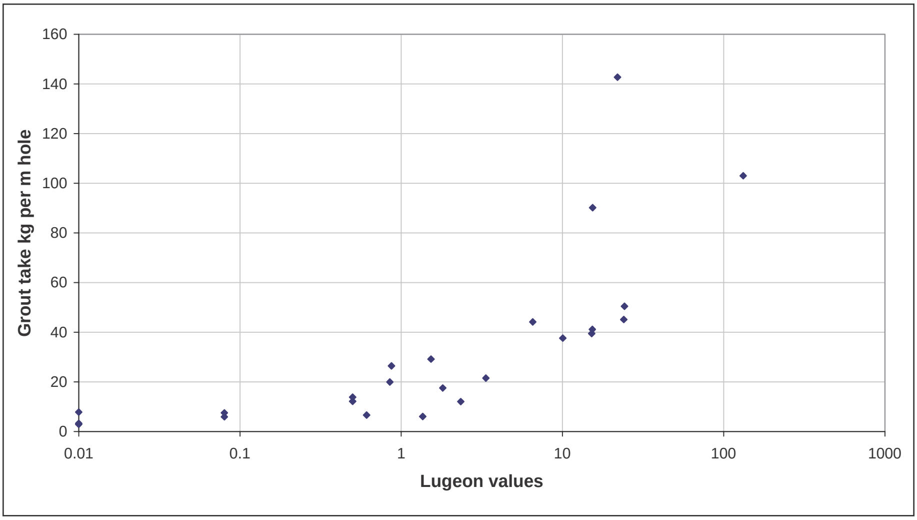

Figure 9. Grout take versus Lugeon value from Snow (1968)

PREDICTION OF GROUT TAKES

Grout takes are notoriously difficult to estimate. This is for two reasons. Firstly, just like the results of packer testing, the grout takes of individual holes varies significantly. Secondly, the grout take is a quantity that is very sensitive to the construction methodology including such things as the grout type, pressures and the philosophy adopted.

Theoretically, the grout take should be a function of the volume of the fissures within the rockmass. A well-known approach is from Snow (1968) who provides a relationship between aperture and hydraulic conductivity. Figure 9 shows the theoretical result for Contract 904 for this approach, which also makes the assumption that the grout will flow into the apertures for an average 5m distance. Gustafson (2012) provides an update of the Snow analysis.

Practical experience and theoretical analysis since Snow shows that the amount of grout injected is more likely to be affected by grout pressure than the apertures measured before grouting. A more practical approach is to consider past experience in similar rockmass with similar grouting pressures. Unfortunately this information is often hard to come by and is often presented in ways that makes it difficult to use. Klüver and Kveen (2004) present the results of grout take in road tunnels in the hard rock tunnels of Norway. The quantities vary widely but the results show that typical values per m of grout hole range from less than 40kg/m of hole, up to greater than 60kg/m of hole. Obviously these grout takes are going to depend greatly on the specification, the rockmass, the grout type and mixture, the grout pressures and the philosophy used in the grouting. We encourage more reporting of the type presented by Klüver and Kveen, and particularly reporting of grout takes per metre of hole.

CONCLUSIONS AND ACKNOWLEDGMENTS

This paper has examined the use of some analysis techniques for estimating probing and grouting requirements for rock tunnels. The analysis techniques are based on the assumption that packer test results are log-normally distributed and that the test results themselves provide a reasonable sample of the rockmass. There are a few fundamental requirements for setting up packer testing which we recommend:

1. The packer test length should be standardized at 3m.

2. The packer tests should be a series of adjacent tests at 3m intervals in the area of interest in the boreholes.
3. The coverage of packer tests should be as representative as possible along the length of the tunnel. For tunnels crossing under a hill or mountain, the expense of drilling boreholes in the deeper section is an issue which will argue against this requirement. However, the deeper sections are also the ones subject to the higher head and potentially greater inflows.
4. The correction for head loss due to velocity in the packer test should be applied based on flow tests of the actual packer setup used for the testing.

With respect to grouting, we suggest that actual grouting trials, using the grouting philosophy preferred by the client are carried out as part of the client's investigation, if grouting is expected or required to be used in the project, to provide a better basis for estimation.

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MINED ROCK TUNNEL THROUGH DAM ABUTMENT AT WARM SPRINGS DAM, SONOMA COUNTY, CA

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ABSTRACT

HNTB-Gerwick JV was contracted by the United States Army Corps of Engineers (USACE) to perform 65% design for a tunnel alternative to replace an emergency water supply line (EWSL) at Warm Springs Dam, Sonoma County, CA. The tunnel alternative consists of a 72 inch diameter pipe in carrier tunnel, 4.0 m (13.0 ft) wide by 3.4 m (11.0 ft) high. The proposed tunnel consists of 610 m (2,000 ft) of mined tunnel and 107 m (350 ft) of cut and cover tunnel. The mined tunnel would be constructed through the left abutment of the dam in sandstone and shale under high head of water. Key design and constructability issues include seismic design considerations, mined tunneling in weak rocks, groundwater control and dam safety.

INTRODUCTION

The Dry Creek (Warm Springs Dam) project, located in Sonoma County near Geyserville, CA as shown in Figure 1, was authorized by the Flood Control Act of 1962 and completed in 1983 for the purposes of flood control, water supply and recreation. The Congressman Don Clausen Fish Hatchery, located on the downstream side of the dam, was constructed in 1979 to mitigate for the loss of fish resulting from the construction and operation of the dam. An emergency water supply line (EWSL) was constructed for the hatchery in 1992 after it was determined that the original emergency water supply sources did not meet the minimum hatchery operation requirements. The existing EWSL consists of a 20 inch diameter pipe line suspended from the top (inside) of the main outlet tunnel for the dam. After the federal designation of Coho Salmon as a threatened species in 1995, the USACE expanded hatchery operations with the Coho Salmon Rescue Program. Consequently, the existing EWSL can no longer meet the hatchery's water requirements, which include the supply of 1.84 m^3/s (65 cfs) of water within minutes of closure of the outlet works gates.

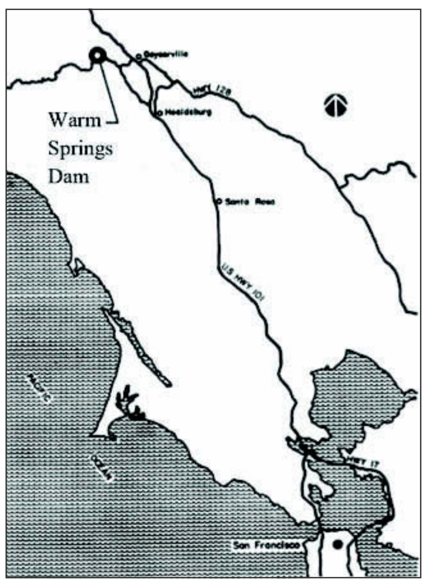

Figure 1. Project location map

In 2007, the USACE completed an engineering evaluation of EWSL replacement alternatives, which include a tunnel alternative consisting of a 72 inch diameter pipe in a carrier tunnel constructed through the sedimentary rock (sandstone and shale) of the left (north) abutment of the dam. HNTB-Gerwick JV was subsequently authorized by the USACE, San Francisco District to complete engineering and design for the tunnel alternative to the 65% design level. Key design and constructability issues include seismic design considerations, mined tunneling in weak rocks, groundwater control and dam safety.

TUNNEL ALIGNMENT AND CROSS SECTION

A site plan indicating the proposed EWSL tunnel alignment is presented in Figure 2. The proposed tunnel profile is presented in Figure 3. The proposed cross section for the mined portion of the carrier tunnel is presented in Figure 4. The proposed 72 inch diameter pipe taps an existing 72 inch diameter wet well in the dam's control structure at the upstream end and conveys the water to a new bifurcation vault at the downstream end where the water is diverted to the new fish hatchery stilling well, Sonoma County Water Agency (SCWA) pipes, or both. The overall distance from the control structure to the stilling well is approximately 902 m (2,960 ft) consisting of approximately 716 m (2,350 ft) of 72 inch diameter steel pipe in a cast-in-place reinforced concrete lined carrier tunnel, of which 610 m (2,000 ft) is mined and 107 m (350 ft) is cut and cover, followed by 84 m (275 ft) of 72 inch reinforced concrete cylinder pipe (RCCP) in a shallow trench from the end of the tunnel to the bifurcation vault, followed by 84 m (275 ft) of 36 inch diameter ductile iron (DI) pipe in a shallow trench from the bifurcation vault to the stilling well. Considerations for development of the vertical alignment, horizontal alignment and cross section are as follows.

Vertical Alignment

The elevation of the proposed carrier tunnel at the upstream end (refer to Figure 3) is controlled by the location of the proposed connection of the new EWSL to the existing 72 inch diameter wet well inside the control structure shaft. The elevations of the new EWSL and carrier tunnel decrease at a 0.1% grade in the direction from the control structure shaft to allow free drainage of the tunnel during construction and in the permanent condition, with consideration to anticipated construction tolerances.

Horizontal Alignment

The horizontal alignment of the proposed carrier tunnel is influenced by the presence of an existing drainage tunnel (refer to Figure 2), which is located beneath the dam at approximately the same elevation as the carrier tunnel. The alignment of the carrier tunnel has been set to achieve a minimum horizontal clearance of approximately 15 m (50 ft) from the drainage tunnel to the centerline of the carrier tunnel. The proposed horizontal alignment achieves the required curvature through a series of straight line segments (i.e., chords) with infrequent, uniform small angle bends of approximately 20 degrees each. The proposed angle bends all turn in the same direction (i.e., no reverse curves) such that the thrust blocks can be limited to one side of the tunnel only, allowing for a continuous inspection walkway. The tunnel alignment has been set to cross the existing grout curtain wall on a line that is approximately perpendicular to the line of the wall in order to minimize the length of "penetration" (the proposed tunnel passes approximately 12 m (40 ft) below the existing grout curtain wall as shown in Figure 3).

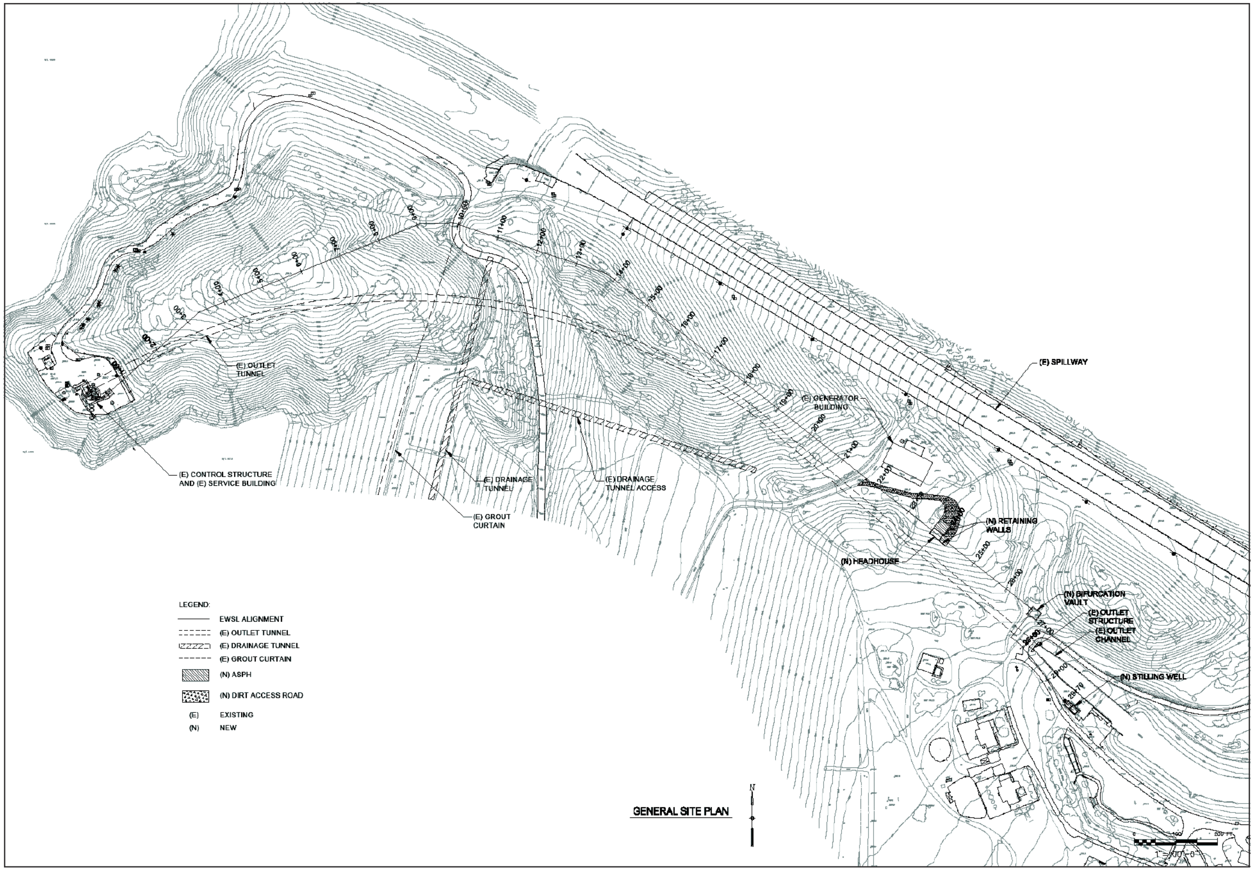

Figure 2. Site plan with proposed tunnel alignment

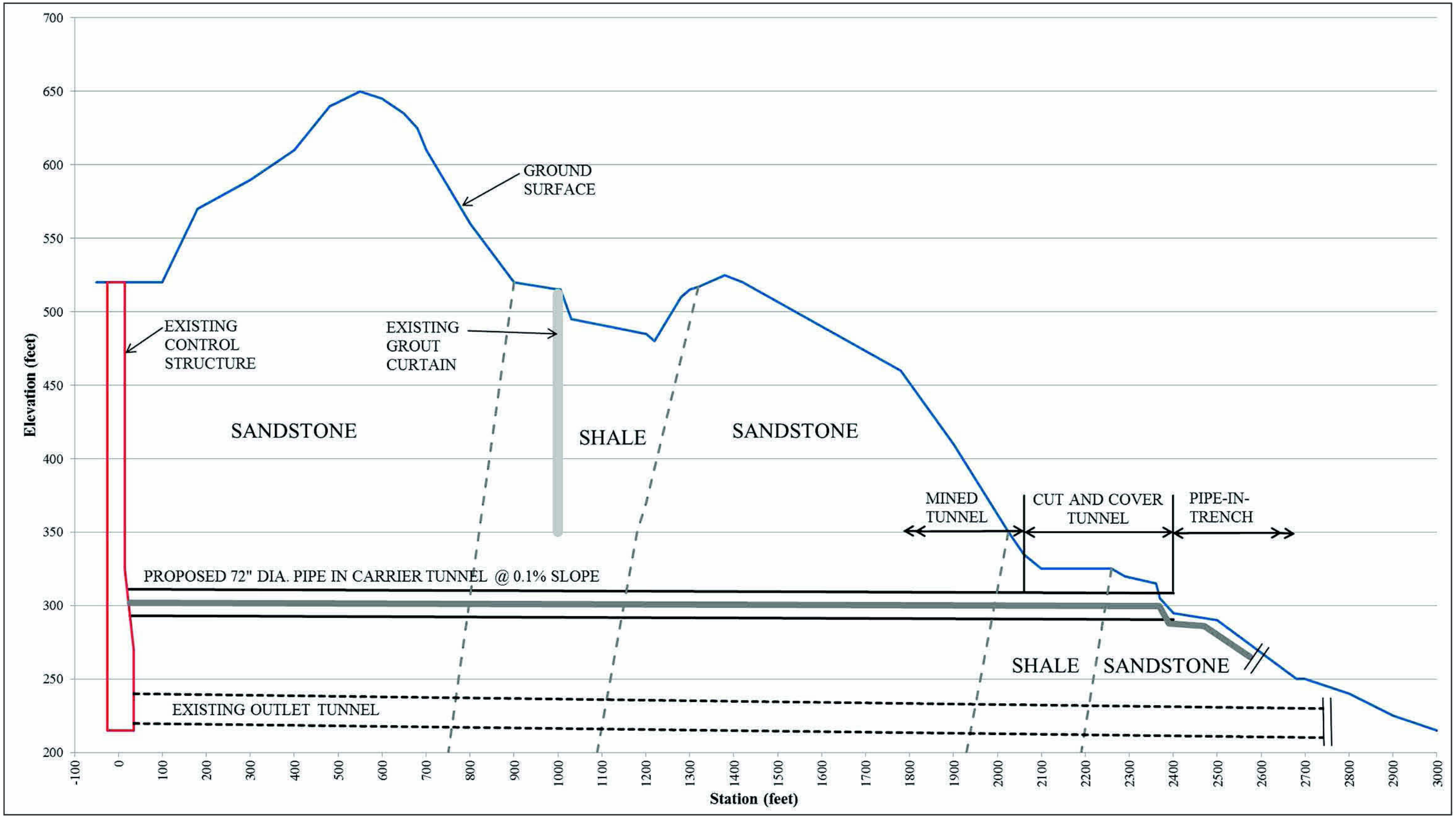

Figure 3. Proposed EWSL tunnel profile

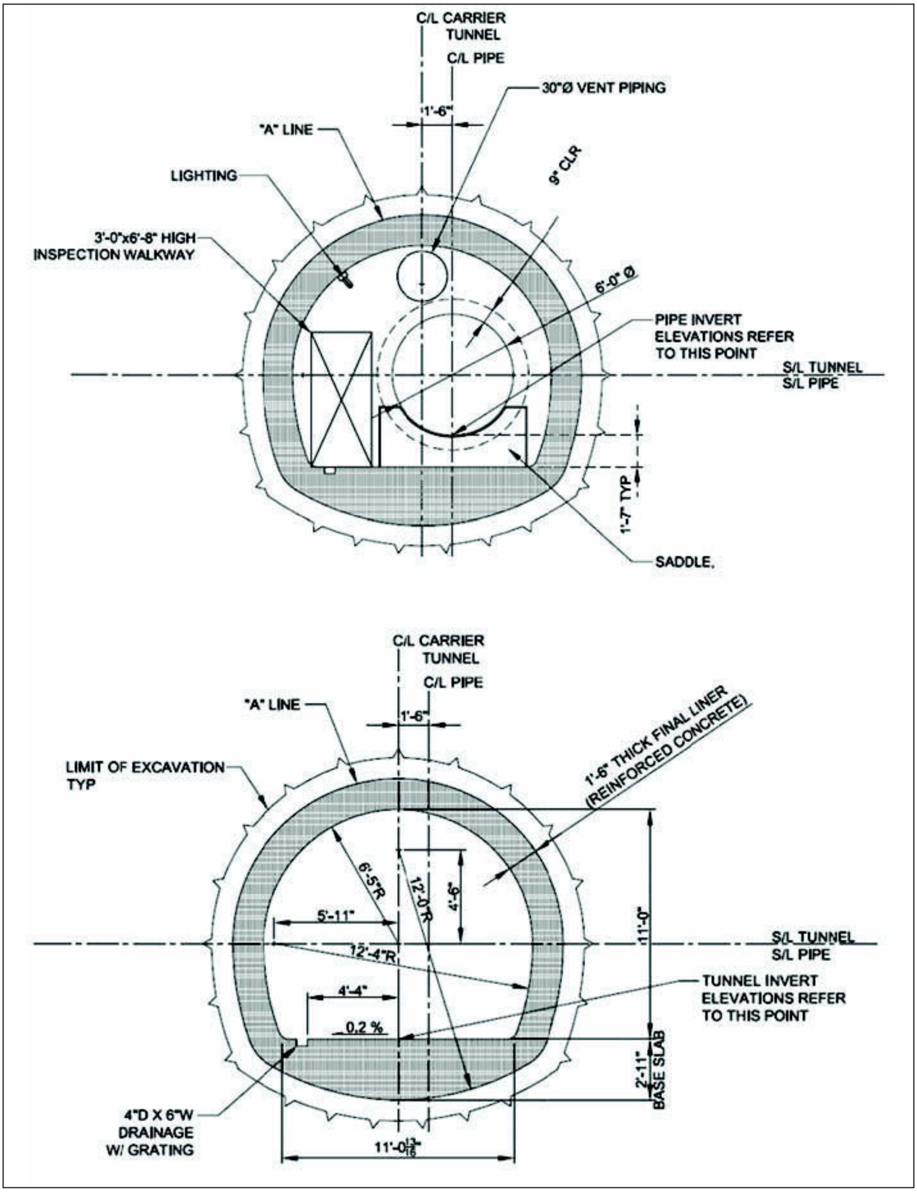

Figure 4. Proposed mined tunnel cross-section—lining dimensions and layout

Cross Section

The interior dimensions of the proposed tunnel cross section are approximately 4.0 m (13 ft) wide by 3.4 m (11 ft) high as presented in Figure 4. The tunnel has been sized to accommodate the 72 inch EWSL, with minimum 0.5 m (18 inch) clearance from the tunnel structure to allow for inspection, and an inspection walkway with an unobstructed clearance envelope of 0.9 m (3 ft) wide by 2.0 m (6 ft-8 inch) high as per the USACE's requirements. The tunnel cross section also includes a 30 inch diameter ventilation pipe and lighting with electrical service.

PROJECT GEOLOGY

Rocks underlying the dam site and reservoir areas are principally those of the Late Jurassic to Late Cretaceous Great Valley sequence and Franciscan assemblage. The two formations are separated to the north and south by the Dry Creek Fault zone. Franciscan rocks on the south side of the Dry Creek Fault are an assemblage of sandstone (greywacke), interbedded shales, altered volcanic rocks (greenstones), ultramorphic rocks and serpentinites, and minor amounts of glaucophane and related schists. North of the Dry Creek Fault zone, including the left (north) abutment of the dam through which the proposed tunnel alignment passes, the Great Valley sequence consists of bedded carbonaceous shales with intraformational arkosic sandstones and cobble conglomerates. All have experienced several periods of intense folding and faulting.

Bedrock is covered by stream channel and terrace deposits that are generally described as a mix of clays, silts, gravels and sands of varying density and strength. In general, the soil deposits are about 3 to 12 m (10 to 40 ft) thick. With the exception of beneath the downstream test fill area, the soil deposits and upper areas of weathered bedrock were removed below the dam. Ground water naturally occurs within the rock and soil formations, naturally flowing toward the valley in the rocks, and following the topography in the valley.

Warm Springs Dam is located in an area having high seismicity. Active nearby faults include the Healdsburg and Macaama faults which are located 3 and 10 kilometers (2 and 6 miles) away from the dam site, respectively and have the potential to cause earthquakes greater than 7.0M. The San Andreas fault, which can produce earthquakes larger than 8.0M, is located about 31 km (19 miles) from the dam site. The Dry Creek Fault, which crosses the dam site (though not the proposed tunnel alignment), is not believed to be a seismic source, but may experience sympathetic rupture with other nearby seismic sources. In addition to strong ground shaking, the dam site is subject to co-seismic fault rupture, landslide and liquefaction hazards.

SEISMIC DESIGN CONSIDERATIONS

Based on a review of previous seismic analysis for the Warm Springs Dam site, seismic events originating from either the Healdsburg Fault or the San Andreas Fault are most likely to cause design level seismic loads at the project site. The fault parameters were used to generate synthetic ground motions for stability evaluations. Ground motions at the tunnel depth were estimated by deconvolution of ground surface motions developed using Next Generation Attenuation (NGA) equations. Two levels of design earthquakes are considered in the seismic design: Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE). Design earthquake definitions and associated structure performance requirements are as per EM 1110-2-2104, Chapter 3 (USACE 2003).

Ovaling of the mined tunnel was evaluated through 2D time-history dynamic analysis of the transverse section. The inputs for the dynamic analysis were the time series of the motions provided by the seismic hazard study. Racking of the cut and cover tunnel was checked in accordance with acceptable state of practice guidelines as presented in FHWA-NHI-10-034, Chapter 13 (FHWA 2009). Strain due to axial and curvature deformation was computed using the simplified free-field deformation procedure also as presented in FHWA-NHI-10-034, Chapter 13 (FHWA 2009).

Beam-spring models were used for the 65% structural analysis of the mined tunnel lining. The beam spring model represented the lining by a series of linear-elastic beam elements. The rock or soil medium was represented by non-linear (elasto-plastic) radial and tangential springs. Two types of soils/rock parameters were modeled: shale and sandstone. Rock loads were computed following empirical relationships and applied to the model. Hydrostatic pressure was applied orthogonal to the lining. Live load cases are not critical by inspection and therefore were not analyzed. Loads were factored as required by EM 1110-2-2104, Chapter 3 (USACE 2003) for site specific ground motions. Seismic displacements time series were applied directly to the lining through the springs; five MCE and one OBE type load combinations were analyzed. The output of the analysis includes bending moments, axial loads, shear and deformations in the lining. Since the lining is subjected to combined normal force and bending moment, the design is carried out using the concrete section capacity interaction curves.

The design of the final lining for the mined tunnel was governed by the seismic load cases for the Healdsburg Fault. The final lining currently consists of an 18 inch thick reinforced concrete modified horseshoe lining with curved invert and sidewalls. Additional design effort will be required during final design to optimize the lining. This effort will include refinement of seismic design inputs and analysis with continuum (finite element) models to supplement the beam-spring models used to date.

PREVIOUS TUNNELING EXPERIENCE AT SITE

Three tunnels have been previously constructed in the same abutment of the Warm Springs Dam as the proposed EWSL tunnel (USACE 1986):

- Drainage Tunnel Access (1970)
- Drainage Tunnel (1978–1979)
- Outlet Tunnel (1978)

The locations of the existing tunnels relative to the proposed EWSL tunnel are indicated on the site plan in Figure 2. The location of the existing Outlet Tunnel relative to the proposed EWSL tunnel is also indicated on the profile in Figure 3.

The three existing tunnels are of comparable size to the proposed tunnel and were constructed in geology similar to that anticipated for the proposed EWSL tunnel (i.e., moderately broken sandstone and bedded to severely sheared shale of the Great Valley sequence). The invert elevations of the Drainage Tunnel Access and Drainage Tunnel are approximately six meters (20 ft) higher than the invert elevation of the proposed EWSL tunnel. The invert elevation of the Outlet Tunnel is approximately 24 m (80 ft) lower than the invert elevation of the proposed EWSL tunnel.

The three existing tunnels were constructed using drill and blast methods with the exception of zones of caving shale, which were excavated using hand tools and mechanical equipment. Temporary support consisted of horseshoe-shaped steel rib sets with rock bolts and shotcrete. Set spacing was usually 1.2 to 1.5 m (4 to 5 feet) with closer spacing in a few areas where crown bars and spiling were also used.

The principal difficulties encountered in mining were caused by the intersection of shaly seams parallel to the bedding with a major joint set approximately perpendicular to the bedding. Overbreak of 30 to 60 cm (1 to 2 ft) was common from a height approximately one-third the distance up from the spring line to the crown, while overbreakage of up to 30 cm (1 ft) was common lower down. An unusually large cavity, 4 to 6 m (15 to 20 ft) in height and about 1.2 m (4 ft) wide, occurred in the downstream portion of the Outlet Tunnel caused by fallout of unstable rock.

Mining through zones of weak shale commonly utilized six inch channel iron spiles/forepoles to prevent caving. Several rib sets within the sheared shale became deformed under excessive loadings and some footings sank one to two inches into the soft shale floor; the Contractor installed cross braces at the invert (above the foot plate) to mitigate this problem.

None of the tunnels experienced significant groundwater inflows during construction, however, the tunnels were constructed prior to construction of the main dam embankment and subsequent filling of the reservoir, which has altered the hydro-geological conditions at the site.

ANTICIPATED CONSTRUCTION METHODS FOR MINED TUNNEL

The tunnel will likely be excavated with a roadheader as opposed to a tunnel boring machine (TBM) or drill and blast methods. The relatively high mobilization costs for a TBM are likely to be prohibitive given the relatively short length of the mined tunnel. Drill and blast methods are currently not allowed by the USACE at this site. The strength of the sandstone (unconfined compressive strength of 23.2 to 87.8 MPa (3,370 to 12,740 psi) with an average of 50.7 MPa (7,350 psi) and Brazilian indirect tensile strength of 1.9 to 9.9 MPa (270 to 1,430 psi) with an average of 4.1 MPa (600 psi), as determined by the site investigation for 65% design) is conducive to excavation with a medium sized roadheader (52 to 54 metric tons (115 to 119 kips)).

Initial support for the most competent ground (Ground Class I) will likely consist of pattern rock dowels with a 3 inch layer of fiber-reinforced shotcrete above the springline of the tunnel. Initial support for slightly less competent ground (Ground Class II) will include modified horseshoe lattice girders and fiber-reinforced shotcrete extending around the full perimeter of the excavation. When the sheared shale is encountered (Ground Class III), initial support will consist of lattice girders around the full perimeter (i.e., an invert strut will be added to the lattice girder for Ground Class II). Excavation through this weak ground condition will likely require spiling above the crown and fiberglass reinforcing rods (bolts) driven into the tunnel face in advance of excavation so as to develop and maintain a stable heading.

CONTROL OF WATER/DAM SAFETY

Hydrology of the Left (North) Abutment

The hydrogeology of the left (north) abutment of the Warm Springs Dam was described in the EWSL Final Engineering Report (USACE 2006) as follows:

> "*Groundwater elevation upstream of the grout curtain has been measured as high as 146.3 m (480 ft) msl and downstream as low as 97.5 m (320 ft) msl. The water table upstream of the grout curtain appears to fluctuate in relation to the pool behind the dam. This indicates good connection between groundwater and the lake as well as good transmissivity through the rock. The water level downstream of the grout curtain remains relatively constant. It is assumed that the drainage tunnel significantly dewaters the abutment downstream of the grout curtain.*"

The peaks and troughs of the reservoir elevation, which typically vary annually between 129.5 m (425 ft) msl and 137.2 m (450 ft) msl but have been as high as 146.3 m (480 ft) msl, generally align with the peaks and troughs of the coinciding drainage tunnel flow rates, which is further evidence of the connectivity between the reservoir and the rock beneath the dam (and extending into the abutment).

Piezometers installed along the proposed tunnel alignment as part of the site investigation for 65% design show a piezometric drop of up to 24.4 m (80 feet) from one side of the grout curtain to the other at the approximate elevation of the proposed tunnel. Packer testing performed as part of the site investigation indicated the permeability of the rock is generally quite low with few exceptions. The equivalent rock mass permeability, k_e, of the rock was determined to be less than 1×10^{-5} cm/s for 53.5% of the tests, less than 1×10^{-4} cm/s for 71% of the tests and less than 1×10^{-3} cm/s for 94% of the tests. Analysis of potential groundwater inflows into the tunnel excavation indicate inflows can be reduced to manageable levels using traditional pre-excavation grouting techniques (i.e., using Portland cement grouts as opposed to microfine cement or chemical grouts and limiting grout injection pressures to less than the calculated overburden pressures).

Potential Failure Mechanism

A dewatering/control of water work plan was developed with the following primary failure mechanism in mind: Excavation of the tunnel encounters a singular feature, or a number of distributed features, in the rock that provide direct hydraulic communication with the reservoir above. Water from the reservoir would have a pathway, albeit a tortuous one, to the tunnel excavation, which in turn would provide a direct route to points downstream. If the fractures contained an erodible soil infilling, this infilling could pipe into the tunnel, significantly increasing the flow rate into the tunnel. The worst case scenario would involve exposure of a fault consisting of a highly permeable, highly erodible gouge material leading directly from the tunnel to the reservoir. The probability of occurrence for this worst case scenario is considered to be remote.

Mitigation Measures

The design of the tunnel includes mitigation measures to counter the failure mechanism described in the preceding section:

- Exploratory (probe) holes will be drilled from the tunnel heading in advance of the tunnel excavation. Groundwater inflow will be measured from the exploratory holes. Should the inflow exceed a limiting value (yet to be determined), pre-excavation grouting would be performed at the tunnel heading through holes drilled around the perimeter of the excavation. Mandatory grouting (regardless of measured groundwater inflow) will likely be prescribed as the tunnel heading nears the reservoir. Grout mix designs and grouting pressures will be selected to effectively penetrate the discontinuities or provide sufficient densification of the ground to reduce permeability to acceptable levels.
- It should also be noted that the use of a roadheader to excavate the tunnel, as opposed to drill and blast methods, will reduce the likelihood of a sudden advance into a water-bearing feature, since only a small portion of the tunnel cross-section will be excavated at once.
- To further reduce the risk of a catastrophic inflow of water during construction, two additional temporary measures will be considered during final design.
  - The first is the mandatory construction of a temporary emergency bulkhead at a location between the tunnel portal and the existing grout curtain wall. The temporary emergency bulkhead would be designed by the Contractor to accommodate his means and methods of tunnel construction while meeting mandatory design requirements such as the ability to close under flowing water conditions. The bulkhead would be constructed before the tunnel heading is advanced upstream of the grout curtain wall. In the event an uncontrollable inflow of water was encountered during construction, the Contractor could evacuate the tunnel and seal off the bulkhead. In this event, all points upstream of the bulkhead would become charged with a water pressure equivalent to the static head of the reservoir. Therefore, it is important the bulkhead be constructed at a location where sufficient overburden exists (at all points upstream) to adequately resist the hydrostatic pressure.
  - The second additional temporary measure is the optional controlled lowering of the reservoir in advance of tunnel excavation. Reduction of the reservoir elevation is directly proportional to the reduction in the hydraulic

head that drives the flow from the reservoir into the tunnel. The reduction of risk must be weighed against the negative operational impacts of lowering the reservoir, both to the USACE and to third parties.

CONSTRUCTION COST ESTIMATE AND SCHEDULE

As of the conclusion of 65% design (2011), the project construction cost is estimated at $39.1 million and the project construction schedule is estimated to be 19 months.

PROJECT STATUS

The Warm Springs Dam EWSL project remains at 65% design for the design alternatives, including the tunnel alternative described herein. The USACE along with the local project sponsors, Sonoma County Water Agency, are considering the local environmental conditions of Dry Creek and how best to meet emergency water supply needs for the hatchery, municipal water needs, and Dry Creek biology needs. Various environmental restoration projects are currently under construction along Dry Creek. The performance of these restoration projects and environmental studies of the creek will provide valuable information needed to select the appropriate alternative for the construction of the Warm Springs Dam EWSL. The tunnel alternative, although more expensive than other alternatives, would provide additional operational flexibility that may be needed to meet fish hatchery, municipal water supply, and Dry Creek flow requirements, with lower O&M costs as compared to the other alternatives. The additional costs of the tunnel alternative would be borne by the local sponsor.

CONCLUSIONS

The proposed tunnel alternative for the EWSL replacement involves several design and construction challenges including seismic considerations, tunneling in weak rocks, control of groundwater and dam safety. Seismic load cases were found to govern the 65% design of the tunnel liner; additional work will be required during final design to refine seismic design inputs. The construction challenges associated with tunneling in the worst case ground conditions (sheared shale of the Great Valley sequence) can be met using tunneling techniques not unlike those previously used in the construction of existing tunnels at the site. These techniques include installation of spiling above the crown and installation of fiberglass reinforcing rods (bolts) into the tunnel face in advance of excavation so as to develop and maintain a stable heading. The risk of tunneling through the left (north) dam abutment in the presence of high groundwater head from the adjacent reservoir can be mitigated with diligent application of traditional rock tunneling groundwater control practices such as probing and grouting from the tunnel heading in advance of the excavation. Additional measures to be considered during final design include lowering of the reservoir in advance of construction and installation of a temporary bulkhead within the tunnel excavation before the heading passes into zones of the abutment with higher hydraulic heads.

ACKNOWLEDGMENT

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CHEMICAL GROUTING FOR WATER CONTROL AT FOUR POINTS SHAFT

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ABSTRACT

The Jollyville Transmission Main is a 6.5 mile long treated water tunnel that runs from a new water treatment plant to the Jollyville Reservoir in Austin, Texas. The tunnel is being constructed in an environmentally sensitive area, and as such employs various mitigation measures to address the areas of concern. Of particular concern was the Four Points working shaft, which was constructed through a formation that hosts groundwater which feeds springs that are vital to the habitat of the Jollyville salamander. This paper discusses the chemical grouting program that was employed at the Four Points shaft to reduce groundwater seepage into the shaft to satisfactory levels.

PROJECT DESCRIPTION

The Jollyville Transmission Main (JVTM) is a 6.5 mi. long treated water transmission main, conveying treated water from Water Treatment Plant #4 (currently under construction) to the Jollyville Reservoir as part of an effort by the City of Austin to keep up with current and expected growth. The City of Austin hired MWH Constructors as Construction Manager at Risk (CMAR) for the treatment plant project, which in turn retained Brierley Associates for assistance with construction management and technical services. Southland/Mole JV (SLM) was awarded the contract for the JVTM portion of the project. The tunnel has an excavated diameter ranging from 118 in. to 128 in., with a final lining of 84 in. ID prestressed concrete cylinder pipe. In addition to the tunnel there are four shafts, ranging in diameter from 15 ft. to 40 ft.; two working shafts and two receiving shafts for the three TBMs used on the project. SLM manufactured the two 118-in.-diameter TBMs, one of which had a new Robbins cutterhead, while the third is a 128 in. Robbins hard rock TBM. The tunnel is being mined through interbedded limestone and dolomite. The Project alignment is shown on Figure 1.

The JVTM, designed by Black and Veatch, includes special design elements that are intended to address the sensitive environmental features through which the tunnel and shafts are mined. One of the most sensitive features is the Edwards Aquifer, a karstic limestone aquifer that supports the habitat of the threatened Jollyville Plateau Salamander. One of the two working shafts on the project, dubbed the Four Points Area (FPA) shaft, was constructed through the Edwards Aquifer and was designed to impact the aquifer flow system as little as possible.

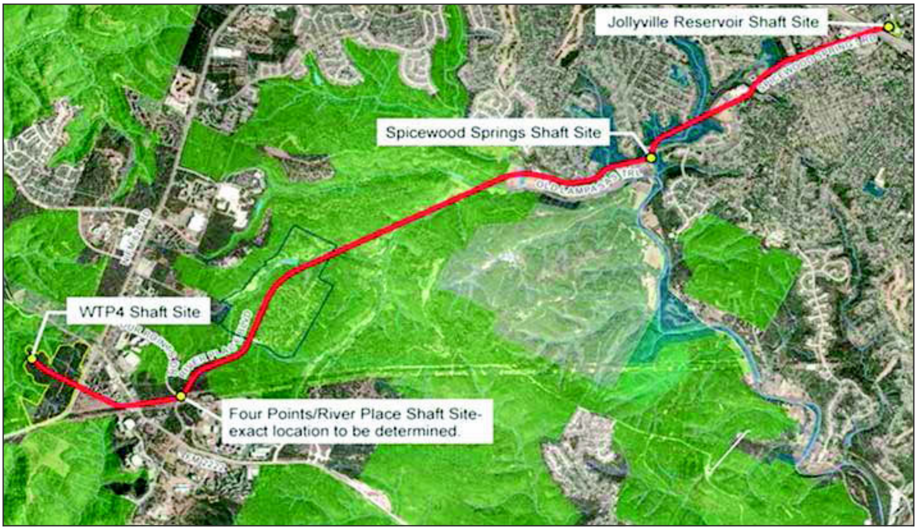

Figure 1. Project alignment

PROJECT GEOLOGY AND SUBSURFACE CONDITIONS

The JVTM is located along the dissected edge of the Edwards and Jollyville Plateau physiographic provinces of central Texas, situated between the Llano Basin province to the west and the inactive Balcones Fault Zone (BFZ) and escarpment to the east. The Edwards and Jollyville Plateaus represent the upthrown fault block of the BFZ that has been dissected by the Colorado River and its tributaries.

At a regional level, the sedimentary rock units along the Edwards Plateau generally strike northwest-southwest and dip gently (less than 5°) to the northeast. The two major joint sets in the greater Austin area trend N40°E and N45°W, and two secondary joint sets trend N10°W and N80°E. All of the joint sets have near vertical dips.

A thin soil veneer, mostly less than five feet thick, covers lower Cretaceous carbonate rocks that are divided into three formations. The geologic formations from youngest to oldest are the Edwards, Walnut, and Glen Rose and are described below.

The Edwards Formation is present on the highlands on the southwest and northeast ends of the alignment (Figure 2) reaching a maximum thickness of 110 feet. It is an interbedded sequence of limestone and dolomite. The Edwards is known for its karstic features up to and including caves. This formation provides the habitat for the endangered karst invertebrates. Additionally, a majority of the springs that feed Bull Creek and provide the primary habitat for the threatened Jollyville Plateau Salamander (JPS) emanate from the Edwards or at the Edwards/Walnut Formation contact.

The Walnut Formation is present on the highlands on the southwest and northeast ends of the alignment, reaching a maximum thickness of about 110 feet. The formation consists of a mixture of nodular limestone and argillaceous limestone.

The Glen Rose Formation is present across the length of the alignment, and none of the geotechnical borings penetrated the base of this formation. The Glen Rose, to the depth investigated, consists of interbedded limestone and dolomite, with limestone beds averaging 10 feet in thickness and dolomite beds averaging 5 feet in thickness.

Two groundwater flow regimes are found in the Project Area: (1) a shallow flow system that occurs in the upper parts of the study area primarily within the Edwards Formation, and (2) a deep flow system present in the lower Walnut and Glen Rose Formations.

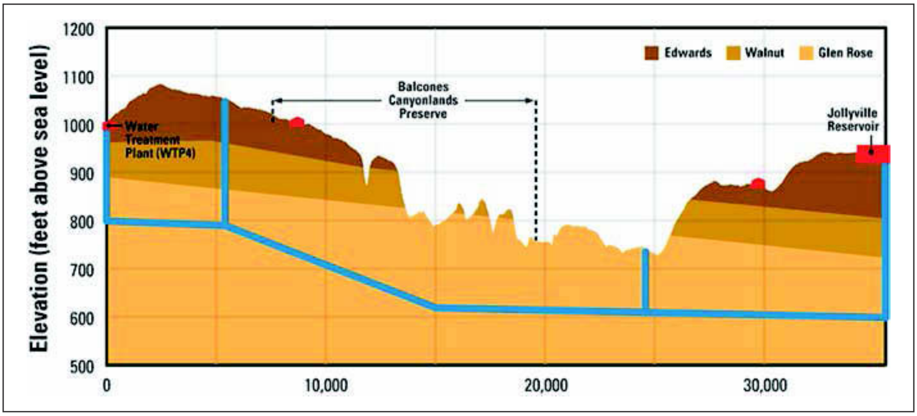

Figure 2. Project profile

Precipitation generally enters the Edwards as recharge, where the formation crops out throughout the Edwards Plateau, and moves downward through the Edwards until encountering a less permeable layer either within the Edwards or at the Walnut contact. It then moves laterally, primarily discharging as springs and seeps along the hillsides found in the Project Area. Many of the springs and seeps occur at outcrops of the Edwards, the Edwards/Walnut contact, and at Walnut bedding planes. Large springs in this area are discharging at less than 10 gpm.

Groundwater flow is primarily away from the topographic highs and parallel to the dip of geologic bedding in the shallow flow system. Dye tracing in the Edwards Formation conducted by City of Austin staff indicated estimated groundwater flow velocities on the order of tens of feet per day, indicating that flow in this shallow flow system is typical of a karst system.

The second groundwater flow regime is the deep flow system within the Walnut and Glen Rose Formations, which is largely hydrologically disconnected from the Edwards Formation as shown by groundwater levels indicating a high downward gradient between the two aquifers. The tunnel alignment was purposely restricted to the lower aquifer, so the shafts are the only possible source of disruption to the environmentally sensitive groundwater feeding JPS springs.

ENVIRONMENTAL SETTING

Environmental Sensitivity

The JVTM alignment passes under several environmentally sensitive areas. On both ends of the alignment are Edwards Aquifer recharge zones where the Edwards Formation is at the ground surface (Figure 2). The Edwards Formation is a Cretaceous age karstic limestone and dolomite with numerous caves identified on or in close proximity to the tunnel alignment. This formation is a primary source of groundwater in portions of central Texas and development activity on recharge zones is carefully managed to protect this vital asset.

The alignment also passes through the Balcones Canyonlands Preserve (BCP), a 24,000 acre nature preserve located within suburban Austin and which has no public access. The BCP was established in 1996 to protect the endangered Yellow-cheeked Warbler, Black-capped Vireo, and six endangered cave invertebrates which include spiders (pseudo-scorpions) and beetles. In addition to these endangered species, there are also 25 cave invertebrates that are "Species of Concern." All these species are protected through the Balcones Canyonlands Conservation Permit (BCCP).

The invertebrate species are restricted to Edwards Formation caves. Additionally, the majority of the alignment is within the Bull Creek watershed, which is almost without exception the only watershed where the threatened Jollyville Plateau Salamander lives. The EPA recommended listing the salamander as an endangered in August 2012, and the official listing is expected to happen prior to the completion of JVTM construction. Design of the JVTM was done with the expectation that this listing would occur so all necessary precautions were designed into the project to avoid delays during construction.

When the BCP was established, and the BCCP was written, there was a provision for an infrastructure corridor through which the JVTM will be constructed. The land set aside in the BCP allows for the "incidental take" of the endangered and threatened cave invertebrates outside the Preserve. Since only shafts will be excavated through the Edwards, take was expected to be insignificant. Additionally, the shaft locations were located outside the primary nesting sites of the two endangered birds. This left the Jollyville Plateau Salamander as the primary species of concern during design and construction of the JVTM.

The JPS is a (gilled) salamander that inhabits Bull Creek and its tributaries. These creeks are spring fed with most of the springs emanating from the Edwards Formation. The springs are critical to the JPS because during the summer months this entire watershed can dry up, and the salamander follows the retreating water back into the karstic rock to live underground until the creek is flowing again. The populations of JPS are concentrated around these springs even during wet periods when the creek is flowing. Not interrupting flow pathways that feed the springs was a major concern during design and construction of the project.

Environmental Commissioning

Understanding the environmental sensitivity of this project, the Austin Water Utility (AWU) and the City's Watershed Protection Department (WPD) organized a working group to develop a plan to minimize and mitigate environmental effects of the project and its construction on surrounding natural areas and natural resources. The Group's efforts were carried forward into an "Environmental Commissioning" (EC) Plan that defined a process under which the project would be completed.

The EC Plan was developed with an overarching goal of "providing a compliance road map that prioritizes avoiding and minimizing environmental impacts." The intent of the EC Plan was to lay out a process whereby environmental goals are established and an ongoing auditing process is then used to help guide the project team towards obtaining those goals through all phases of the project from planning through operation. An EC Team was formed that included biologists, hydrogeologists and engineers from the WPD, as well as members of the Public Works Department that would be managing the project, and representatives of the design team. The EC process occurs throughout the project and involves meetings, reviews, training, oversight inspection, monitoring, and other tasks requiring a collaborative effort between all the parties on the EC Team.

The environmental goals established for this project present the nature and level of environmental protection beyond typical federal, state and local regulatory requirements that will be targeted. The goals defined in the EC Plan include the following:

- Prevent adverse impacts to water quality.
- Maintain existing hydrologic regimes.
- Prevent discharge of pollutants from the sites.
- Meet or exceed the requirements of the BCP permit.
- Avoid, minimize, and mitigate impacts to threatened or endangered species, and species of concern.

- Avoid, minimize, and mitigate impacts to the Jollyville Plateau Salamander.

These broad goals cover a host of protective measures from storm water runoff to tunnel inflow control.

The entities that were responsible for completing this project had various goals including: Public Works staff responsible for completing the project on time and within budget, the Environmental Commissioning Team responsible for meeting the stated environmental goals without regard for budget or schedule, and the Black & Veatch design team balancing the goals of both entities to develop both biddable and constructible documents while providing appropriate environmental protections.

Environmental Commissioning During Construction

During construction of the project the EC Team has been tasked with monitoring surface and groundwater to determine if any impacts are observed that may be due to construction activities. This includes water quality and water quantity, which consists of monitoring wells along the alignment, springs, and stream flow stations. To stay abreast of progress, the EC Team meets on a bi-weekly basis to go over environmental concerns, and geologists on the Team enter the active tunnel headings at least once a week to observe conditions.

Inflow into the excavations was a major consideration during design, and through the EC process, controls were included to minimize this potential. Additionally, inflow "triggers" were set for the shafts and tunnel that would require additional measures to be undertaken to bring flows back down to acceptable levels. These triggers were set based on assumptions of hydraulic conductivity, transmissivity, and storativity.

A process referred to as "Adaptive Management" was also put into place during construction to react to unforeseen or unmanageable events that may potentially harm the environment. This process brings the entire EC Team together to decide a path forward, and to engage experts if warranted. The Adaptive Management process was put into action when monitoring of the groundwater at the Four Points shaft dropped and unexpectedly did not recover following installation of water control measures, as will be described below.

FOUR POINTS AREA SHAFT

The FPA shaft was excavated at a diameter of 36 feet using a hydraulic impact hammer through limestone and dolomite of the Edwards, Walnut, and Glen Rose formations. As required by design, the support system for the top 192 feet consisted of gasketed steel liner plate with an 8–12 in. grouted annulus. The contractor chose to use 10 gage, two-flange steel liner plate. The intent of the gasketed liner plate and grout system was to prevent water inflow from the critical Edwards aquifer into the shaft. Below the liner plate, once the shaft was no longer in the critical zone, rocks bolts and wire mesh was used to the final depth of 270 feet below ground surface.

The Geotechnical Baseline Report (GBR) called for "permeable rings" at elevations of contact between the Edwards and Walnut formations, the Walnut and Glen Rose formations, and anywhere else where the City geologist determined there was significant karstic flow into the shaft. A permeable (or gravel) ring is an area of over-excavation where the extra annulus is be filled with gravel and two sets of liner plates are built over the gravel, shown in Figure 3. The annulus between liner plates is then grouted, in an effort to create a water tight seal behind which there is a permeable pathway for karstic water to flow around the shaft. Only the gravel ring at the contact between the Edwards and Walnut formations was determined to be necessary and was constructed at a depth of 78–87 feet below ground surface in November, 2011.

Given the nature of the environmental concerns for this site, two wells were continuously monitored in the area during shaft construction, JT-112 and JT-128. JT-128

Figure 3. Permeable ring

is situated about 15 feet from the edge of Four Points shaft, while JT-112 is positioned approximately 100 feet away. Readings were taken continuously and uploaded to a website such that water level results could be obtained whenever they were needed. This allowed for real time tracking of the response of the aquifer to shaft construction, as well as for evidence of whether the permeable ring was working as intended or not.

SEEPAGE INTO FOUR POINTS SHAFT AND THE FIRST PERMEABLE RING TEST

Shortly after construction of the permeable ring was completed, discrete inflows became noticeable at the elevation corresponding to the bottom of the ring, evidenced by water flows down the liner plate. The total inflows though the liner plate were estimated at approximately 1–2 gpm, which was well below the trigger level of 10 gpm stipulated in the contract documents. However, well monitoring data showed that JT-128 was dropping at a rate of approximately 0.005 feet per day and was not recovering.

A test was performed on the permeable ring on December 17, 2011. This test was anticipated during the construction of the ring and as such, a standpipe was installed that ran from the shaft collar down into the permeable ring. Using an environmentally safe dye, the test water was dyed green and pumped into the permeable ring with a small pump that tested at a rate of 33 gallons per minute. The goal was to inject 3000 gallons total into the permeable ring and observe how much seepage came into the shaft, if the wells recovered, and if any green water could be pulled from the wells. At the pump rate, the test took 1.5 hours to complete.

After injecting the water for about 15 minutes, the green water was visibly running out of the same elevation that the seeps had been seen, a depth of 87 feet below ground surface. As the water continued to be pumped in, the flow of green water increased and began cascading down into the shaft almost circumferentially around the shaft at the same depth, as shown in Figure 4. Two valves had been installed in the standpipe in order to be able to check the water level in the permeable ring. Upon opening the valves, no water was found. The wells showed no evidence of recovery, and no green water was recovered from the wells.

In light of the continued dropping of the water level in the well, visible seeps from the liner plate, and the results of the permeable ring test, the City of Austin in collaboration with the CMAR and the designer, and upon the suggestion of the EC Team,

Figure 4. First permeable ring test

decided to direct the contractor to perform an enhanced grouting program in an attempt to seal off the seepage.

DESIGN OF ENHANCED GROUTING

The Project team consulted with industry leaders in grouting and shaft remediation to come up with a solution to the shaft seepage. Initially, efforts to come up with a solution were met with some difficulty, and helping the team realize that these seemingly minor seeps were a serious concern was a necessary first step. However, once this understanding was reached, solutions flowed freely. They ranged from water proof membrane covered by a cast in place liner combined with an extensive contact grout program, to just chinking the visibly leaking joints in the liner plate. The only consensus was that a guaranteed leak proof shaft at this stage of the construction would be expensive and take several weeks if not months to complete. This posed a problem given budget and schedule constraints.

Meetings and discussions continued to clearly define the scope of the problem and also to determine just how much could be allocated to fix it. To add to the urgency, local media reported on the story and the pressure for a quick, effective solution grew. To the credit of all involved, the discussions stayed strictly focused on solving the problem. There was none of the fingerpointing, risk avoidance or claim positioning that can kill the effectiveness of any planning.

First, based on the need for a rapid solution, options with long lead times such as concrete lining the shaft were eliminated. Second, given the seriousness of the problem, options that appeared to be quick “band-aid” solutions were eliminated as well. All this led the team to focus on sealing the existing liner plate. While potential means and methods were being collected by part of the team, the other half worked on determining the likely location and nature of the water seeps.

The teamwork between designer, owner, contractor, CMAR, and the EC Team was key to coming up with a rapid solution. Additionally, everyone except the contractor had participated in the initial design and was aware of the reasoning behind some of the unique design features of the shaft lining. This understanding saved a substantial amount of time when evaluating means and methods.

When considering solutions, urethane based chemical grouts injected behind the liner plate quickly became the preferred method. Other ideas such as spray on coatings were rejected because of concerns about surface preparation. Shotcrete as a possibility was rejected because of safety concerns over spalling in a deep shaft. Another concern was that any liner installed inside the shaft would need to resist substantial fluid pressure. Since the steel liner plate and grout was already in place it was decided to just concentrate on sealing that system. Even though the liner plates were gasketed, each seam represented an opportunity for a leak. The challenge now was to design a grouting program to ensure that the seams were all water tight.

Urethane based grout was preferred over other methods for several reasons. First, these grouts were proven to seal water inflows with substantial application history. Second, they can be installed at lower pressures but, unlike cement based grout; will expand to fill all voids. The injection pressure was of great concern since not only could the liner plates not handle a significant injection pressure, but there was also great concern about filling voids in the rock strata. Finally, urethane grouts are relatively simple to mix and inject, and are also considered non-toxic in that it is ANSI/NSF approved for contact with potable water.

Urethane based grouts come in two different types, hydrophilic and hydrophobic. While some of the single component hydrophilic grouts are the simplest to install, they require continuous water flow for activation as well as to sustain the reaction. For very low flow situations such as this, it is very difficult to predict how far they will travel along the crack or what quality the seal will be. Hydrophobic grouts, on the other hand, need the presence of water for the initial reaction but will then continue to set for a specified time. The set time and initial reaction time from first contact with water is controlled by the mix, as opposed to the amount of water available for reaction like hydrophilic grouts. For that reason it was decided a hydrophobic grout injected at low pressure was the best option.

Based on injection hole spacing recommendations from the manufacturer it was decided that a maximum set time of 30 seconds would be good starting point for the hydrophobic grout. This was expected to give the grout sufficient time to penetrate the distance between injection points but not penetrate further into the critical rock strata and permeable ring. This also yielded an expansion ratio of at least 15 times its original volume which was expected to be sufficient to seal the anticipated voids. Because precise control of the injection mix and pressure was critical to preventing penetration into the rock, it was decided that a subcontractor who specialized in this type of grouting should be used. By this decision, the lead time associated with trial injections and test batches typically associated with this type of grouting could be eliminated.

Working closely with the manufacturer, a basic performance specification was quickly developed for use of DeNeef Hydro Active Cut urethane grout. The intent of this specification was to provide clear guidelines for the subcontractor to eliminate risk of damaging critical environmental features. The contractor would be free to vary hole spacing and injection procedures to cut off water as long as he stayed within limits that protected the surrounding rock.

Further investigation of the inflows indicated that they were occurring primarily at the cold joint between grout placements behind the liner plates. Based on these observations, as well as confidence in the methods used by the contractor, it was decided that sealing this area might be all that was required. Thus the grouting plan was revised to first address this joint and then seal other areas if required. A schematic of the conceptual design is shown in Figure 5.

Coming up with a clear plan for the subcontractor was only a part of the preparation required. To make sure the EC Team members charged with ensuring the protection of the environmental features understood and approved of the plan was just as important. They were responsible for assuring the public, media and city council members that all reasonable actions were being taken to prevent environmental disasters. Without EC Team consensus the corrective actions could end up being worse than doing nothing.

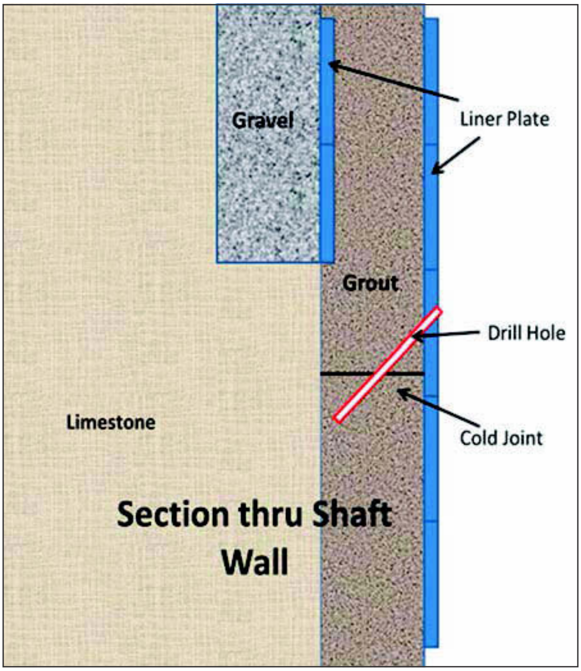

Figure 5. Design of cold joint grouting

Fortunately, relationships and time spent during the design process made this step relatively simple.

EXECUTION

The subcontractor chosen to perform the shaft grouting was Epoxy Design Systems (EDS) out of Houston, Texas. EDS performs various types of infrastructure rehabilitation, and has extensive experience with the use of chemical grout and epoxy resin injection.

As per the design plan, EDS used DeNeef Hydro Active Cut polyurethane, a hydrophobic urethane based grout. The grout utilized a catalyst, DeNeef Cut Cat F, added to it in order to control the reaction. The product was formulated to give a 30 second set time, the maximum allowed by the design, as this was the time EDS decided to be most beneficial. This means that after the EDS mixed the catalyst into the grout, any moisture introduced to the mixture would cause the initial reaction to begin 30 seconds later. Because of this, once the catalyst was mixed into the grout, care would have to be taken to not allow any moisture introduction until injection.

Before work could begin on sealing the seepage, the cold joint location had to be confirmed. EDS used pneumatic grinders to cut out small holes in the liner plate in order to look at the grout in the annulus of the area below the gravel ring. The cold joint was found to have up to approximately 2” of separation between the grout lifts on the west side of the shaft, to as tight as approximately ¼” on the east side of the shaft. The appearance of the separation on the west side seemed to indicate that the separation had potentially been enlarged significantly by the groundwater movement through the joint. There was significant calcium buildup and fine aggregate, presumable from the grout, in the space. Another potential cause could have been the wood fibers that were used to seal the bottom of the uppermost lift didn’t allow for the grout to create a flush

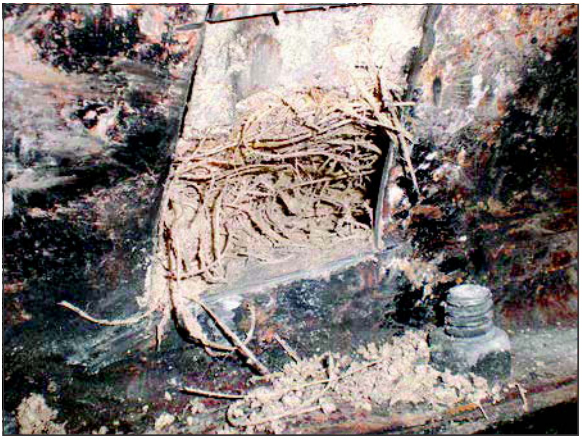

Figure 6. Cold joint

joint, shown in Figure 6. Whatever the cause, this evidence did seem to confirm that the cold joint was the primary source for the groundwater seepage through the liner plates.

Once the location of the cold joint was confirmed, EDS began work on the remediation. Working out of a personnel basket supplied by Southland/Mole, EDS used the pneumatic grinders to cut out 55 holes around the perimeter of the shaft. The holes were spaced approximately 2 feet apart, sized around 2"×2" and were placed a few inches above the cold joint. This spacing was selected based on EDS' experience of the grout flow rates and the evidence of the separation of the joint.

After EDS completed cutting holes out of the steel liner plate, they used a pneumatic hand drill with an 18" long, ⅝" diameter drill bit to drill holes in the exposed grout. In order to ensure that the grout would permeate the entire cold joint, the holes were drilled from slightly above the joint at roughly a 45 degree angle, across the joint, and slightly into the lowermost grout lift as called for in the design. This step was critical, because it was important that the drill hole not intersect the actual gravel ring, or connect to it in any way. Doing so could potentially cause the inadvertent grouting of the gravel ring itself. The avoidance of this possibility was controlled by the hole angle; as well as the set time of the grout, which limited the flow distance; and the injection pressure. Upon completion of drilling, EDS installed a grout port, shown in Figure 7, in each hole and washed out each hole.

The final step was the actual grout injection. EDS brought 5 gallon buckets of the Hydro Active Cut and 32 oz. containers of Cut Cat F down in the personnel basket, along with the pump (Figure 8). One 32-oz container of catalyst was mixed into each 5 gallon bucket of grout as they were used. This particular ratio ensured that the grout would be fully set within around 2 minutes of contact with water, and would expand to 15–20 times. After the resin was properly mixed, EDS injected each hole using a Graco Merkur pneumatic spray pump.

The behavior of the visible seeps, as well as the grout take, gave indications during injection that the grout was working as desired and was not filling into the gravel ring. The seeps were visibly drier after grouting than before, and the grout take along the west quarter of the shaft, where the window had revealed a larger joint separation, took about 50% more grout than along the east quarter.

Figure 7. Grout port

Figure 8. Pneumatic spray pump

Figure 9. Well data for JT-128

SECOND PERMEABLE RING TEST

A test was performed on the permeable ring on January 27, 2012. The test was done approximately 3 hours after completion of grouting, in order to give the grout sufficient time to gain strength so the pressure would not affect the seal. The test water was dyed green again, and pumped into the permeable ring with a small pump at a tested rate of 33 gallons per minute. This would give the desired 6,000 gallons injected into the permeable ring. The test took approximately 3 hours to complete.

Initially, there was no visible seepage from anywhere around the circumference of the shaft at the elevation of the grouting. After about 27 minutes of injection, one area displayed a few drips. A few other areas also appeared as more water was injected. After one hour, the water level was checked in the permeable ring with the valve that had been installed in the standpipe. This check confirmed that the ring was filled to about 18” in height. At the end of the 3 hour test, there were around 8–10 areas that displayed very slow drips, and one spot that had a small, continuous flow. However, there real success of the grouting was measured by the response of well JT-128 (Figure 9). The real time internet updating system allowed the well response to be obtained at the end of the test. The well had recovered 0.6 ft. in the course of the test. By the end of the day, the well had recovered about 1.7 ft., and over the course of the next few days, the well equalized to about 1 ft. above where it had been before the test, afterward climbing steadily at an approximate rate of 0.3 ft. per day. Ultimately, the well leveled off about 2 ft. above the point it had been prior to grouting. Additionally, sampling of down gradient wells showed that the dye travelled through the aquifer along the same pathways as was seen prior to shaft excavation. For the EC Team, this was the real success of the effort.

CONCLUSION

The use of special grouting technology at the Four Points shaft helped to solve a unique challenge that presented itself within sensitive parameters. The importance of protecting critical environmental features was recognized by all involved parties, which facilitated the successful and rapid implementation of the remediation program. Based on the recovery and ultimate equalization of JT-128, as well as a visible reduction of seepage in the shaft, the EC Team agreed that the grouting was an ultimate success, and this was a testament to the teamwork and ingenuity of the Project team to come up with a cost effective and workable solution to an uncommon challenge.

GROUNDWATER INFLOW CHARACTERIZATION FOR A TUNNEL CONSTRUCTED ADJACENT TO AN EXISTING CONCRETE-LINED PRESSURE TUNNEL

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ABSTRACT

The new 7 m diameter (23 ft) Gorge 2nd Tunnel will be constructed next to an operating hydropower tunnel. Observations during design investigations and existing tunnel inspections indicate hydraulic connectivity between the host bedrock and existing tunnel. This paper discusses the approach used to pretdict groundwater inflows into the new tunnel excavation, including analyses to evaluate influence of a constant head source in the existing tunnel on construction inflows into the new tunnel. The approach includes finite element modeling and statistical risk analysis to assess hydraulic input uncertainty. Results are used to design a groundwater management program—including pre-excavation grouting and a water collection and storage system.

INTRODUCTION

The Gorge Powerhouse is located on the upper reaches of the Skagit River, in northern Washington State, and is one of three generating facilities operated by Seattle City Light (SCL) as part of the Skagit Hydroelectric Project. The project location is shown in Figure 1. The facilities at Gorge Dam began providing power to Seattle in 1924. The original tunnel, still in service to this day, conveys water to three turbine units. An additional penstock and a turbine unit were added in the 1950s to increase power generation. The powerhouse is capable of generating 176 megawatts (MW) at a gross head of 116 m (380 ft). This represents approximately 10% of Seattle City Light's total generation capability. The nameplate-generating capacity is 207.5 MW.

The Gorge 2nd Tunnel (G2T) Project will bore a second power tunnel through rock between the Gorge Dam and the Gorge Powerhouse to increase the efficiency of the

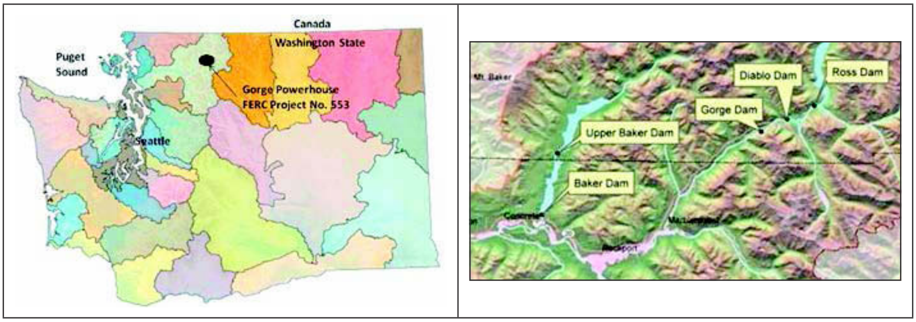

Figure 1. Site location map

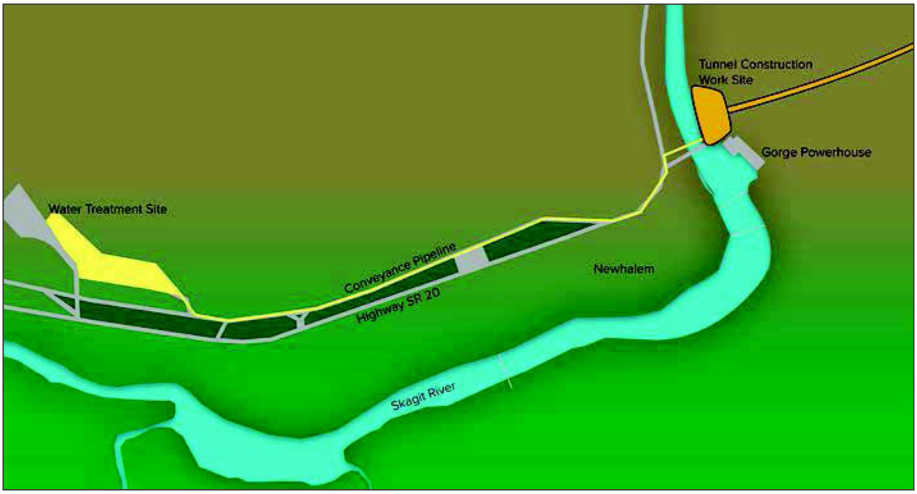

Figure 2. Water management layout

powerhouse. The G2T will be a largely unlined, 7 m diameter (23 ft) tunnel bored parallel to the existing tunnel. The new tunnel will reduce frictional head loss of the water during tunnel conveyance, raise the head pressure at the turbines, increase the torque on the generators, and produce more power for any given flow.

The tunnel construction portal is situated within a bench located adjacent to the Skagit River. One of the portal constraint challenges is water management, particularly in dealing with water from the tunnel during construction, such as when encountering water-bearing zones, which could produce a significant volume of inflow. Large inflows have the potential to overwhelm the treatment facilities and overflow into the river. The tunnel water treatment system was located to a site across the river so that when unexpected flows occur, they do not overflow into the river. The conveyance system is shown in Figure 2.

Estimates and management of tunnel discharge waters are critical design components of this project to protect the Skagit River. This included designing conveyance pipeline from the portal to the treatment area that could handle a large inundation. Understanding the potential magnitude and flow characteristics of these waters was necessary in order to design a system to attenuate, treat, and discharge them. This paper discusses the information collected and methods used to evaluate input parameters such as hydraulic conductivity and to estimate discharge characteristics including peak and average flows during construction.

As part of this analysis, the constant head effects of the adjacent tunnel were also considered. The presence of this tunnel was used in modeling both potential formation hydraulic conductivities as well as addressing potential effects on tunnel inflows during construction.

GEOLOGY

The project area is located in the metamorphic core of the North Cascades range of north-central Washington State. The Gorge Powerhouse facilities are located in the glacially carved Skagit River valley. The area's geomorphology is largely defined by steep bedrock valleys that were modified by glaciers. The bedrock along the tunnel alignment consists primarily of orthogneiss, a foliated (mineralogically banded), metamorphosed

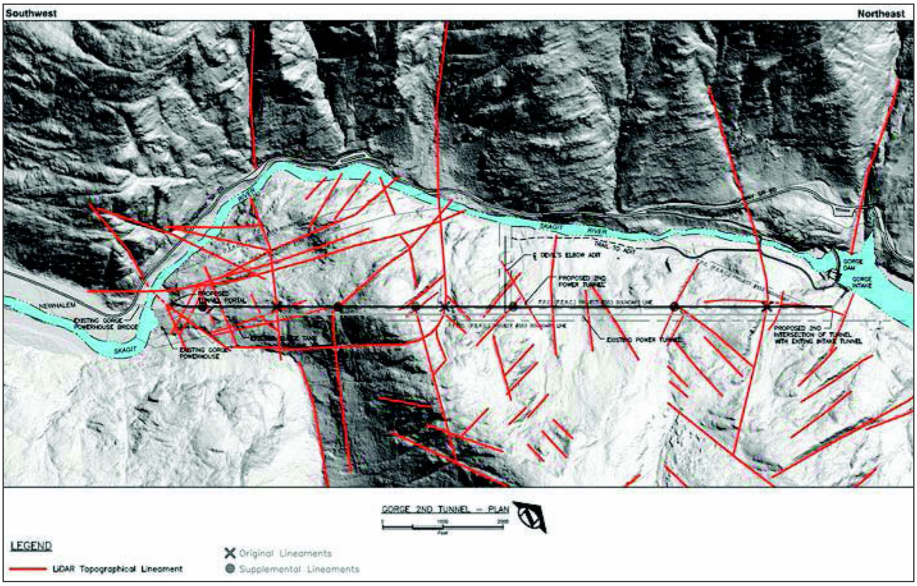

Figure 3. Plan view of G2T project showing faults and lineaments. Xs mark the locations of the larger faults modeled and circles mark the location of the minor faults modeled. (Bbase map is a LiDAR image.)

igneous rock of granitic composition that crops out mostly between Newhalem and Ross Lake, to the east.

The bedrock at the project site has been thoroughly metamorphosed and exposed to various episodes of faulting and shearing, which has created zones of weakness and potentially higher hydraulic conductivity that were incorporated into this analysis as discussed below. Potential project area faults that cross the tunnel alignment were identified through a review of project area topographic maps and aerial photography. Faults commonly form linear depressions such as valleys, canyons, small depressions, and notches along ridgelines, and linear faces on bedrock outcrops that are visible in maps and photographs. The fractured and sheared rock caused by faulting is commonly softer and more easily eroded, resulting in the formation of depressions along the fault trace. Based on the review of project area topographic features, three northwest-southeast trending potential faults and multiple minor lineaments were identified crossing the tunnel alignment. Lineament concentration was generally higher in the western portion of the alignment.

HYDROGEOLOGY

Groundwater inflow into the tunnel will occur through joints, fractures, and shear zones in the rock as intact rock has a very low hydraulic conductivity. As part of the site investigation, piezometers were installed in borings B-2, B-3, and B-5. The location of these borings is shown in Figure 3. Readings from these piezometers indicate the head along the alignment increases from east to west, with head readings that approximately reflect the head in the adjacent existing tunnel.

Connectivity between the piezometers and the existing tunnel was further tested during a 2010 inspection of the existing tunnel. This tunnel was drained as part of the inspection, effectively eliminating the influence of this recharge source. Groundwater levels in two of the three piezometers remained relatively unchanged; however, head in the third piezometer, located in B-2, dropped over 46 m (150 ft). This boring is located near one of the larger faults along the project. Significant leakage into the drained tunnel was also recorded in this section.

Estimates of hydraulic conductivity were developed based on the results of the field investigation. Packer tests were conducted in eight locations as part of the site investigation. These results indicated that fracture zones had hydraulic conductivities up to and potentially greater than 1×10^{-1} cm/sec; however, in some cases test results were compromised by equipment limitations. An intact rock test returned a hydraulic conductivity of 2×10^{-6} cm/sec.

During drilling, artesian conditions were encountered in subhorizontal borings B-1 and B-3, which are located at the west end and central portions of the alignment, respectively (Figure 3). Flows from B-1 were initially in excess of 189 liters per minute (Lpm; 50 gpm), stabilizing to approximately 114 Lpm (30 gpm) for several weeks, until the borehole was grouted. Flows in B-3 were less, initially 38 Lpm (10 gpm), lowering to 11 Lpm (3 gpm).

Flowing conditions in boring B-1 at a depth of approximately 98 m (320 ft) were modeled using the Rocscience two-dimensional finite element model Phase2® to back calculate a hydraulic conductivity for this high permeability zone (Figure 4). The finite element model was set up with triangular node distribution and a graded mesh around boundaries to improve accuracy. The following boundary conditions were used to establish predrilling conditions:

- Tunnel invert and borehole intercept were both at approximately elevation 165 m (540 ft).
- High permeability zone was in hydraulic connectivity with the existing tunnel and acted as a porous media.
- The flow rate of 114 Lpm represented steady state conditions.
- All flow came from the high permeability zone less than 0.3 m thick (1 ft).
- Regional water table was at approximately elevation 183 m (600 ft).
- Constant head within the tunnel was approximately 259 m (850 ft).
- Leakage from the existing tunnel developed a localized mounding of groundwater.

The effect of a pressurized tunnel is to mound the groundwater to near 226 m (740 ft) above the tunnel. To back calculate a hydraulic conductivity, the borehole was modeled as a zero head boundary, essentially acting as a pipe out of the model. Flow rates across this boundary were measured and the hydraulic conductivity varied until discharge rates were equivalent to approximately 114 Lpm (30 gpm). The modeled formation hydraulic conductivity was 8×10^{-3} cm/sec, Figure 5 presents the modeling results.

MODELING APPROACH

Groundwater inflow into the tunnel will occur through joints, fractures, and shear zones in the rock. Inflows of groundwater into the new tunnel will flow back to the portal, where they will be collected and conveyed to treatment facilities. Inflow predictions are provided as a basis for planning and design development. The reliability of groundwater inflow predictions is strongly influenced by the accuracy of the input parameters: conductivity, head, and specific yield. The uncertainty of encountering undefined reaches of higher permeability (e.g., faults, open joints) must also be considered because inflow predictions are very sensitive to variations at the high end of the permeability range. Natural variability in the parameters along the tunnel creates additional uncertainties as to the representative values to use in the analysis.

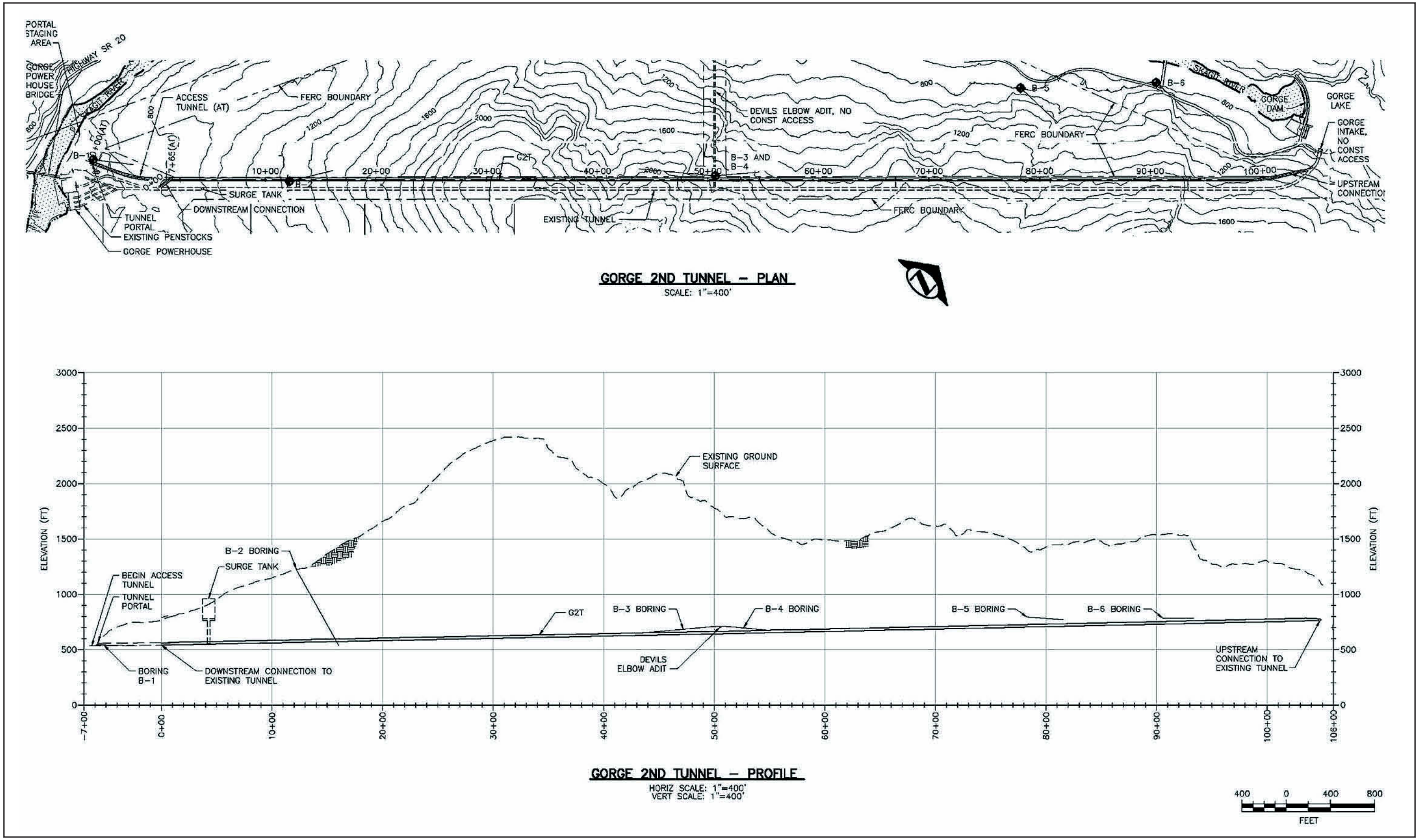

Figure 4. Tunnel plan and profile with borehole location

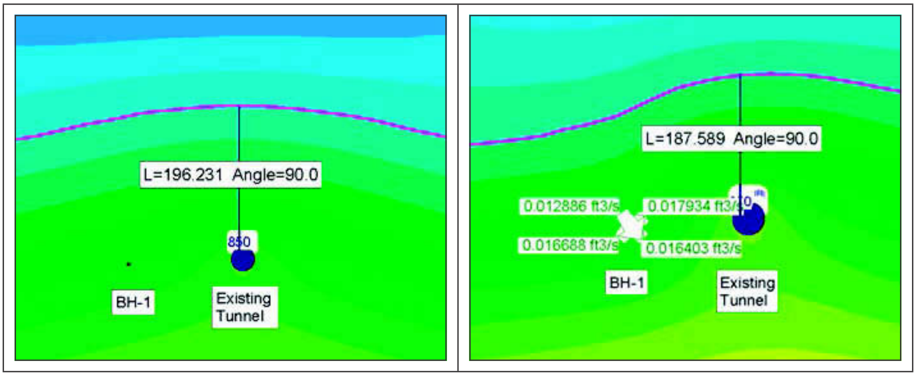

Figure 5. Image on the left is steady-state conditions without a boundary condition at BH-1. The image on the right presents the effect of a zero head boundary at the boring.

The impact of uncertainty on predicted inflows is addressed explicitly using risk analysis. This approach, initially developed by Golder (2001), allows a rational assessment of the impact that the likely occurrence rates of all inputs will have on tunnel inflows. A Monte Carlo simulation is used to evaluate the combined impact of variations on heading inflow and sustained portal inflow as the tunnel is advanced. The risk analysis program @Risk (Version 5.0) was used to perform the Monte Carlo simulation.

Two types of tunnel inflow are analyzed: (1) maximum instantaneous inflow (heading inflow) as the tunnel is driven, and (2) the sustained flow (cumulative inflow) out of the portal as a result of all inflows along the tunnel. The analysis mimics the anticipated tunnel drive as a single heading from the portal. As the tunnel is advanced, water inflows fluctuate as water-bearing features are encountered and previous features dry up or become steady state inflows. High yield features such as faults or fractures are assumed in the model. The tunnel excavation is divided into 23 m long (75 ft) intervals on which inflow calculations are made independently. An interval is based on preliminary estimates of tunnel boring machine (TBM) daily advance rates.

At any given time, the heading inflow (Q_o) is calculated using Goodman's steady-state solution (Equation 1, Freeze and Cherry at 490, 1979).

$$Q_o = \frac{2\pi k H_o}{2.3 \log\left(\frac{2H_o}{r}\right)} \tag{1}$$

Q_o = inflow rate per unit tunnel length
k = hydraulic conductivity
H_o = groundwater head above tunnel centerline
r = tunnel radius

Goodman's transient solution (Equation 2, ibid., at 491) is used to calculate the **cumulative** inflow (Q(t)) into all intervals behind the heading with respect to the excavation time:

$$Q(t) = \sqrt{8\frac{C}{3} k H_o^3 S_y t} \tag{2}$$

$Q(t)$ = cumulative inflow per unit tunnel length at time t
k = hydraulic conductivity
H_o = initial groundwater head above tunnel centerline

C = constant equal to 0.75
S_y = specific yield
t = time since start of inflow into excavation

Heading and cumulative inflows are summed to estimate the total volume of water to be handled at the portal at any one time. The process is repeated for each advance of the heading, which is assumed to occur at a constant upper-bound rate of 23 m per day (75 ft). Stops or slowdowns during construction will result in a reduced portal inflow, as the cumulative inflows have more time to dissipate. The upper-bound excavation rate ensures a conservative inflow prediction. Conservatively, this analysis ignores potential reductions in inflow from pre-excavation grouting, a method used to reduce conductivity in the rock formation ahead of the tunnel heading.

For simplicity, faults are modeled as discrete water-bearing features. These zones are assigned a higher conductivity and specific yield compared to the rest of the tunnel. Once the initial groundwater head is dissipated through each feature, inflows at faults are assumed to be driven by the hydraulic grade line (HGL) in the existing power tunnel.

MODELING INPUT

The inflow analysis requires three site-specific inputs: groundwater head above the tunnel; hydraulic conductivity; and specific yield. Available data are typically used to directly define probability distribution functions for formation conductivity and groundwater head. The site investigation results and hydraulic conductivity analysis presented above were used to develop triangular distribution assumptions for both groundwater head and hydraulic conductivity. Values for specific yield were developed based on general correlations. Faults identified above were included in the model as discrete high flow zones at the approximate location where the surface expression crossed the tunnel alignment.

Groundwater Head

Groundwater head estimates were developed using data from three vibrating wire piezometers installed during the field investigation in borings B-2, B-3, and B-5 (Figure 3). Based on the groundwater monitoring results and the profile surface topography, a table of anticipated groundwater heads was developed along the alignment. Fifteen triangular groundwater head probability distributions were developed for this analysis, ranging from an expected head from 0 at the portals to up to 84 m (275 ft).

Hydraulic Conductivity

Estimates of hydraulic conductivity were developed based on the results of the field investigation, tunnel inspection, and construction records from the first tunnel. As noted above, significant inflows were observed during the 2010 tunnel inspection. Construction records also indicate high flows in the western portion of the tunnel. For the purposes of estimating hydraulic conductivity, the western third of the alignment was modeled with a higher average hydraulic conductivity.

Four separate estimates of hydraulic conductivity were established for the tunnel. An eastern and western average bedrock hydraulic conductivity was established based on the tunnel inspection observations, higher artesian flows, and construction records, with a cutoff at approximately Station 4400. Hydraulic conductivity for faults was also divided on this cutoff, with a higher assumed hydraulic conductivity to the west, reflecting potential inflow from the Ladder Creek fault. The average hydraulic conductivity was assumed to be approximately the rate back calculated from the artesian conditions in borehole B-1, which was where the highest flows were encountered during the subsurface investigation. Maximum hydraulic conductivity values along faults were assigned based on the packer test results. Values for faults to the east were reduced by half. Table 1 summarizes the cumulative distribution functions used in this analysis.

Specific Yield

For modeling purposes the rock was assumed to hold water like an unconfined aquifer. Specific yield is defined as the volume of water that is released from storage per unit surface area of the aquifer per unit decline in the water table (Freeze and Cherry at 61, 1979). Goodman et al. (1965) define specific yield as the volume of drainable voids divided by the total volume. For rock, fractures are assumed to be equivalent to drainable voids. A triangular specific yield probability distribution was used based on a most likely, a minimum, and a maximum value. Specific yields were also estimated for the fault zones. Table 2 summarizes the triangular distribution functions used in the analysis.

MODELING RESULTS

Statistical simulations of 10,000 runs were used to develop inflow predictions. The combined impact of the variation of all input parameters to inflow analysis resulted in large variations between the mean, 95th percentile, and maximum values of inflow. As would be expected, the presence of fault zones has the most significant impact on the estimated inflows. These features store large volumes of water and have high conductivities. Therefore, they contribute large spikes of groundwater inflow.

Empirical Heading Flows

The initial predicted heading inflow is shown in Figure 6. The horizontal axis corresponds to tunnel stationing. This figure shows three curves that correspond to the mean, 95th percentile, and maximum inflow values from the statistical analysis. The left vertical axis shows the heading inflow, while the right vertical axis shows the mean groundwater head.

Maximum average heading inflows in the western third of the tunnel are predicted to be nearly 350 gpm, with the potential of flows approaching 1,893 Lpm (500 gpm) at the intersection of a fault. To the east, average maximum average heading inflows are predicted to be less than 151 Lpm (40 gpm) for the tunnel. Larger heading inflows are predicted to occur in the eastern fault zones, in the range of 795 Lpm (210 gpm). High inflows represent what might be encountered as the TBM mines into a specific

Table 1. Hydraulic conductivity distributions

	Hydraulic Conductivity (cm/sec)		
Geologic Unit	**Minimum**	**Mean**	**Maximum**
Western Bedrock	1.00E-08	5.00E-07	1.00E-05
Eastern Bedrock	1.00E-06	5.00E-05	1.00E-04
Western Fault Zone	1.00E-03	1.00E-02	1.00E-01
Eastern Fault Zone	5.00E-04	5.00E-03	5.00E-02

Table 2. Specific yield distributions

	Specific Yield		
Geologic Unit	**Minimum**	**Most Likely**	**Maximum**
Bedrock	0.0005	0.005	0.009
Fault Zones	0.05	0.1	0.2

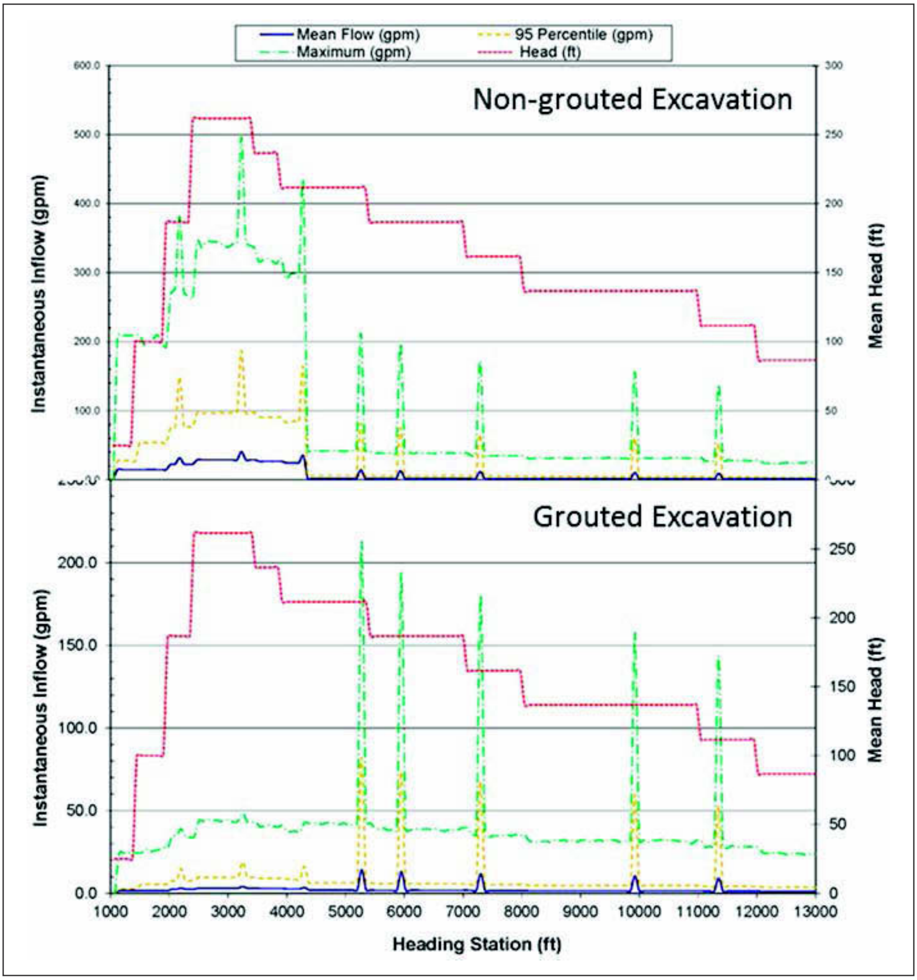

Figure 6. Predicted heading flows with the upper graph presenting estimates without grouting in advance and the lower graph presenting flows assuming grouted conditions

water-producing feature within the fault zone. It is expected that several similar features could be encountered sequentially as the TBM traverses through the length of the fault zone. Such high flows would be difficult and expensive to manage at the heading. Probe hole drilling and pre-excavation grouting in advance of tunneling were added to the project design to address these predicted flows. In order to account for the effects of grouting, the western fault and western bedrock hydraulic conductivity was reduced by one order-of-magnitude. The results of this analysis are presented in Figure 5. While the inflows to the east remain unchanged, inflows in the western portion are reduced to less than 189 Lpm (50 gpm) for average peak flows and flows at fault zones.

Empirical Cumulative Flows

The predicted portal inflow with no pre-excavation grouting is shown in Figure 7. This flow represents the total volume of water to be handled at the portal. The horizontal axis corresponds to tunnel stationing. This figure shows three curves that correspond

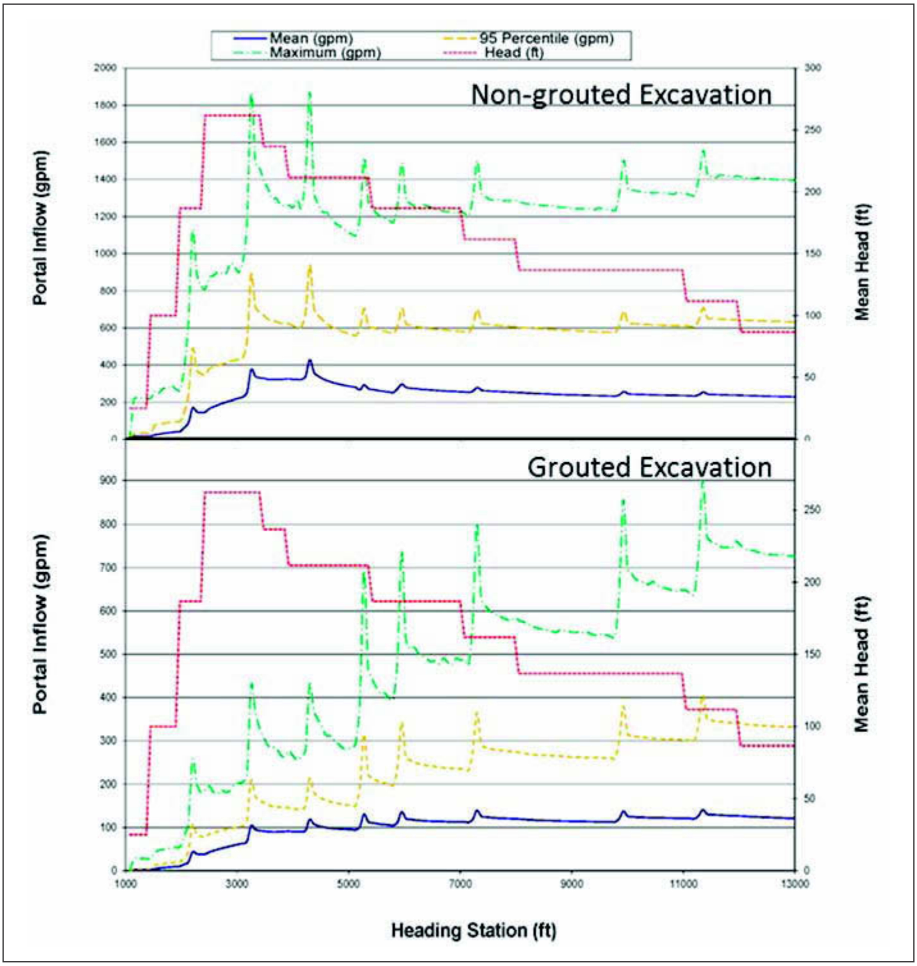

Figure 7. Predicted cumulative flows with the upper graph presenting estimates without grouting in advance and the lower graph presenting flows assuming grouted conditions

to the mean, 95th percentile, and maximum inflow values from the statistical analysis. The left vertical axis shows the portal inflow, while the right vertical axis shows the mean groundwater head. The largest predicted portal inflow occurs as the heading passes through the fault zones. These zones cause the predicted inflow to spike for a short period of time before dropping off to a more sustained flow. The spikes and subsequent drop-off result in an overall gradual increase in the portal inflow as the tunnel is advanced. The spikes are larger than the predicted heading inflows because they capture the effects of the entire fault zone (e.g., numerous open features) as opposed to a specific feature.

Based on this analysis, it can be determined that inflows from the western portion would be quite high, with peak portal flows near 7,078 Lpm (1,870 gpm) and average flows eventually approaching 5,300 Lpm (1,400 gpm).

In order to reduce these values, probe hole drilling and pre-excavation grouting were modeled as described above. Figure 7 presents the revised estimated inflows.

Table 3. Summary of boundary conditions

Section	Regional Groundwater Elev., m (ft)	Tunnel Head Elev., m (ft)	Existing Tunnel Invert Elev., m (ft)	Proposed Tunnel Invert Elev., m (ft)
Station 4+00	183 (600)	259 (850)	168 (550)	166 (544)
Station 14+00	213 (700)	262 (860)	178 (584)	173 (566)
Station 50+62	229 (750)	264 (865)	215 (706)	196 (642)

Table 4. Summary of inflows into the proposed tunnel

Hydraulic Conductivity (cm/sec)	Flows by Station, Lpm (gpm)		
	4+00	14+00	50+62
0.1	2,646 (699)	3,093 (817)	2,108 (557)
0.01	265 (70)	310 (82)	220 (58)
0.008	212 (56)	231 (61)	151 (40)
0.0001	2.6 (0.7)	3.0 (0.8)	2.3 (0.6)

Peak maximum flows at the portal are reduced to 3,407 Lpm (900 gpm), and average maximum flows would eventually reach 2,801 Lpm (740 gpm).

EXISTING TUNNEL INTERACTION

The presence of the existing tunnel operating under a constant head condition represents a site condition not taken into account by the above analysis. While the existing tunnel is lined, the amount of groundwater inflow and tunnel defects observed during the 2010 tunnel inspection indicates the tunnel is relatively leaky in the western third of the tunnel. This assumption is also supported by the response of the vibrating wire piezometer in boring B-2. Actual flows from the existing tunnel will be constrained as flow will be limited to through existing drain holes and tunnel defects. Estimated flows are also useful for comparison with long-term heading inflows calculated empirically.

The finite element software Phase2® was used to model the potential effect of a constant head condition in the vicinity of the proposed tunnel. The finite element model was set up with triangular node distribution and a graded mesh around boundaries to improve accuracy. The three separate locations modeled were:

- Station 4+00: Near maximum head in the existing tunnel relative to the proposed tunnel invert.
- Station 14+00: Near boring BH-2, where response to tunnel dewatering indicates good hydraulic connectivity with the existing tunnel.
- Station 50+62: Near the maximum invert elevation difference between the tunnels and maximum head above the existing tunnel invert.

Boundary conditions were established for each section, including regional groundwater and head in the existing tunnel. The horizontal spacing between the tunnel and existing tunnel is relatively consistent 18 m (60 ft); however, the vertical spacing is variable. Table 3 summarizes the boundary conditions for each of the three sections analyzed.

The proposed tunnel was modeled by assuming a zero pressure head boundary at the tunnel edge. Inflows were measured across this boundary. Hydraulic conductivity was varied from 0.1 cm/sec to 0.0001 cm/sec and includes the value calculated from the BH-1 field observations above. Table 4 summarizes the flows calculated for this analysis and for Figure 8.

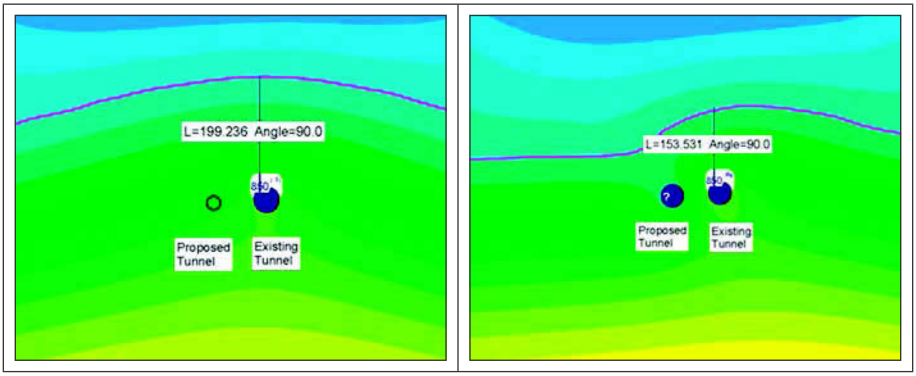

Figure 8. Image on the left is steady state conditions without a boundary condition at the proposed tunnel, Station 4+00. Image on the right presents the effect of a zero head boundary at the new tunnel.

Maximum heading inflows are similar to anticipated peak flows estimated empirically for conditions at the heading, and several times lower than inflows estimated empirically for sustained flows from high permeability features. This suggests that the influence of a constant head condition associated with the existing tunnel is modest. Differences in flows between stations are relatively minor and appear to be a function of total head above the proposed pipeline. As discussed above, actual flows from the existing tunnel will also be limited via drain holes and existing tunnel deficiencies. While a temporary higher head condition may develop because of the presence of the adjacent tunnel, inflows to the tunnel from the existing tunnel should be relatively minor.

CONCLUSIONS

Instantaneous heading inflows and sustained portal inflows have been predicted in relation to tunnel heading position for the Gorge 2nd Tunnel. The predictions use steady-state and transient solutions developed by Goodman et al. (1965). Uncertainty in hydraulic conductivity, head, and specific yield along the alignment were characterized by probability distributions. The impact of the uncertainty in these ground parameters on heading and portal inflows was evaluated using Monte Carlo simulation. The result of the analysis is heading and portal inflow probability distributions for each heading position along the alignment. These predictions are intended to cover the range of likely occurrences of hydraulic conductivity, groundwater head, and specific yield.

The results of the field investigation, adjacent tunnel inspection, and construction records indicate that the western third of the tunnel is expected to have higher average hydraulic conductivities as well as higher values for fault zones. Predicting flows from this portion of the tunnel would be technically difficult and expensive. Therefore, probe hole drilling and pre-excavation grouting were added to the design for the western third in order to reduce inflows from this portion of the alignment. Reduction in conductivity was conservatively estimated at a single order-of-magnitude and may well be greater. The cost of probe grouting is anticipated to be significantly less than the potential cost of water management, including construction delays if flows force the cessation of TBM advancement.

The presence of an existing tunnel was modeled using a two-dimensional finite element model to evaluate the potential influence of a constant head condition near the new tunnel. Results indicate that the influence of this tunnel would be relatively modest and well below empirically calculated values for heading and cumulative flows.

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STATE-OF-THE-ART DRILLING AND GROUTING TO CONTROL GROUNDWATER FOR A SHAFT IN DOLOMITE

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Mark Hampton ▪ Hayward Baker Inc.

Matt Didier ▪ Hayward Baker Inc.

ABSTRACT

Water actuated down-the-hole hammer drilling techniques were used to install grout holes to depths as great as 348 feet for a full-depth grout curtain around the perimeter of the planned concrete-lined wet well gate shaft on the Thornton Composite Reservoir Project. Initial grout mixes and grout curtain closure were determined based on the results of water pressure tests. Grout takes ranged from 4 to 900 gallons per stage. Once installed, the grout curtain allowed excavation and construction of the wet well to proceed without interruption. Throughout the project, water inflows were maintained well below the total specified value of 30 gpm.

BACKGROUND—THE TARP PROJECT

Chicago and 51 of its suburbs are served by combined sewers, in which both sanitary and storm flow are conveyed through the same pipes. As the area was developed and paved, the amount of water entering the sewer system during rain events increased beyond the capacity of the sewers and treatment facilities and the excess combined sewer overflows (CSO) were released into local waterways.

In 1972, The Metropolitan Water Reclamation District of Greater Chicago (MWRD) adopted the Tunnel and Reservoir Plan (TARP) to protect Lake Michigan (the region's drinking water supply) from raw sewage pollution, improve water quality of area rivers and streams, and reduce flood damage.

Phase I of TARP consists of four tunnel systems: Mainstream, Des Plaines, Calumet and Upper Des Plaines. The tunnels were completed between 1975 and 2006 and consist of 306.4 km (109.4 miles) of deep, large-diameter rock tunnels providing 2.3 billion gallons (BG) of volume to capture CSOs. After a storm event, CSOs can be stored until treatment capacity is available.

Phase II of TARP, intended primarily for flood control but also to enhance the pollution control provided by Phase I, consists of three reservoirs: Majewski, McCook, and Thornton (Figure 1). These reservoirs will increase the TARP storage volume to 17.5 BG. (MWRD, 2012).

OVERVIEW OF THORNTON COMPOSITE RESERVOIR

The Thornton Reservoir is located at the end of the Calumet System and is being constructed in two stages. The first stage, the Thornton Transitional Reservoir (a temporary 3.1 BG reservoir) was completed in 2003 in the West Lobe of the Thornton Quarry. The second stage, the Thornton Composite Reservoir, (a permanent 7.9 BG reservoir) will be located in the North Lobe of the Thornton Quarry (Figure 2) and is planned to be completed by 2015.

The project includes several elements which were divided into three contracts. The first addressed preparing the reservoir to receive CSO and included a grout curtain and an associated rock dam and haul tunnel plugs. The second includes installation of connecting tunnels and control gates to convey the CSO from the existing Calumet system to the reservoir. The third portion of the Thornton Composite Reservoir is making a live connection to the existing Thorn Creek Diversion Tunnel to divert that flow into the Thornton Reservoir. This paper focuses on the connecting tunnel and gates, and the 98 m (320 ft) wet well shaft used to access the bifurcation and gate area of the new connecting tunnel (See Figure 3).

GEOLOGY

The project area lies on the eastern flank of the Kankakee Arch which trends northwest across Indiana and Illinois. The bedrock in the project area consists of a 4,000-foot thick lower Paleozoic sequence of dolomite and shale that is underlain by Precambrian granitic rock. The overburden material consists of thin glacial till and valley fill deposits.

The topography of the project area is largely flat. In general, the subsurface conditions along the tunnel alignment consist of dolomite rock mass (of the Niagaran Group of the Silurian System) covered by 3- to 15-foot-thick fill and till deposits.

Locally, three main discontinuity sets exist in the bedrock; two joint sets that are nearly vertical and the third along bedding planes. Joint conditions have been noted to be closed or open; or clay filled or heavily fractured (MWH Americas Inc., 2009).

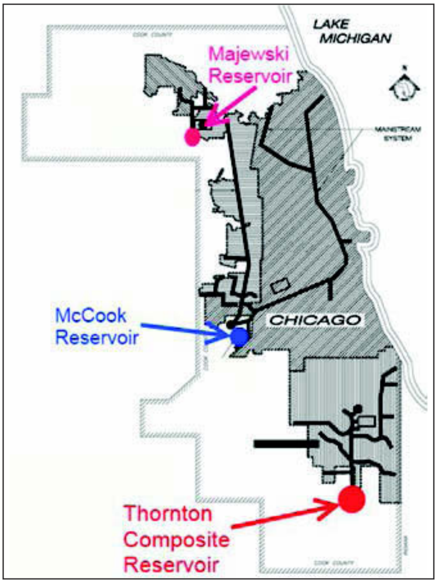

Figure 1. Tunnel and reservoir plan

Figure 2. Aerial view of Thornton Quarry

WET WELL SHAFT GROUTING PLAN

The concrete-lined Wet Well Shaft is 19.2 m (63 ft.) in diameter and extends 103.6 m (340 ft.) below ground surface (328 ft. into the dolomite). There was concern that water conveyed through open fractures in the bedrock would be excessive and interfere with shaft excavation and lining. Grouting around the shaft was specified to limit groundwater inflow to 114 liters per minute (30 gpm) or less.

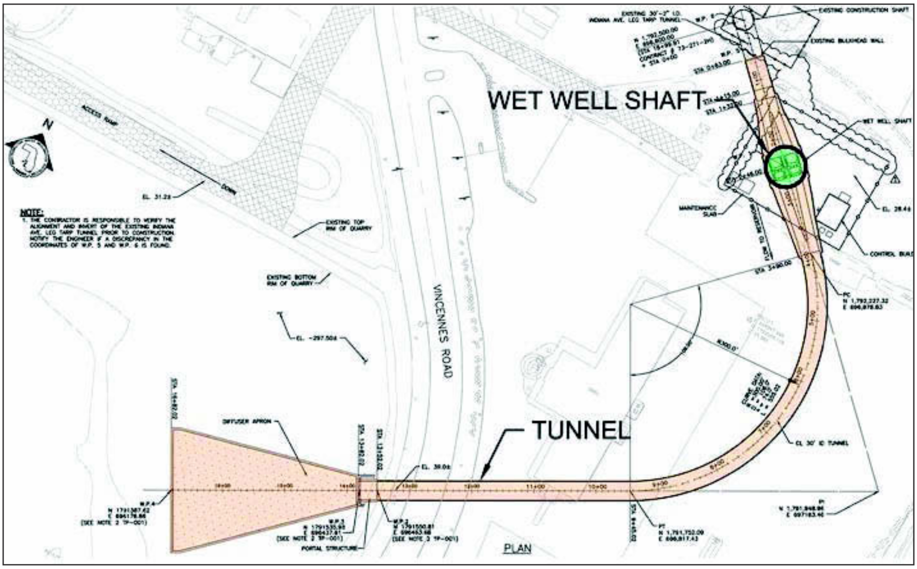

Figure 3. Tunnel plan with gate overview and shaft

Grouting Plan Requested

Contract documents indicated that pre-excavation water control cement grouting was to be completed around the perimeter of the shaft prior to rock excavation to a depth of 25 ft. into bedrock to permeate the fractures in the upper rock. During shaft excavation, if excessive water flow was encountered, excavation was to be stopped and post-excavation cement grouting was to be completed around the shaft in 30 ft. increments. See Figure 4a.

Alternate Grouting Plan Provided

In lieu of pre-excavation grouting of the rock interface and potential post-excavation rock grouting during excavation operations, a 348 ft. full-length grout curtain was installed around the perimeter of the shaft from existing ground elevation to below the bottom of the planned wet well shaft and tunnel invert to prevent potential excavation delays and additional mobilizations of equipment. See Figure 4b.

WORK SEQUENCE

Stand pipes were installed through the overburden prior to rock drilling and grouting. Grout holes were 3⅞-inch (100 mm) diameter and were installed using a 5° batter around the perimeter of the shaft to facilitate intersection of vertical rock joints. Drilling and grouting was performed using a primary-secondary sequence to achieve maximum practical closure. After each primary location was drilled, the hole was washed and water tested prior to grouting. The grout mix design was selected based on the results of the water test, and holes were grouted in a bottom-up sequence using a single packer system in 20 ft. stages.

Drilling

Grout holes were drilled as deep as 348 ft. using a state-of-the-art drill rig equipped with an on-board triplex piston water pump to provide high-pressure water supply for

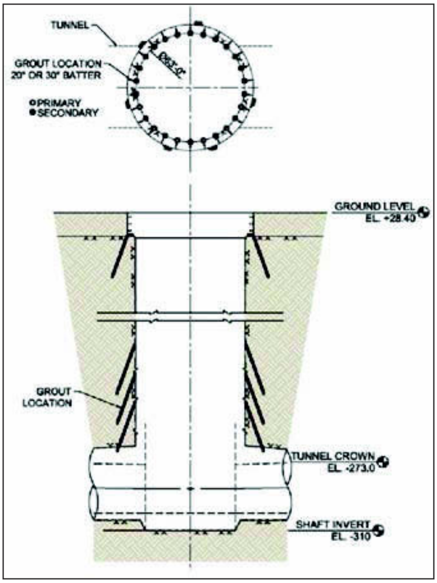

Figure 4a. Grouting plan as requested

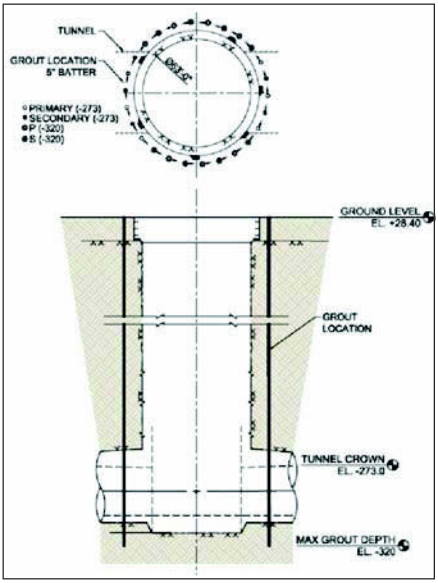

Figure 4b. Grouting plan as completed

a water-actuated down-the-hole hammer (Figure 5). The rig was outfitted with a deck-mounted drill pipe rack to allow drilling as deep as 600 ft. The auto-loading and unloading capabilities of the drill rig greatly limited contact between the personnel and the drill tooling, improving safety of drilling operations. The rig also had real time data logging and wireless data transfer functionality.

Figure 5. State-of-the-art drill rig used on the Thornton Shaft Grouting project

PRESSURE TESTING AND GROUTING

Five balanced and stable cement grout mixes were designed to be used based on fracture conditions encountered in each location. The grouting contractor was able to switch between approved grout mix designs based on the pressure grouting results in each stage and on real time results of the grouting operations. When pressure testing indicated more open conditions, a higher viscosity grout mix was used. Once the proper grout was batched, it was placed in mechanically agitated storage tanks connected to a progressive cavity. The pump supplies grout to mobile injection units which are set up at the drill collar.

Grout injection was performed with a self-propelled mobile injection unit fitted with a hydraulically-powered hose reel with depth encoder to raise and lower the grout hose and packer assembly, high-pressure air storage for packer inflation, and electronic instrumentation including mass flowmeters, pressure transducers, pressure gauges and network hardware for wireless data transmission (Figure 6). Wireless transmission allowed the grouting contractor to remotely control, monitor, and record in real time the

Figure 6. Self-propelled injection unit

Figure 7. On-site iGrout monitoring station

Figure 8. Example of iGrout control screen for water testing and grouting operations

water and grouting flow and pressures with the iGrout Automated Grouting Control and Data Acquisition System.

The iGrout Control application contains the interface for use by the grout technician who controls the pumps and monitors the progress of the grouting and water testing operations. The iGrout interface also accesses to the iGrout Web database which includes report generation, formation of progress drawings, and database management. The iGrout interface also allows the engineer, project manager, job superintendent and others to review the collected data in real time and from anywhere in the world (see Figures 7 and 8).

Figure 9. The excavated shaft with minimal ground water infiltration

RESULTS

A total of 7,000 gallons of grout was injected into the 26 holes. Shaft excavation commenced after completion of all grouting locations. Shaft construction was uninterrupted, in the dry, with minimal ground water infiltration (Figure 9). No additional grouting was required.

CONCLUSIONS

Changing the grouting approach from a minimal pre-excavation grouting with later staged, post-excavation grouting to comprehensive curtain grouting allowed better control of the overall cost of grouting and excavation. Potential, unquantifiable, costly shaft excavation interruptions for drilling and grouting were eliminated, as were costs for multiple equipment and crew mobilizations and demobilizations. In addition, not having to drill and grout from inside the shaft and the associated insertion and removal of personnel and equipment for both operations improved overall project safety.

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PART 10

Hard Rock Tunneling

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DESIGN OF SOUTH HARTFORD CONVEYANCE AND STORAGE TUNNEL

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ABSTRACT

The South Hartford Conveyance and Storage Tunnel (SHCST) is a major component of the Hartford Metropolitan District's Clean Water Project (CWP). This tunnel is intended to capture and store Combined Sewer Overflows (CSO) from the southern portion of Hartford, CT and Sanitary Sewer Overflows (SSO) from West Hartford and Newington, CT. The SHCST project's primary goal is to eliminate SSOs and minimize CSO discharges to receiving waters. This will be accomplished with a series of connection structures, consolidation sewers, manhole structures, tunnel shafts, a shallow rock tunnel, and a deep rock tunnel. After each storm event, a tunnel dewatering pump station will deliver the stored overflow volume to the District's Hartford Water Pollution Control Facility (HWPCF) for treatment. The SHCST will include a tunnel pump station at its eastern end, located adjacent to the existing wastewater treatment facility. It is currently planned that a future deep rock tunnel will be constructed to the north of the SHCST project (North Tunnel), and will convey flow and connect to the currently proposed tunnel. The project is estimated to cost approximately $500 MM and will be constructed in multiple construction contracts. The project components include a deep rock tunnel 21,800 feet in length, 25 feet excavated diameter, several miles of consolidation sewers, multiple hydraulic drop shafts with deaeration chambers and a 27 MGD tunnel dewatering pump station. The rock TBM tunnel will be excavated in shale, siltstone and basalt through several fault zones. AECOM is currently performing the final design for this project. This paper discusses the major aspects of the final design of South Hartford Conveyance and Storage Tunnel.

INTRODUCTION

The South Hartford Conveyance and Storage Tunnel (SHCST) project is a significant component of the Hartford Metropolitan District's (MDC) Long Term Control Plan (LTCP) which is overseen by the Connecticut Department of Energy and Environmental Protection (CTDEEP). This project will address a portion of the MDC's Clean Water Project (CWP), which will reduce combined sewer overflows (CSOs); eliminate sanitary sewer overflows (SSOs); and reduce nitrogen released into the Connecticut River.

The purpose of the SHCST project is to eliminate West Hartford and Newington SSOs, eliminate Franklin Area CSOs discharging to Wethersfield Cove and to minimize CSO discharges to the South Branch Park River. The locations of each overflow are shown in Figure 1.

In 2010, the District prepared a Preliminary Design Report (PDR) for the SHCST project, which included relief of the Folly Brook Trunk Sewer and proposed to keep the retrieval shaft within the City limits of Hartford. Figure 1 shows the 2010 PDR

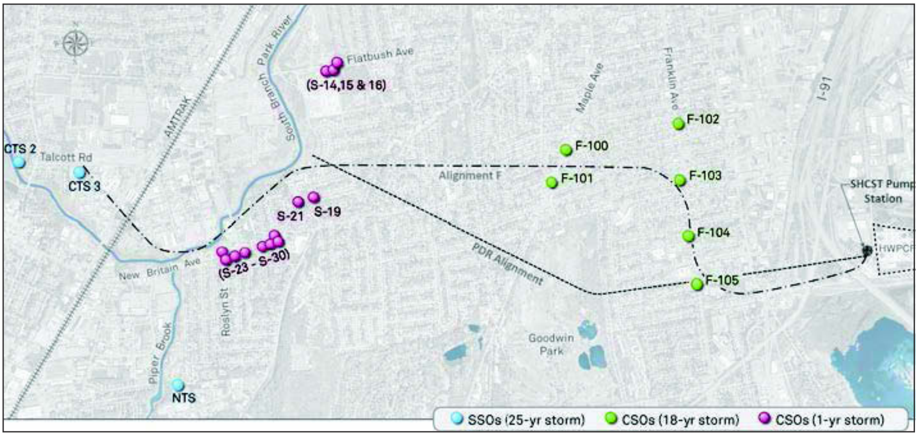

Figure 1. Location of CSOs and SSOs tributary to the SHCST

recommended tunnel route. Subsequent to the PDR, the objectives of the SHCST have slightly shifted. In accordance with ongoing revisions to the LTCP, relief of the Folly Brook Trunk Sewer is no longer necessary. Additionally, the MDC has decided to perform less sewer separation in the Franklin Avenue drainage area. To replace the sewer separation, new relief points are proposed within the Franklin area and will be diverted to the SHCST. Figure 1 also shows the current recommended tunnel route (Alignment F).

The MDC is also conducting a separate study on the overall alignment for the North Tunnel, which is planned for later phases of the overall CWP program. However, a portion of this study is reviewing the potential to connect the North Tunnel to the SHCST. The initial draft conclusion is to make this connection near the western half of the SHCST alignment to avoid the additional capital and O&M cost of a second deep pump station.

During dry weather, the SHCST will not receive flow as the existing MDC collection system can adequately convey flow to the Hartford Water Pollution Control Facility (HWPCF). During wet weather, when the capacity of the existing collection system is exceeded, the SHCST will receive overflows that would have previously discharged directly to receiving waters.

This paper provides a basis of design for the SHCST project. New diversion structures will be constructed at each CSO/SSO relief point to divert overflows to new consolidation sewers (near surface). These, in-turn, will discharge flow to hydraulic drop shafts which will convey the flow in a controlled manner to the deep rock storage tunnel. Once in the tunnel, flow will be pumped to the new headworks at the HWPCF. The components of the SHCST project described in this paper are as follows:

- Deep rock tunnel (22' ID @ 21,800 LF) with a launch shaft near the HWPCF in Hartford and a retrieval shaft in West Hartford
- 12,200 LF of near surface consolidation sewers (24" to 66" in diameter)
- Seven hydraulic drop shafts
- 27 MGD tunnel pump station
- Odor control at all potential air release points.

The sizing of the tunnel was based on the volumes from the 1-year, 18-year and 25-year design storm per the LTCP and updated collection system modeling from the MDC's Program Management Consultant (CDM Smith). The LTCP specified a different

Table 1. Tributary overflows to the SHCST

Contribution	Design Storm	Peak Flow (MGD)	Volume (Mgal)
West Hartford/Newington SSOs	25-yr	27	17
South Branch Park River CSOs	1-yr	68	6
Franklin Area Relief	18-yr	313	39
Total			62

level of control for each tributary area. Table 1 shows the peak flows and volumes to be stored in the SHCST for each major source and respective design storm.

Surge, air entrainment and pressure waves can occur in CSO tunnels filling rapidly, with detrimental results such as geysering, blowback and flow instabilities. Based on the preliminary hydraulic analysis, it appears that surge in the SHCST is unlikely, due to the relatively large tunnel diameter in comparison to the incoming peak flows.

Sediment deposition can present an ongoing maintenance burden if not controlled. An initial sediment deposition analysis and modeling was completed. Based upon this effort, a slope of 0.1% appears adequate for the deep rock tunnel to cost-effectively minimize sediment deposition issues. This slope is consistent with the state of practice for other large diameter CSO tunnels as steeper slopes will increase project cost. The tunnel will still require periodic maintenance to remove sediment build-up over the life of the facility.

An alignment study was conducted to evaluate various configurations of rock tunnels and consolidation conduits. Seven (7) conceptual rock tunnel alignments and associated consolidation conduit options were developed and evaluated. The purpose of this alignment study was to identify a cost effective and acceptable tunnel alignment that balances the expectations of the many stakeholders impacted by the project.

All the alignments began in property owned by the District adjacent to the HWPCF. However three different locations were identified as possible deep rock termination points. Two of these locations were located in space owned by various City of Hartford departments on the east side of the South Branch of the Park River and the third was in an unused parking lot on Talcott Road in a light industrial area on the west side of the river (in West Hartford). This third location significantly reduced the length of consolidation conduits and allowed the South Branch of the Park River to be crossed deeply in rock using the deep rock tunnel instead of crossing the river with shallower and more risky consolidation conduit.

Alignment F was identified as the preferred alignment and recommended to advance to final design. This alignment provides the maximum reduction in consolidation conduit length which reduces the associated cost, business impacts and construction risk. Figure 2 shows the configuration of selected Alignment (Alternative F).

GEOTECHNICAL SETTINGS

The site area lies in the Central Lowlands physiographic province that extends in a north-south direction in the middle of the state. The central lowland area consists mainly of the sedimentary rocks and the associated igneous basalts of Triassic and Jurassic age. The Hartford Basin of Connecticut and southern Massachusetts is a half graben in structure, 90 miles long, and filled with approximately 13,000 ft of sedimentary rocks, and basaltic lavas and intrusions (Hubert et al., 1978). The source area for the sedimentary rocks was mainly the metamorphic rocks of the Eastern Highlands. Volcanic flows separated the deposition of the lacustrine and fluvial deposits, which were derived from the erosion of the highlands to the east. Displacements along the faults continued throughout the depositional period. The depositional sequence resulted in a series of

Figure 2. Selected alignment (Alternative F)

features including the alluvial fan, lake, alluvial mudflats and floodplain deposits separated by basaltic flows.

Following the deposition of most of the sediments, the tectonic activity continued along the east edge of the basin. Displacements along the eastern border fault rotated the basin downward to the east that resulted in the easterly dipping beds. The Jurassic extensional tectonics is associated with the separation of the continents. That was the last major tectonic episode affecting the geology of the region. Age dating of the Triassic/Jurassic faulting in southern Connecticut has indicated that the last activity along the faults is approximately 175 million years ago (NNEC, 1975). All faults in the project area are therefore considered to be inactive.

The region has undergone a period of glaciations that has reshaped the terrain. Glaciers ground down the area's peaks, scraping away any weak or weathered rock and laying down a heterogeneous layer of ground-up rock. This till layer is present over much of the lower lying bedrock surfaces. The sediments of Glacial Lake Hitchcock filled in the deeply-incised Connecticut River Valley. The lake deposits are present in varying forms from Rocky Hill, Connecticut to Northern Vermont. Glaciers shaped the topography and left the area with much of the topographic relief present today. More recent alluvial deposits are common along the Connecticut and Park Rivers and their tributaries.

In the site area, the following soils are present overlying the bedrock, in general order of sequence from ground surface downwards: Artificial Fill, Alluvium, Beach Deposits of Lake Hitchcock, Glaciolacustrine Deposits, Glaciofluvial Deposits, and Glacial Till. Bedrock is not widely exposed in the project area. The formations that are in the general vicinity of the project and potentially could be encountered along the proposed tunnel are the Portland Arkose, the Hampden Basalt, and the East Berlin Formation. These units consist of shale and basalt with fractured and fault zones (Figure 3).

The initial geotechnical investigation program consisted of nine deep rock borings, six shallow borings, and twelve geophysical survey lines. The program included geophysical logging (acoustic televiewer) in all deep boreholes, water pressure (packer) testing in all deep borings, six in-situ stress determinations in two deep boreholes, falling head tests in the soil profile in selected borings, observation wells in four borings,

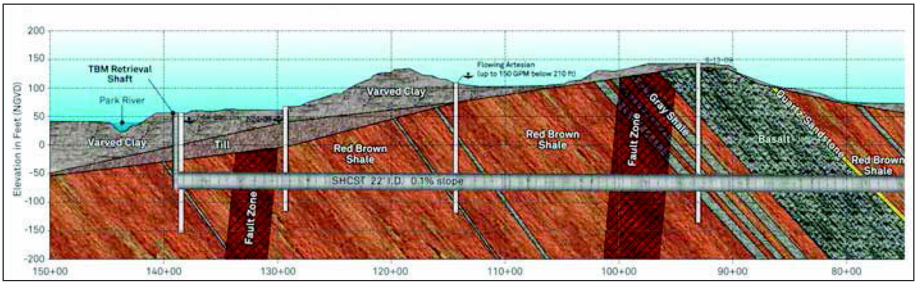

Figure 3. Geological profile

vibrating wire piezometers in five borings, groundwater level monitoring, and laboratory soil and rock testing.

MAIN TUNNEL

The deep rock tunnel would be approximately 21,800 feet in length and have a finished internal diameter of 22 feet. The tunnel will be excavated by a Tunnel Boring Machine (TBM) which is suitable for tunneling in hard rock conditions. The tunnel profile is entirely within bedrock at a depth low enough to accommodate the North tunnel system (currently under evaluation by the MDC and part of a separate study). There are several different types of rock TBMs which are manufactured to operate in specific types of ground conditions. These include main beam, single shield, double shield, and convertible (hybrid) hard rock/earth pressure balance (EPB) machines. The selection of the appropriate type of the TBM is an important decision which will impact the type of final lining, construction safety, quality, cost and schedule. The final recommendation on the type of rock TBM will be based on several factors among which rock and groundwater conditions along the tunnel alignment represent very important considerations. This selection will be based directly on the borehole data obtained from the final design geotechnical investigation program.

It is anticipated that the rock mass along the tunnel alignment will primarily consist of competent shale, sandstone, and basalt bedding dipping 10 to 20 degrees with occasional known fault zones. It may also contain diabase dikes which, if encountered, may contain fractured rock and flowing water.

The size of the construction shafts will depend on the TBM diameter, TBM type, and the dimensions of the permanent structures that will be housed in each shaft. For a 25-ft diameter TBM (required to excavate the 22-ft ID tunnel), the minimum clear shaft diameters that are required to allow launching and retrieval of the TBM are 35 feet and 30 feet, respectively. Larger diameters may be required to accommodate the permanent structures or to suit the contractor's means and methods.

Key considerations in selecting the appropriate construction methods include preventing groundwater drawdown and providing support of excavation. The shafts will be excavated using two methods for ground support. Slurry wall panels, laid out to approximate a circular shape, will extend from top of grade through overburden and will anchor into top of competent rock. The slurry walls will act as temporary support walls during construction and as the permanent final liner.

Through the rock, the shaft will be excavated using drill-and-blast method and the rock face will be supported using rock dowels and sprayed shotcrete.

Starter and tail tunnels will be required to assemble the TBM and to store equipment and muck cars. The starter and tail tunnels will be excavated by drill-and-blast method with a horseshoe cross-section.

One and two-pass lining systems are both considered viable options for the SHCST. The final recommendation of the tunnel lining system will depend on ground and groundwater conditions along the tunnel alignment and the construction cost of each option. Both options should be carried forward through final design phase.

The anticipated ground conditions along the tunnel alignment necessitate the use of final lining for the tunnel to meet the design criteria and ensure long term stability, durability, and hydraulic performance. Viable lining options for SHCST are cast-in-place concrete (CIP) and precast concrete segments.

Important considerations in selecting the type of tunnel lining include the following:

- Durability and ability to withstand the service environment without significant degradation during the tunnel design life
- Constructability
- Life-cycle cost.

A quantitative approach, adopted by EPA and ASCE, will be used to assess the corrosion of the final lining. This approach estimates the loss of material as a function of time, concrete properties and CSO characteristics.

The recommendations for advancing the tunnel design are summarized below:

- Define geotechnical parameters for tunnel analysis and design.
- Perform groundwater infiltration and ground settlement analysis to quantify the risk of consolidation settlement due to dewatering.
- Analyze geotechnical data to support the selection of the tunnel lining system and type of TBM. Based on the available geotechnical information and construction cost estimate, both tunnel lining options, namely cast-in-place concrete and precast concrete segmental rings, should be carried forward during the final design.

Site plans were prepared to identify existing site conditions, areas for site access, staging and operations, work zone layouts and constraints, equipment and materials storage, utility protection and relocations, site drainage and grading, erosion and sedimentation controls, and electrical power requirements. A temporary site plan and a permanent site plan were developed at the tunnel launch site and pump station. The temporary site plan designates specific areas during construction for the tunnel boring machine, the tunnel crane pad, the tunnel mucking operations, short and long term storage areas for tunnel segments, the pump station crane pad, contractor offices, workshops, storage areas and parking areas. The permanent site plan identifies the locations of the tunnel pump station, screening/degritting building, HVAC and electric buildings, and odor control facilities.

A conceptual planning level cost estimate, schedule and contract packaging was performed. Costs from similar historical projects were obtained and utilized to develop unit costs and extrapolated for the SHCST project. A detailed cost estimate was performed to estimate the construction cost of the main deep rock tunnel, TBM launch shaft, and TBM retrieval shaft associated with the selected Alignment F. Two construction options were considered in the detailed cost estimate, namely tunnel excavation by a double shield TBM along with installation of precast concrete segmental rings and tunnel excavation by a main beam TBM followed by installation of initial rock support and cast-in-place concrete final lining. The cost estimate for the entire SHCST Project is approximately $500 MM. The project construction duration is estimated at approximately 72-month (6-year).

The recommended contract packaging is to release six construction contracts: (1) Preliminary Utility Relocation, (2) Tunnel, (3) Pump Station, (4) Franklin/ Maple Consolidation Conduits, (5) Flatbush/Arlington/Newington/ New Britain Consolidation Conduits, and (6) West Hartford Consolidation Conduit. The contracts were grouped to align construction skill sets but allow for the phased release of the bid packages. The overall construction schedule is to be coordinated such that the tunnel, pump station and consolidation conduit contracts are constructed independently but conclude coincidently.

MDC management has stated that a goal for the project is that odor complaints must not occur. Therefore, the odor control strategy for the SHSCT system is focused on minimizing odors from the two main shafts at the tunnel ends and at the six intermediate drop shaft sites. Ventilation rates of approximately 80,000 to 85,000 CFM have been estimated for both the upstream and downstream shaft. Ventilation rates ranging from 2,300 to 7,500 CFM have been estimated for the intermediate drop shafts.

Active fan driven odor control systems are recommended at the tunnel ends and passive systems are proposed for the six intermediate drop shafts. Activated carbon is recommended as the odor control treatment process. The odor control systems can either be located in buildings above grade and possibly even below grade in vaults, particularly for smaller systems. This is to address visual impacts in neighborhoods from these industrial type treatment systems. Early estimates of foot print size indicate the larger odor control facilities at the tunnel ends can be roughly 2,000 square feet in area and the smaller systems at the intermediate drop shafts can be roughly 300 square feet in overall size.

DROP SHAFTS

Seven hydraulic drop shafts are used to convey flow in a controlled manner from the shallower consolidation conduits to the deep rock tunnel. Utilizing a two level screening process, the selection process assessed each site's characteristics and recommended either a baffle-plunge or tangential vortex based upon cost effectiveness, hydraulic performance, and operation and maintenance considerations (Figure 4).

The tangential vortex drop structure type was selected for all of the sites along the tunnel alignment (with the exception of the retrieval site) due to its widely accepted use for deep rock CSO storage tunnels, history of acceptable performance, and cost effectiveness when compared to the baffle-plunge drop structure. The baffle-plunge drop structure type was selected for the deep rock tunnel retrieval site because of the existence of the larger diameter shaft being constructed at this site for the TBM retrieval. Once such a large shaft is present, the baffle-plunge becomes ideally suited for such applications because of its compact surface area impact. Based on the drop shaft selections, potential operations criteria and maintenance requirements were developed for each of the proposed drop structure sites.

CSO/SSO CONSOLIDATION CONDUITS

New diversion structures constructed near existing CSO/SSO locations will utilize transverse or side flow weirs to direct the design overflows from existing pipes into the consolidation conduits. These conduits then convey flows to the deep rock tunnel through either vortex or baffle drop shafts.

The consolidation conduits will be installed using a combination of microtunneling, guided boring, shallow rock tunneling, and open cut construction techniques. It is anticipated that three consolidation pipes along Alternative F will be installed using microtunneling methods. This includes a 24-inch guided bore of the Newington Consolidation Pipe (NCP), a 42-inch microtunnel installation of the New Britain Consolidation Pipe (NBCP), and a 48-inch microtunnel installation of the Flatbush Consolidation Pipe (FCP). When considering microtunneling as the likely means of installation, effort has been made to locate conduits within soil however there is the potential for mixed-face microtunneling in areas of till.

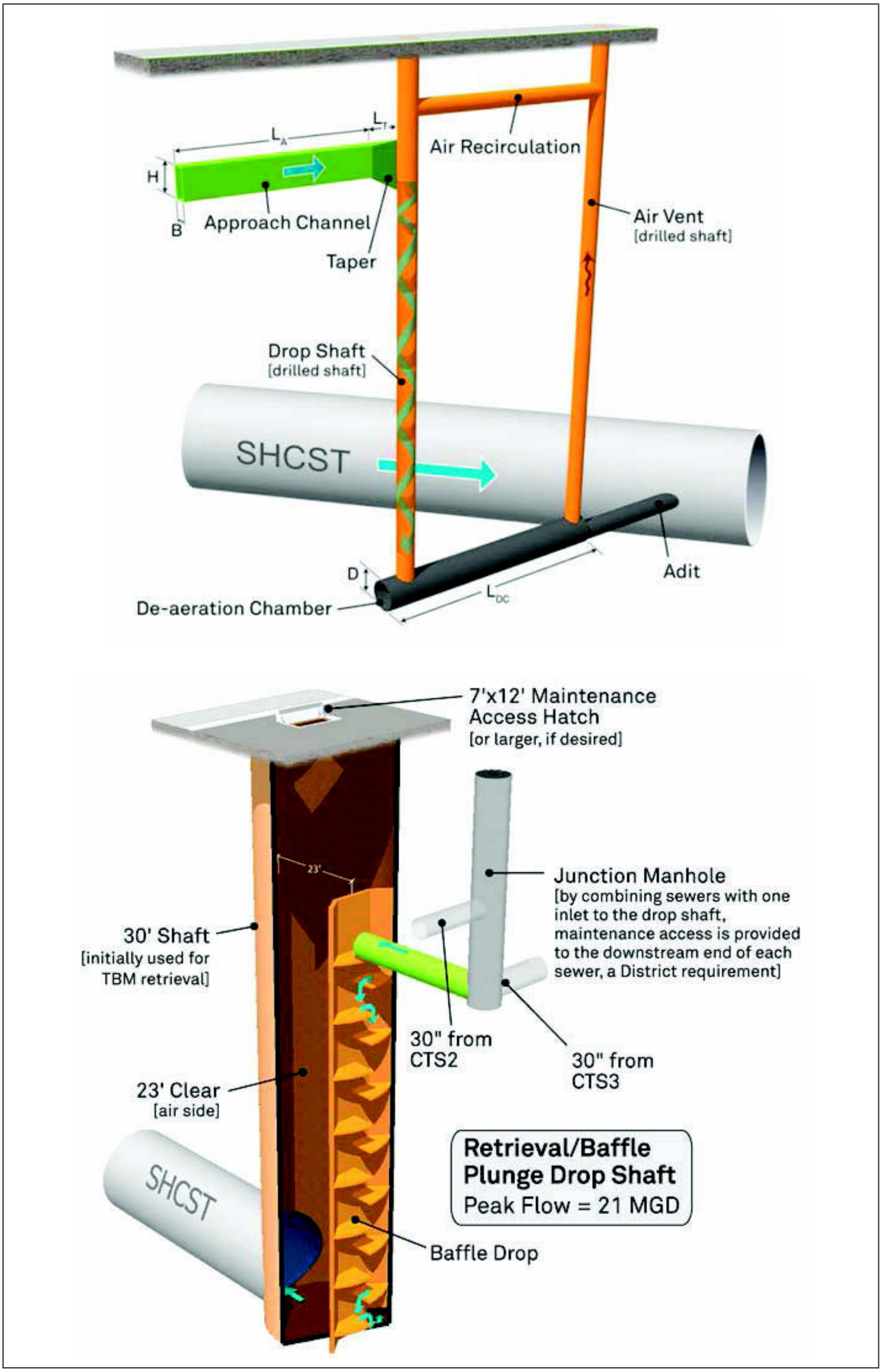

Figure 4. Vortex and baffle drop shaft alternatives

Table 2. Consolidation conduit summary

Consolidation Conduit Reach	Contributing Overflows	Peak Flow (MGD)	Length (ft)	Pipe Diameter (in)
Franklin Avenue consolidation pipe (FACP)	F102-F105	74	2,350	66
Maple Avenue consolidation pipe (MCP)	F100 + F101	65	700	60
Arlington consolidation pipe (ACP)	S19, S21	15	1,350	27
New Britain Avenue consolidation pipe (NBCP)	S23-S30	29	1,100	42
Flatbush Avenue consolidation pipe (FCP)	S14-S16	26	2,700	48
West Hartford consolidation pipe (WHCP)	CTS2 + CTS3	11	1,100	30
Newington consolidation pipe (NCP)	NTS	4.2	1,825	24

The open cut method of pipe installation will be utilized for installation of the 30-inch West Hartford Consolidation Pipe (WHCP), the southern section of the 24-inch NCP, and the 27-inch Arlington Consolidation Pipe (ACP). The open cut method creates more temporary disturbance to traffic, business and residences as this work is performed primarily within the roadways, however may be the preferred installation method due to the depth of the pipe, geotechnical conditions, and cost considerations. Open cut installations typically will be shallower than microtunneling installations.

Based on existing geotechnical information, it is anticipated that the 66-inch Franklin Avenue Consolidation Pipe (FACP) and the 60-inch Maple Avenue Consolidation Pipe (MACP) will be constructed using an open face rock tunneling machine. Consideration will be given in final design to standardizing the diameters of these tunnel consolidation sewers to potentially reduce costs. Table 2 provides a summary of the consolidation conduits.

PUMP STATION

The design of the tunnel pump station (TPS) is being led by AECOM's major subconsultant Black & Veatch. The TPS is designed to pump out the SHCST when capacity at the HWPCF has dropped to an acceptable level. At this point, stored flows will receive adequate treatment prior to discharge to the Connecticut River. The proposed TPS will be located within the HWPCF complex.

The TPS will be designed to pump out at a maximum 27 MGD capacity. This rate will allow the 62 MG SHCST to be pumped out within 55 hours (2.3 days). The proposed tunnel invert elevation at the TPS site is –170 feet and the discharge elevation at the plant is +6. Therefore, the total maximum static head is 176 feet.

The recommended pump equipment consists of four 9 MGD vertical non-clog centrifugal pumps. This will provide a firm pumping capacity of 27 MGD with one unit out of service. Variable frequency drives (VFDs) are recommended for the pumps as turndown capability to approximately 4 to 5 MGD can be achieved.

The TPS will discharge directly to the new Headworks facility currently under design at the HWPCF. The force main is currently sized to be 36-inches in diameter. The recommended connection point at the discharge end is at a new junction structure just upstream of the new influent pumping station. A surge tank will be provided on the

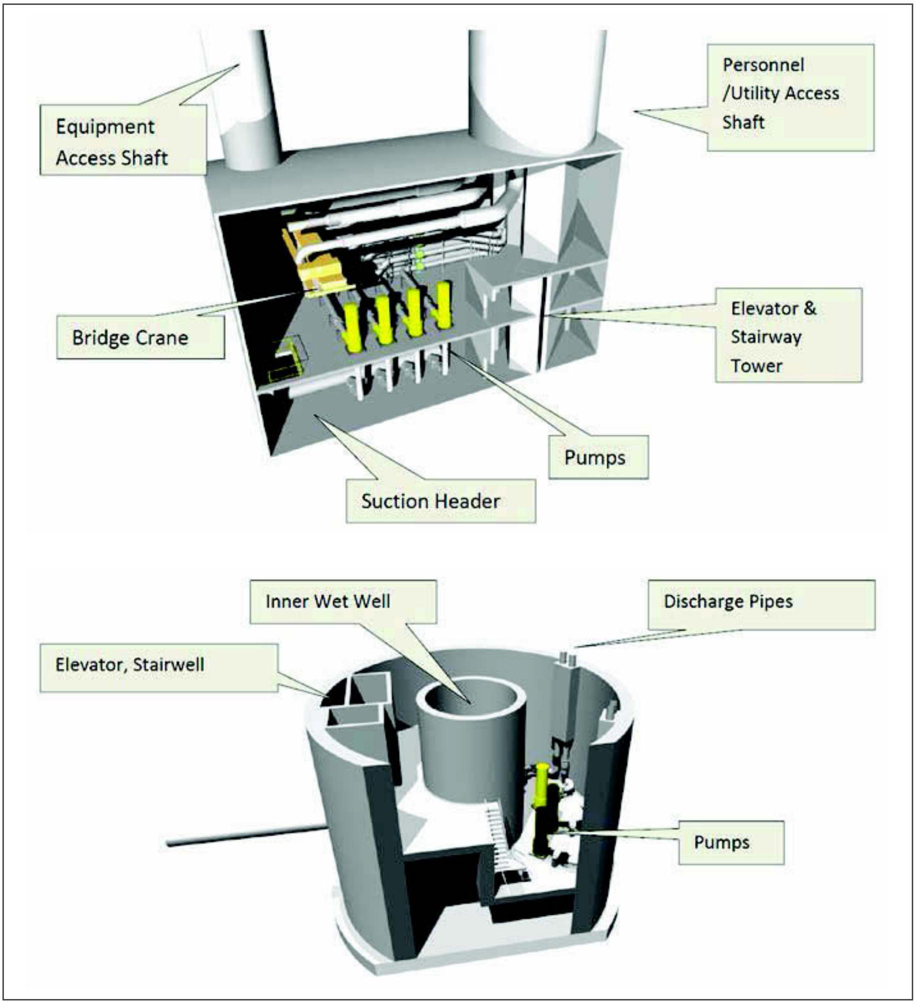

Figure 5. Cavern and shaft pump station alternatives (Source: Black & Veatch)

discharge force main to minimize surges in the system. The surge tank will be situated at the TPS site.

Two pump station configurations are presented as the finalist options. One of these is a cavern pump station and the other is a circular pump station with a suction header pipe system (Figure 5). The two configurations are comparable in overall cost and the cavern pump station has some advantages in terms of non-cost criteria, mainly centered on maintenance attention associated with crane lifts. It is recommended that MDC personnel visit both type of facilities at other deep installations; this will provide additional information from users, allowing for a more informed decision on pump station type.

Once a decision on pump station type has been made by MDC, meetings with Building code and Fire control officials will be held to ascertain whether the layout will be acceptable as is or whether additional support systems such as stairways, etc., would be required for compliance with their interpretations of codes and practices.

A new 9,800 kW overhead electrical power service from CL&P will be required for the tunnel boring machine (TBM). This power feed will be converted to a permanent power feed for the TPS, once the TPS is completed and made operational. Current power requirements for the TPS and related facilities are on the order of 3,055 kW.

Screenings and grit capture will be accomplished in a separate 35-foot diameter dedicated shaft. The shaft which will be used as the launch shaft for the TBM tunnel will be converted to the grit/screenings shaft. Bar screens will be provided to protect the TPS pumps from solids and debris which would either clog or damage the pumps. A rake lowered by crane will either push or pull the screenings up from the shaft. Grit and other heavier debris will be removed from the pit by a clamshell bucket. The screenings shaft will be used for tunnel construction, allowing construction of the TPS to proceed in parallel with tunnel construction.

The TPS and the Grit/Screenings facility will be roughly 150 feet apart and will be connected with a 48-inch diameter suction header. An at-grade building will be provided over the below grade pump station to house support facilities critical to the operation of the pump station and to allow for pump station access and egress. Personnel access/egress will be by elevator. A separate stair tower will be provided for emergency situations. The grit/screenings facility will also be enclosed in a building to better contain odors and to promote a more visually appealing facility to neighboring businesses.

CONCLUSIONS

This paper presents the design of a deep rock conveyance and storage tunnel, drop shafts, consolidation conduits, and a pump station in Hartford, CT. The geological settings and subsurface investigation program is discussed and the general aspects of the preferred alignment selection is described. Relevant alternatives for the drop shafts and the pump station are explained and recommended options are presented.

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CONSTRUCTION CHALLENGES FOR A CITY OF AUSTIN DEEP INTERCEPTOR SEWER TUNNEL

Eric Dawson ■ Parsons

Jon Kaneshiro ■ Parsons

ABSTRACT

Construction of the Austin Downtown 1Tunnel took place from 2010 to 2012 and consisted of 20,600 ft of 8–10 ft diameter bored tunnel (lined with 36–90 inch polymer pipe), six new shafts from 70–90 ft deep by 12–25 ft excavated diameter using a variety of construction techniques (large diameter polymer concrete pipe lined), tie-in to the Toomey Lift Station shaft, tie-in to the Govalle tunnel, and tie-in diversions to the South and North Austin Interceptor sewers. This paper describes the construction challenges, including: low cover, grouting of groundwater inflows, and three crossings of Lady Bird Lake.

INTRODUCTION

The City of Austin, specifically, the downtown urban core of the City, underwent significant growth during the past 10 years, and significant growth continues, resulting in increased wastewater flows. Much of this development involves converting low-density urban use types into higher density types, resulting in increased wastewater flows. The existing wastewater infrastructure that serves the downtown area is rapidly reaching its capacity. The City of Austin retained Parsons to provide design and construction phase engineering services for a deep tunnel interceptor sewer to provide capacity for the increased wastewater flows. Construction of the Austin Downtown Tunnel took place from 2010 to 2012 and consisted of 20,600 ft of 8–10 ft diameter bored tunnel (lined with 36–90 inch polymer pipe), six new shafts from 70–90 ft deep by 12–25 ft excavated diameter using a variety of construction techniques (large diameter polymer concrete pipe lined), deepening and tie-in to the Barton Creek Lift Station Relief Tunnel (BCLSRT) at the Toomey Lift Station shaft, tie-in to the Govalle tunnel, and tie-in diversions to the South and North Austin Interceptor sewers at the Lamar and Riverside shafts. Figure 1 shows the tunnel alignment along with the geological profile.

The tunnel contains a gravity-flow wastewater interceptor constrained on the downstream end by the elevation of the existing Govalle Tunnel at the tie-in point, and on the upstream end by the elevation of the existing Toomey Lift Station at the terminus of the Barton Creek Lift Station Relief Tunnel (BCLSRT). The tunnel extends through softer limestone (Austin Chalk), harder limestone (Buda and Georgetown), shale/clayshale (Eagle Ford), and clay/clayshale (Del Rio), which underlies unsaturated and saturated alluvial soil (Colorado River Terrace Deposits). The entire area is part of the Balcones fault zone, and the tunnel crosses a number of fault contacts between formations or between members within formations.

As can be seen in Figure 1, the tunnel roughly parallels Lady Bird Lake, crossing it three times. As a result, saturated alluvium that has hydraulic connection to a nearly limitless supply of water in Lady Bird Lake exists over the entire tunnel. In light of the overlying saturated alluvium, low rock cover over the tunnel and possible groundwater inflows were identified as important risks for construction of the tunnel.

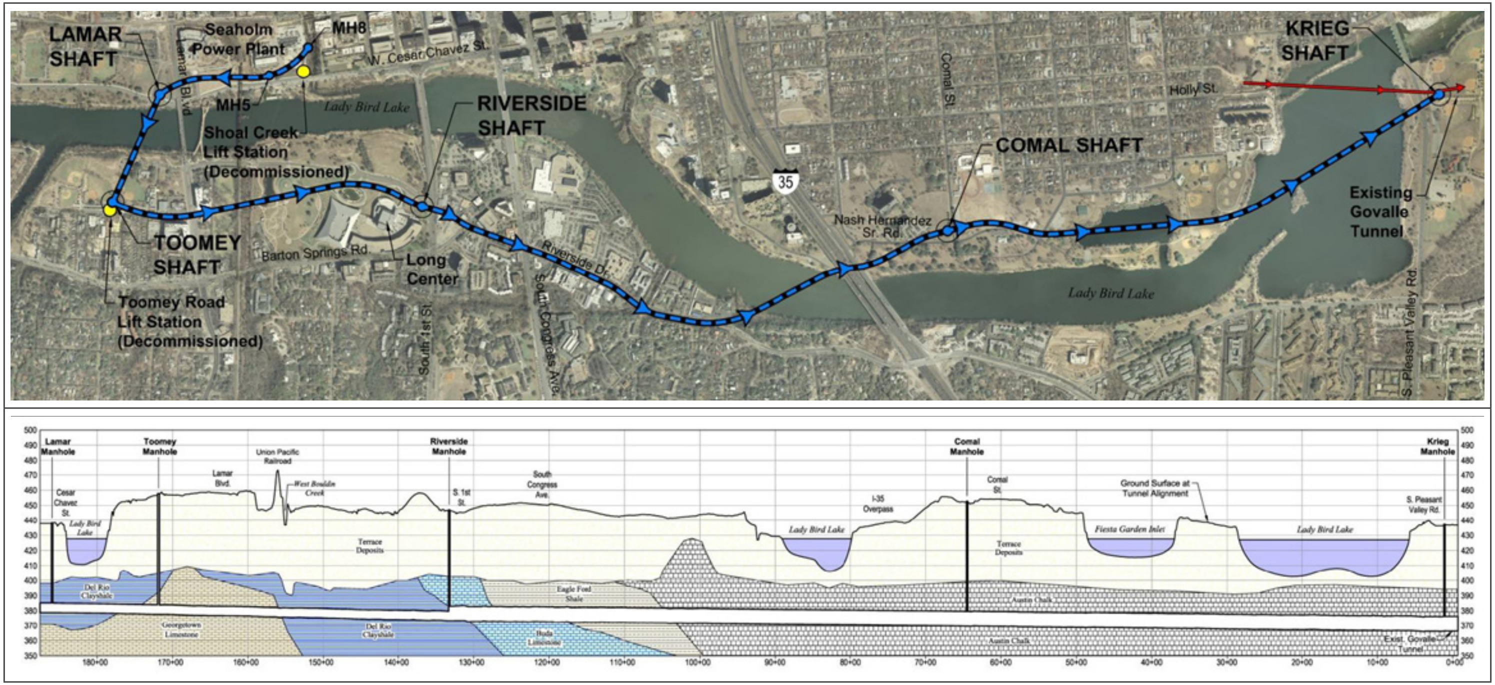

Figure 1. Generalized plan and geologic profile

DESIGN PHASE MITIGATION OF LOW COVER RISK

Efforts to mitigate the risks from low rock cover and potential groundwater inflows during the design phase focused on several areas, including hydraulic design of the interceptor sewer to keep it as flat and deep as possible, developing contract document minimum requirements for shaft construction to prevent groundwater inflows and tunnel boring machine (TBM) capabilities, and additional investigations.

Hydraulic Design

It was recognized early in the planning that maximizing rock cover between the tunnel and the overlying saturated alluvium would be an important consideration, and considerable efforts were made to keep the tunnel and interceptor sewer as deep as practical. This was limited by elevations of the tie-ins to the Govalle Tunnel on the downstream and BCLSRT terminus (Toomey Lift Station shaft) near the upstream end of the tunnel, since the sewer interceptor is a gravity-flow pipe.

The slope of the pipe was set as flat as possible (0.05%) and still provide velocities sufficient to prevent accumulation of deposited solids on the bottom of the pipe. Future projected wastewater flows in the interceptor pipe are much greater than current flows. The smaller current year flows were used to select the minimum allowable slope that would still provide regular “cleansing” velocities. The larger projected future peak flows were used to determine the diameter and ultimate carrying capacity of the interceptor sewer. The pipes are greatly oversized for current year flows, because of the large difference in average current year flows and estimated future peak flows. Additionally, the projected flows in the pipe were analyzed to justify matching the flow lines of the pipes where diameter changes occurred instead of matching pipe crowns. This allowed the pipe to be several feet deeper in the most upstream part of the tunnel.

The far upstream end of the Govalle Tunnel was constructed with a steeper slope than the rest of the tunnel, which was constructed at a minimum slope of 0.05%. It was discovered that if the tie-in point to the Govalle Tunnel was moved roughly 3,000 feet downstream, the tunnel could be lowered by approximately 9 feet. This added length to the tunnel, but the added depth made it possible for it to be constructed in rock instead of saturated alluvium. This change made a big difference in the feasibility of the project.

Construction Shaft Requirements

The construction shafts would extend down through saturated alluvium and into rock. A previous Austin tunnel project had difficulties sealing the shaft excavation support with rock, which resulted in large water inflows into the shaft. To ensure groundwater would be positively controlled at the shafts and to avoid potential conflicts with the contractor about water inflows, minimum requirements for the shaft excavation supports were incorporated into the contract documents. The construction shafts were required to use a method, such as secant piles or slurry walls, that would “socket” into the rock and positively cut off groundwater. The contractor elected to use secant piles for the four new shafts in original construction scope, and did not have difficulties with groundwater flows into any of these construction shafts. There were two additional shafts added to the projects during construction, and they were constructed by augered casing method using corrugated metal pipe.

Geotechnical Investigation

The initial geotechnical boring program did not identify any areas that lacked bedrock cover over the tunnel, but the contact of the formations with the overlying saturated Terrace Deposits along the tunnel is erosional and varies in elevation. Since the borings only reflect conditions at discrete points, it was possible that local areas of low

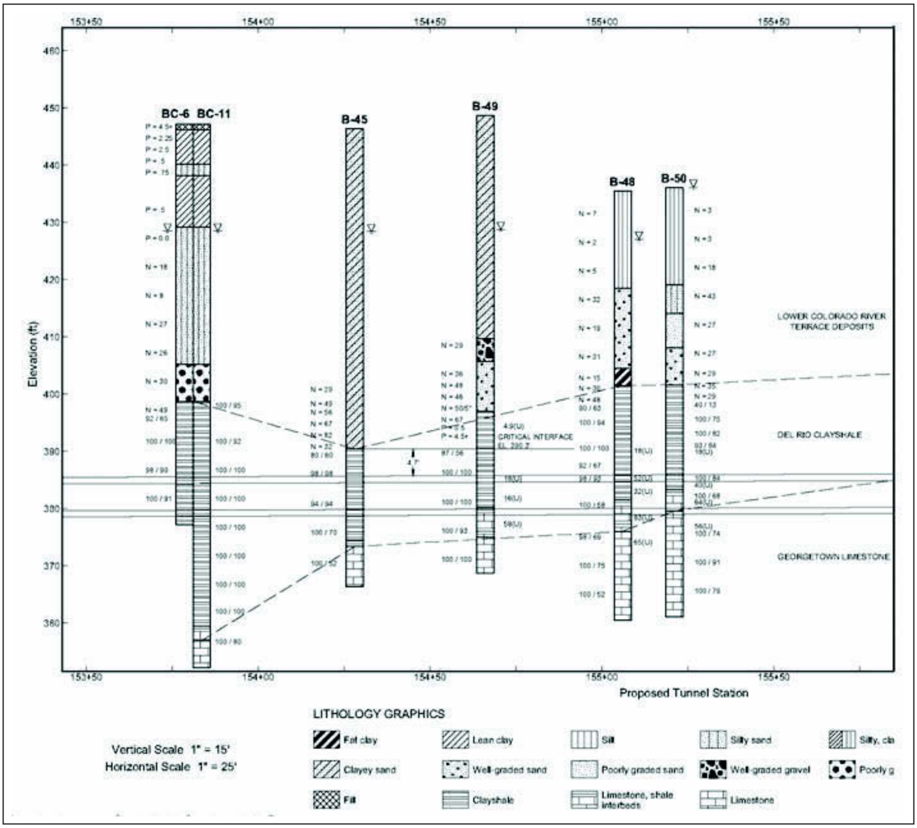

Figure 2. Profile of low cover area

or no cover existed. As a result, it was decided to perform a geophysical investigation along the alignment to better define the depth to bedrock. The land portion of the alignment was investigated using seismic methods, and the water portion of the alignment was investigated using resistivity methods.

Geophysical investigation results highlighted three areas where depth to the alluvium/bedrock interface was deeper than what was indicated in previous borings. Two of those areas were investigated with additional borings. The third area, at an inaccessible portion of the shoreline of Lady Bird Lake, was not accessible to a land- or water-based drilling rig, so a confirmatory boring was not possible.

Three confirmatory borings were drilled in the two other areas of concern identified in the geophysical investigation. The boring in one area yielded results almost identical to the original boring completed in that area, and did not confirm the finding from the geophysical study.

In the second area of concern, two confirmatory borings were drilled. One confirmatory boring was consistent with an adjacent previous boring, but the second confirmatory boring found the alluvium/rock interface to be about 8 feet lower than had been previously identified. This reduced the rock cover to about 5 feet, which is significantly less than one tunnel diameter, in expected fractured clayshale material. Figure 2 shows a profile of the tunnel and alluvium/rock interface for the low cover area.

This area was in the last reach before the tunnel intercepted the BCLSRT terminus at a lift station shaft. Since the downstream reaches were set as flat as possible, the reach leading to the BCLSRT terminus was steeper to arrive at the bottom of the existing structure. Deepening the lift station shaft structure at the BCLSRT terminus was considered to lower the tunnel and increase rock cover, but it was decided the benefits of deepening the structure did not outweigh the cost, and mitigation of the low cover and groundwater inflow risk would be addressed by minimum specification requirements for the TBM.

CONSTRUCTION PHASE MITIGATION OF LOW COVER RISK

The contractor for the project was SAK/Quest, a joint venture of SAK Construction and Quest Civil Constructors. Early in the project they approached Ayman Benyamin, Austin Water, to discuss possible additional mitigation measures for the known low cover area, which is located near a creek, so there was a chance for historical erosion channels there. There is also a railroad track that crosses adjacent to the low cover area, which could magnify the consequences of potential ground settlement caused by a large groundwater inflow to the tunnel that could not be controlled.

Several options for mitigation low cover risk were discussed, including lowering the tunnel by lowering the upstream existing Toomey Lift Station shaft. As the discussions proceeded, the City and contractor worked together to drill additional borings in the area of concern to better delineate the extent of the low cover area, and to better define the depth to rock in the vicinity of the creek and railroad. The additional boring nearest the previous boring with the least rock cover showed increasing cover moving away from the known point of minimum cover. The other two additional borings were located at the creek and adjacent to the railroad, and showed rock cover consistent with what had previously been found.

In summary, the additional borings provided relatively good news. First, the area with the least rock cover (less than one tunnel diameter) appeared to have a limited extent, between two borings located in open parkland approximately 100 feet apart. Second, the low cover area did not appear to extend to the creek and railroad track, where consequences of potential ground settlement would have been most severe. Figure 2 shows the profile of the tunnel including the rock/alluvium interface at the low cover location.

At this point in the project, the City was contemplating extending the tunnel by about 2.000 feet. This extension would take the place of a planned microtunnel project being implemented to re-route existing sewers to the tunnel interceptor. The microtunnel project was going to be held up due to a remediation project, which would in turn hold up redevelopment projects in the area. The extension of the deep tunnel would pass well below the remediation area, allowing the other projects to keep their timelines.

Extension to the tunnel would be on the other (north) side of Lady Bird Lake, and would cross an area geologically similar to the low cover area, and because of the pipe slope, the elevation of the tunnel would be higher for the extension. It was recognized that for the extension to be feasible, the Toomey Lift Station shaft would need to be lowered, allowing the tunnel depth to be increased for the extension.

The contractor developed a proposal for deepening the Toomey Lift Station shaft by approximately 6 feet, the maximum amount possible that would maintain the minimum required slope. Deepening the Toomey Lift Station shaft by 6 feet would increase the cover at the low cover area by a little more than 2 feet. The tunnel upstream of the Toomey Lift Station shaft, including the extension, would also be lowered by 6 feet.

The City elected to move forward with the change to lower the Toomey Lift Station shaft and the tunnel to take advantage of low cover risk mitigation and to keep the tunnel extension as a viable option. The tunneling proceeded through the low cover area without incident and with only minor water seepage into the tunnel. The City was also able to add the extension to the project at a cost less than the bids they had received for microtunneling the work since additional mobilization/demobilization would not be required, and the deep tunnel approach did not require as many shafts.

The Toomey Lift Station shaft structure was built in a construction shaft from a previous tunnel project that had significant groundwater inflow problems during construction. It was constructed as a cast-in-place structure within the construction shaft. It was planned to remove the cast-in-place floor of the structure, to excavate below the floor to deepen the structure, and then cast-in-place a new concrete floor and wall extensions. There were concerns about groundwater saturating the backfill between the shaft supports and the exterior of the lift station structure that could infiltrate into the structure while it was deepened. Exploratory core holes verified the presence of large amounts of water. The contractor implemented a successful grouting program for the space outside the lift station structure to cut off the groundwater prior to deepening the structure.

WATER INFILTRATION NEAR A SHAFT ON THE TUNNEL EXTENSION

After nearly 20,000 feet of near incident-free tunneling, and construction or deepening of seven deep construction shafts, the TBM approached the shaft for Manhole 5 (MH5) on the tunnel extension. As the TBM reached the shaft for MH5, water and alluvial material rushed in through the face of the TBM. The flood doors on the TBM face were closed, and the TBM crew evacuated the tunnel. The initial inflow of water was estimated at 400 gpm. This flow rate decreased to about 200 gpm, where it stayed relatively steady. The contractor installed pumps in the downstream shaft and was able to keep up with the water, allowing work to continue in the downstream portions of the tunnel. Along with the water inflow, it was estimated that two or three cubic yards of sediment had been washed into the tunnel.

The construction shaft for MH5 was 80 feet deep. The shaft was constructed with a 10-foot-diameter corrugated steel pipe casing grouted within a 12-foot diameter augered excavation. The geology of the shaft consists of approximately 65 feet of alluvial soil, underlain by approximately 8 feet of Del Rio clayshale, and approximately 6 feet of Georgetown limestone, which extends below the bottom of the shaft. The TBM heading was in mixed face conditions when it arrived at the shaft, with the Del Rio clayshale exposed near the upper part of the tunnel. Figure 3 show a profile of the MH5 area.

Once the pumping system was installed and observed to be adequate for the water inflow, work began to assess and address the water inflow. Work crews re-entered the tunnel and removed the sediment that had been washed-in, and inspected the heading. They found the majority of the water flowing around the TBM shield and into the tunnel.

Several options were considered to enable the resumption of mining. One option was to simply resume mining into the shaft. This possibility was discounted because of the risk of having the shaft fill with water and large amounts of sediment when the TBM broke holed-through into the shaft, which would result in hazardous conditions for the TBM crew, and could result in significant ground settlement around the shaft.

Instead, efforts moved forward with a grouting program with urethane grout to cut off the water inflow to the tunnel. Grouting was done from inside the MH5 shaft. A series of holes were drilled at different depths, lengths, and locations around the circumference of the shaft. Additionally, several different urethane grouts were tried. After a little more than two weeks, the grouting program was successful in reducing the water inflow to a minimal amount, and it was decided to resume mining, which proceeded into the shaft and on to the end of the tunnel without further incident.

During grouting, it was thought that if the source of the water could be identified, the grouting program could be better targeted to achieve groundwater cutoff. Some potential sources for the water infiltration included a natural geologic feature that connected

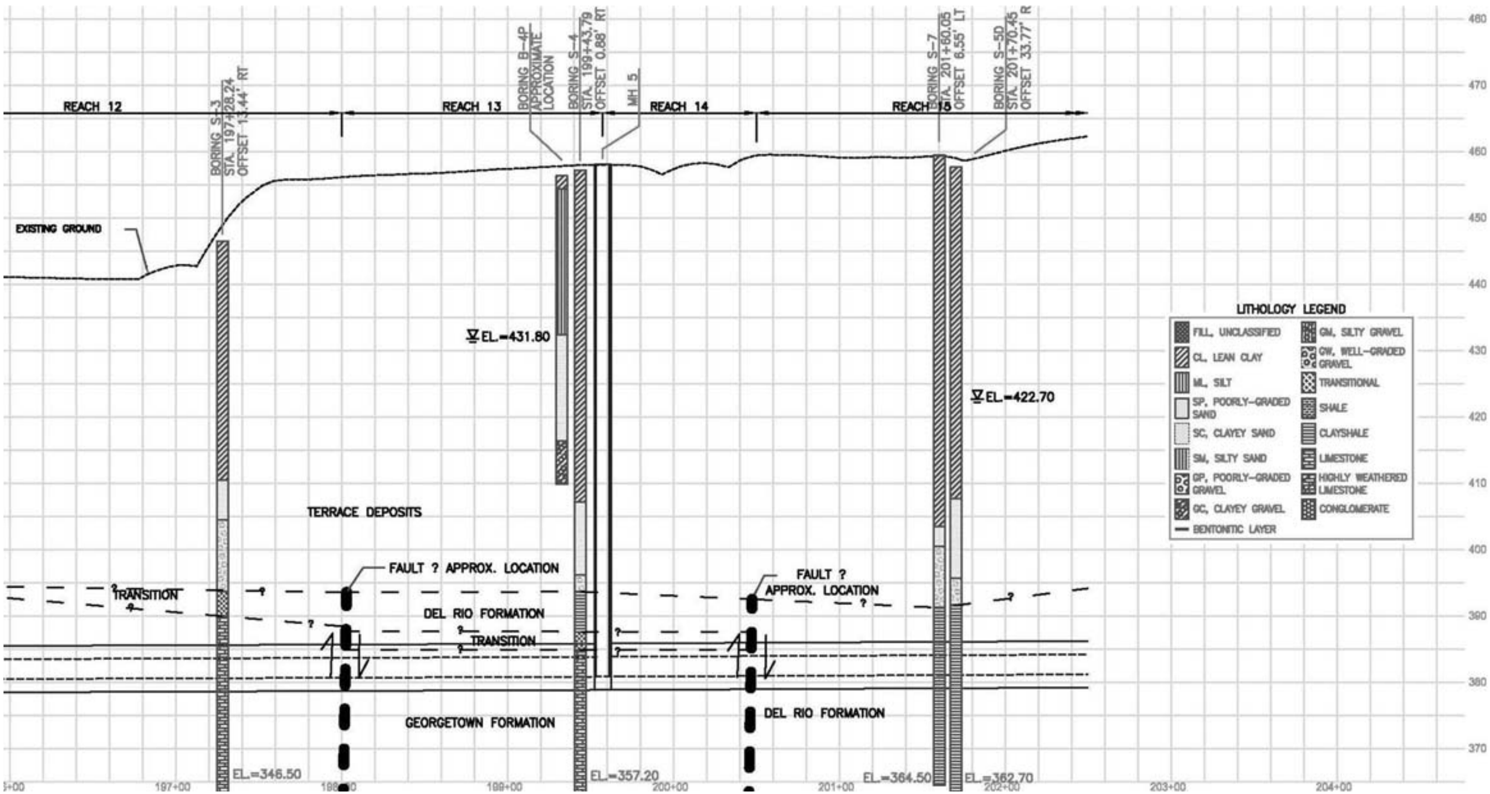

Figure 3. Profile of MH5

the saturated alluvium with the tunnel, a soil boring without a good seal, or a flaw in the grout for the shaft. During the urethane grouting, it was not possible to identify any of those potential causes as the clear cause. It also was not possible to definitely rule any of them out. In the end the exact cause could not be determined. It was deemed too risky to open the tunnel support to determine where the water was coming from. The contractor and the City agreed to share the cost associated with the water inflow.

ACKNOWLEDGMENT

The authors thank Ayman Benyamin for contributing to this paper.

RESULTS OF LOW COVER RISK MITIGATION

Efforts made to reduce uncertainty and to mitigate known risks during design, resulted in a successful project. Because the owner, engineer, and contractor worked as a team to mitigate risks during construction and fully cooperated to address the water inflow incident also contributed to the success of the project. From the owner's perspective, they had a competitive bid, and were able to add valuable scope to the project during construction, with minimal changes due to unforeseen conditions. From the contractor's perspective, their suggestions for value engineering and risk mitigation were honestly considered and were implemented when possible.

TARP MAINSTREAM TUNNEL SYSTEM CONNECTION TO MCCOOK RESERVOIR GOES INTO HIGH GEAR

Clay Haynes ▪ Black & Veatch

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Dave Schiemann ▪ USACE Chicago District

ABSTRACT

The Metropolitan Water Reclamation District of Greater Chicago's (MWRDGC) Tunnel and Reservoir Plan (TARP) and the McCook Reservoir will further reduce flood damages and combined sewer overflows (CSOs) for the city of Chicago and Cook County, Illinois. The McCook Reservoir will receive approximately 10 billion gallons of water via the Main Tunnel System which connects the TARP Mainstream Tunnel to the McCook Reservoir and the Distribution Tunnel System which connects the TARP Des Plaines Tunnel to the McCook Reservoir. The U.S. Army Corps of Engineers (USACE) tasked Black & Veatch to design the Main Tunnel System. This paper describes the Main Tunnel System project components; how this final piece fits into the Chicago's TARP program; and provides an update on the construction progress to date.

The Main Tunnel System design includes a 27.5-m (90-ft) diameter and 92-m (300-ft) deep Main Gate and tunnel Construction Access Shaft and associated wet-well shaft arrangements to house six high-head and large 4.4-m by 9-m (14.5-ft by 29.5-ft) wheel gates; a 10-m (33-ft) diameter and 490-m (1,600-ft) long Main Tunnel in rock including a tunnel bifurcation (with steel and concrete lining) at the Main Gate/Access Shaft; a live tap connection to the existing Mainstream Tunnel; and energy dissipation and portal structures.

The construction of the Main Tunnel System has been divided into the Main Gate/Access Shaft contract and the Main Tunnel System contract. Construction of the Main Gate/Access Shaft was completed in August 2011 and Main Tunnel System construction is currently ongoing. Construction progress updates and discussion of key construction issues to date are presented in this paper. Kiewit Infrastructure Co. was awarded the construction of the CUP McCook Main Tunnel System with a Notice to Proceed on January 1, 2012.

HISTORY

The MWRDGC has been dealing with CSOs and flooding problems since the late 1960s and formally adopted the Tunnel and Reservoir Plan (TARP) in 1972. Phase I of TARP, which included construction of 175 km (109 miles) of deep storage and conveyance tunnels with diameters up to 10-m (33 ft), was completed in 2006. Phase I resulted in substantial improvements in surface water quality enhancing the Chicago riverfront. Additional improvements are expected as Phase II comes on-line, including three large reservoir systems, as shown in Figure 1.

Phase II includes a series of storage reservoirs to increase flood storage capacity and further reduce CSO discharges, with additional storage capacity projected to come on-line over the next several years.

MCCOOK RESERVOIR OVERVIEW

Authorized in the Water Resources Development Act of 1999, the McCook Reservoir Project is a key component of Chicago's ongoing TARP Project. The McCook Reservoir will provide approximately 10 billion gallons of additional CSO and flood water storage for TARP. The reservoir will store excess CSO and floodwater from TARP's Mainstream and Des Plaines deep tunnel systems during periods of wet-weather peak flows. This stored volume will be pumped to the MWRDGC's Stickney Wastewater Treatment Plant (WWTP) for treatment prior to discharge to Des Plaines River.

The McCook Reservoir is currently under construction and being excavated in dolomite limestone. An aerial image of the reservoir is shown in Figure 2. The reservoir walls are nearly vertical (Figure 3) and excavated to depths up to 107-m (350-feet) below grade. The McCook Reservoir development as part of the Phase II of TARP includes tunnels for TARP connections servicing the Des Plaines and Mainstream systems. The subject of this paper is the McCook Main Tunnel System (MTS) connecting the TARP Mainstream tunnel to the McCook Reservoir.

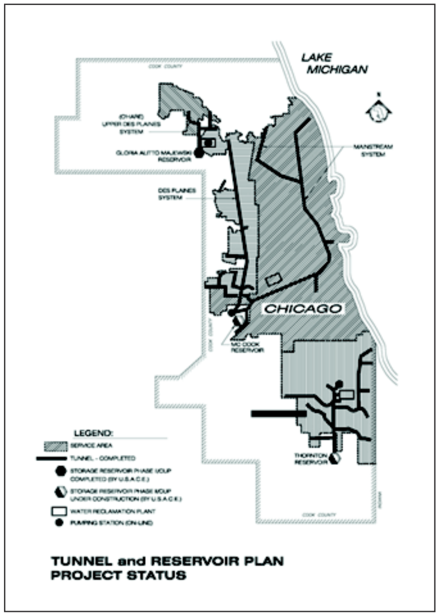

Figure 1. Schematic layout of Chicago's TARP (Source: MWRDGC)

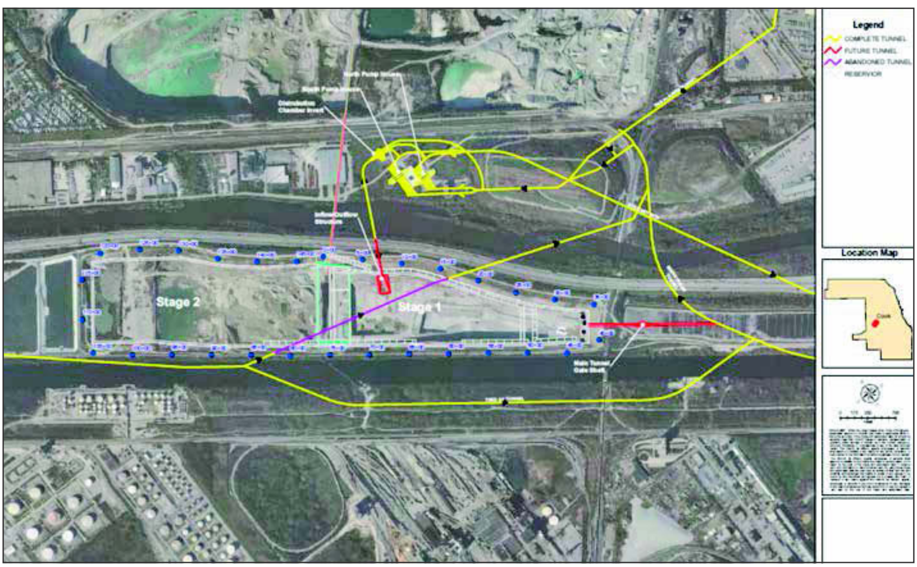

Figure 2. Aerial view of McCook Reservoir and quarry

Figure 3. McCook Reservoir and limestone quarry

OVERVIEW OF THE MTS PROJECT

The MTS tunnel is the primary inlet for CSOs and floodwater from the TARP tunnels into the McCook Reservoir. The tunnel will be concrete lined for long-term stability and to minimize infiltration and exfiltration. The MTS extends from the existing Mainstream Tunnel through the gate shaft and connects to the McCook Reservoir. The MTS is approximately 490-m (1,600-ft) long with a finished inside diameter of 10-m (33 ft). The MTS will be bifurcated through the gate shaft. Kiewit is using drill-and-blast construction methods on this relatively short tunnel. A rock plug will be left in place in the MTS until installation of gates and related structures have been completed and the reservoir is ready to receive flows. The MTS is characterized by the following components:

- Main Tunnel—an approximately 490-m (1,600-ft) long, 10-m (33-ft) inside diameter (ID) drill-blast tunnel connecting the existing Mainstream Tunnel and the McCook Reservoir, bifurcated for approximately 88-m (290-ft) through the Main Gate/Access Shaft.
- Main Gate/Access Shaft—a 27-m (88-ft) ID, 90-m (295-ft) below-grade circular shaft located near the Main Tunnel midpoint to house the gate system. The outer shell of the shaft has been designed and constructed under a separate contract by the USACE. This shaft will be used for construction of the MTS and eventually will contain the high-head wheel gates for controlling flow between the TARP Mainstream Tunnel and McCook Reservoir.
- Construction Shaft (contractor option)—an optional, 7.6-m (25-ft) ID, approximately 87-m (285-ft) below-grade shaft to be located at approximately 91-m (300-ft) downstream of the Mainstream Tunnel Connection. Kiewit elected to build the construction shaft. The construction shaft is not a requirement for operation of the system.
- Gates—installation of six rectangular wheel gates and the associated gate controls. Each bifurcation of the Main Tunnel contains one main gate and two guard gates–one upstream and one downstream of the main gate. The gates and associated gate hydraulic cylinders and controls were manufactured under a separate contract and will be provided to Kiewit as government furnished items. The gates were designed by Black & Veatch and were fabricated by Oregon Iron Works.
- Main Tunnel/Mainstream Tunnel Connection—the live connection from the MTS to the existing Mainstream Tunnel. The connection geometry was analyzed and evaluated to minimize potential turbulence and cavitation using computational fluid dynamics (CFD).

- MTS/McCook Reservoir Connection—the Main Tunnel portal connection to the McCook Reservoir, including the construction of an Energy Dissipation Structure. The portal will be excavated in rock, with long-term support provided by rock bolts and shotcrete. Hydraulic structures have been designed to minimize erosion during reservoir filling and emptying.
- Control Building—a surface facility to house gate operating controls, hydraulic power units and provide limited storage.

The MTS design, construction, operation, and commissioning will be coordinated with the overall McCook Reservoir water control plan as well as the reservoir excavation, high wall stabilization, groundwater protection system construction, and Distribution Tunnel connections.

The MTS includes a live connection to the Mainstream Tunnel. Operating Mainstream Tunnel disruptions will be minimized as part of the live connection construction planning and all other MTS facilities must be completed and ready to receive CSO water before the connection can be completed. This connection will be one of the more challenging aspects of the construction project.

Future Components of McCook Reservoir

Reservoir excavations, distribution tunnel connections, and the final reservoir preparation work is now under design. Rock excavated from the reservoir is hauled to a nearby quarry and the market demand for aggregate influences the rate of reservoir excavations. Final reservoir preparation will include removal of rock plugs between the distribution tunnel and McCook Reservoir, installation of inlet/outlet works, completion of ongoing grout curtain installation around the reservoir perimeter, and reservoir slope stabilization.

GEOLOGIC CONDITIONS

The McCook Reservoir and MTS project site is located in Lyons Township, Cook County, Illinois. It is bordered by the Stevenson Expressway (I-55) and the Des Plaines River to the north. The MWRDGC operates sludge-drying beds to the east of the reservoir. Also, the Illinois-Michigan Sanitary and Ship Canal borders the facility to south and southwest. The site topography is covered by concrete and asphaltic surfaces as part of the solids waste treatment operations and surface deposits comprising variable fill and poorly sorted glacial till with cobbles and boulders.

The overburden is underlain by sedimentary rock. The bedrock surface has been incised by glacial and pre-glacial erosion. The bedrock surface is generally weathered and fractured, with the frequency of fracturing decreasing with depth. Some fractures have been enlarged by solutioning and some vuggy porosity is present. Bedrock consists of massive, relatively homogenous Silurian and late Ordovician dolomites. These rocks form a relatively uniform 300+ feet thick sequence across the site and incorporate the Racine Formation, Sugar Run Formation, Joliet Formation, Kankakee Formation, Elwood Formation and Wilhelmi Formation of Silurian age. The tunnel is located in the Kankakee and Elwood Formations. The geological profile for the site vicinity is shown in Figure 4.

MAIN GATE SHAFT

The excavation and initial support of the Main Gate Shaft was designed by USACE and constructed by McHugh Construction with Notice to Proceed in October 2009 and construction completion in August 2011. Figure 5 and Figure 6 demonstrate various stages of construction of the shaft.

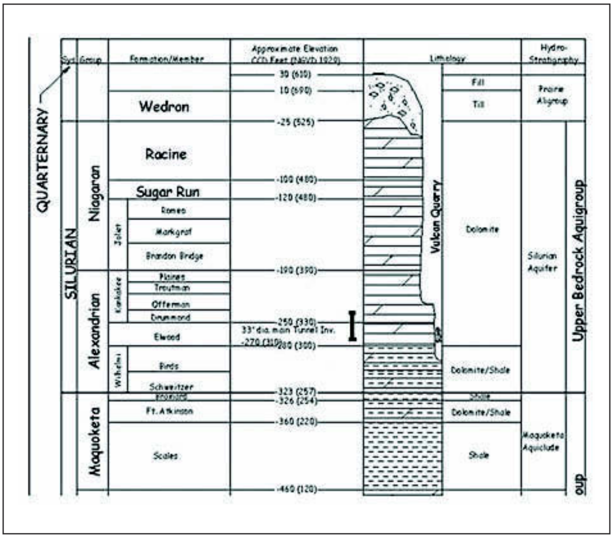

Figure 4. Geological profile of the McCook Reservoir site (Source: Geotechnical Data Report)

Figure 5. Main gate shaft during excavation

Figure 6. Main gate shaft at final depth

The design of the gates and operating cylinders was performed by Black & Veatch under a separate contract and the component fabrication is complete. The gate shaft will have a 27-m (88-ft) finished diameter and a 0.9-m (3-ft) thick concrete liner to a depth of approximately 73-m (240 ft).

The shaft will house the gates and all the components necessary to operate the gates. At the base of the shaft, the Main Tunnel will be split into two sections (bifurcation) so that the flow can be regulated into one or both of the bifurcations. The flow in each bifurcation will be regulated by a set of three gates, one main gate and two guard gates (total of six gates). Provisions have been made to provide man-basket access to the Main Tunnel in order to perform general maintenance on the gates and gate slots.

Gate Design

The design includes six gates (i.e., two main gates and four guard gates). The main gates were designed to resist flow in both directions, whereas the guard gates were

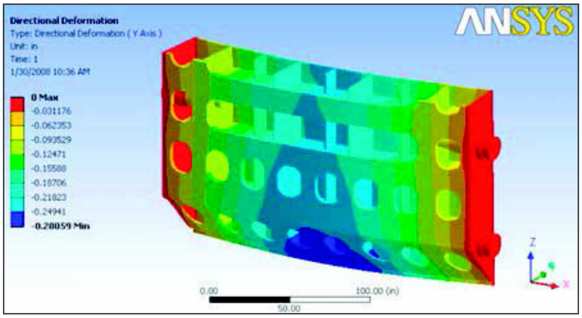

Figure 7. Gate ANSYS model (created from 3-D CAD file)

Figure 8. Gate fabrication in progress

designed to resist flow in only one direction. Each gate is operated by a hydraulic cylinder and operates in a guide slot by means of wheels. The main gates are 5.49-m (18 ft) wide and 9.52-m (31.23-ft) in height. The guard gates are 4.98-m (16.33-ft) wide and 9.21-m (30.23-ft) in height. Each main gate weighs approximately 230 kips. Each gate was designed to resist a static load of 79-m (260-ft) of hydraulic head. Figure 7 shows one example of the gate analysis deformation results using ANSYS. Figure 8 shows the gates during fabrication.

GEOTECHNICAL INSTRUMENTATION

Geotechnical instrumentation has been installed by Kiewit to monitor ground movements during construction. Multiple point borehole extensometers have been installed over the tunnel near the shafts and at the connection with the Mainstream Tunnel. Convergence monitoring points have also been installed in the tunnel crown and quarter arch to monitor tunnel convergence near the shafts. Inclinometers have been installed adjacent to where the tunnel crosses a grout curtain. Surface and subsurface monitoring points have been installed where the tunnel crosses the CSX railroad lines. A utility monitoring point has been installed on a 30-inch gas main that crosses the tunnel alignment. Also, groundwater monitoring wells have been installed on both sides of the grout curtain adjacent to the tunnel crossing.

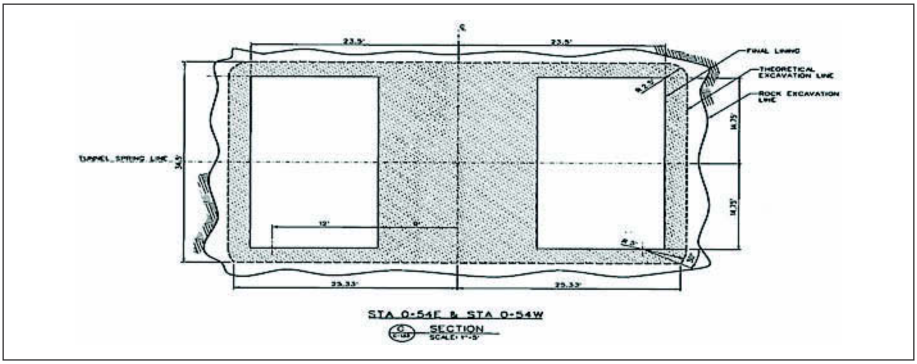

Figure 9. Bifurcation cross section

TUNNEL AND SHAFT EXCAVATION PROGRESS

Kiewit is excavating the tunnel using a two phase approach. The first phase consists of excavating top headings from the Main Gate Access Shaft (MGAS) in both the east and west directions simultaneously. Kiewit is excavating approximately fifty percent of the tunnel cross section with the top heading. The excavated cross section varies in dimensions in the bifurcation between Sta. 0+44 E&W to Sta. 1+82 E&W. From Sta. 1+82 E&W to Sta. 7+07E and Sta. 7+59W, the excavated cross section has constant dimensions. From Sta. 7+07E to Sta. 8+52E, the excavated cross section varies as the tunnel transitions from a circular cross section to a flat-topped circular cross section at the connection to Mainstream Tunnel. This unique cross section was required to reduce turbulence and cavitation during peak inflow events. Figures 9, 10, and 11 show various tunnel cross sections.

Kiewit is using a Sandvik DT-820 drill jumbo to drill blast holes and a Caterpillar R1600G underground mining loader to load, haul, and dump the blasted muck. Resin-bonded rock dowels and shotcrete are being used as initial support. The rock dowel length and spacing varies in the bifurcation due to variations in roof span and excavation geometry (Figure 12).

After the top heading is excavated from Sta. 0+44E to Sta. 7+60E and from Sta. 0+44W to Sta. 7+09W, Kiewit will stop top heading excavation leaving intact sections of rock between the Mainstream Tunnel on the east and the McCook Quarry Highwall on the west. At this point, Kiewit will begin excavation of the lower bench from the MGAS to Sta. 7+60E and Sta. 7+09W, respectively.

Kiewit will begin excavation of a construction shaft at Sta. 5+41E in the near future. This construction shaft in conjunction with a temporary downstream bulkhead will allow Kiewit to connect to the Mainstream Tunnel and install final lining in the eastern section of the project while isolating the MGAS and bifurcation construction from the Mainstream Tunnel.

The connection to the quarry (McCook Reservoir) is an optional item in the MTS construction contract because quarry excavation is ongoing. The section of the tunnel from Sta. 7+09W to Sta. 7+59W and the outlet portal structure and energy dissipation structure cannot be constructed until the quarry is mined to full depth in the area of this connection.

CONCLUSION

The design of the McCook Main Tunnel System is a culmination of many years of effort by the U.S. Army Corps of Engineers (USACE) and the MWRDGC as the project local sponsor. The McCook Reservoir and Thornton Composite Reservoir will mark the

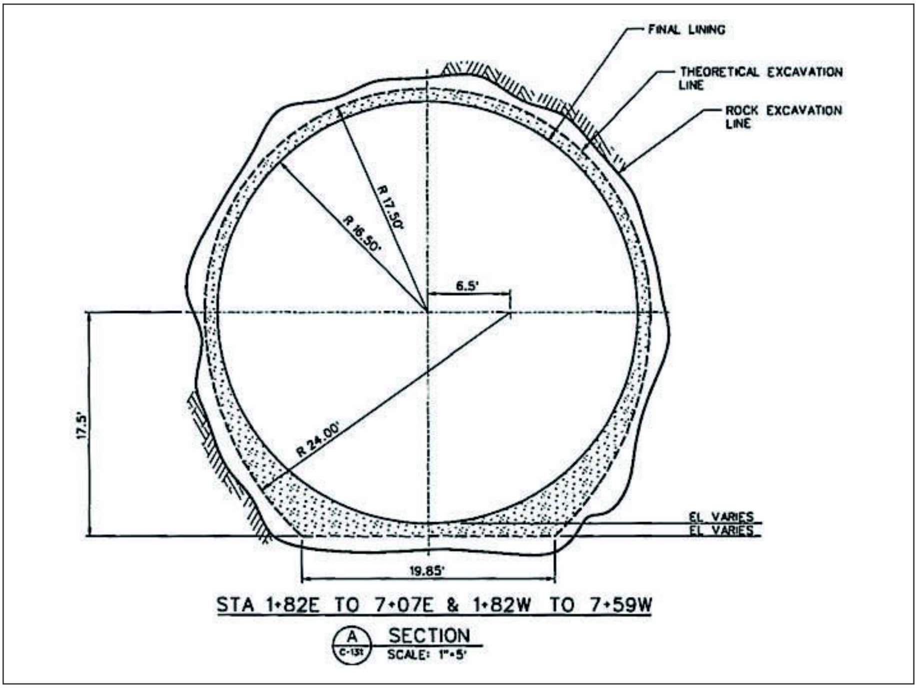

Figure 10. Circular tunnel section

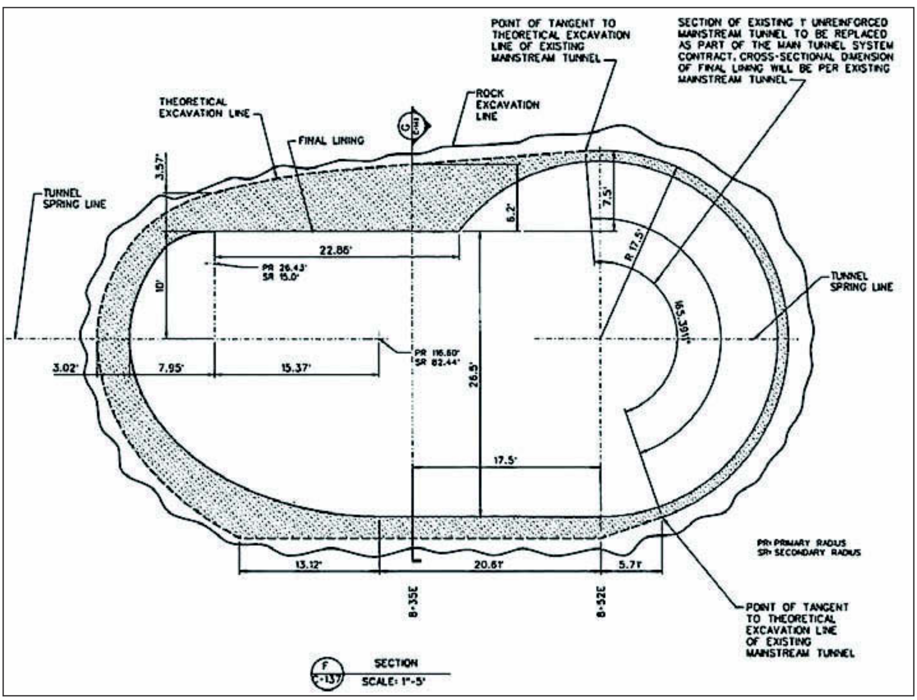

Figure 11. Cross section at connection with mainstream tunnel

Figure 12. Drill jumbo

completion of Phase II of TARP and collectively represent another milestone achievement for protection of Chicago's waterways and providing flood control benefits to many communities in Chicago and Cook County.

Kiewit is making good progress on the tunnel excavation. Most of the tunnel and shaft excavation are expected to be completed in 2013. After that, tunnel lining and MGAS construction will be begin.

The authors acknowledge the guidance, support and cooperation of the staff of the USACE and MWRDGC to this project, and look forward to Kiewit's continued successful construction of this project.

WSSC BI-COUNTY WATER TUNNEL: AN URBAN TUNNEL SUCCESS STORY

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ABSTRACT

This paper presents lessons learned from the planning, design, and construction of the successful $113 million, 8.5 km long (5.3 mi) Bi-County Water Tunnel project. The tunnel, located in the Maryland suburbs immediately north of Washington DC, completes the water supply transmission main from the Potomac Water Treatment Facility across Montgomery County to Prince George's County. Alignment selection, a well-planned community outreach process, and thoughtful geotechnical baseline considerations during project planning by the Washington Suburban Sanitary Commission (WSSC) laid the foundation for success. While construction is still underway, open communication between the contractor, construction manager, designer, and the owner has contributed to all issues being resolved without lengthy or acrimonious dispute.

INTRODUCTION

The WSSC provides water and wastewater service to 1.8 million customers in Montgomery and Prince Georges Counties, Maryland. The Bi-County Water Tunnel completes the water supply transmission main from the Potomac Water Treatment Facility across Montgomery County to Prince George's County. This project was conceived over 20 years ago. WSSC monitored demand to determine when to implement the project and during September 2000, hired a team led by Black & Veatch to perform alignment studies and design. The alignment study and public outreach process determined that a deep rock tunnel was the most favorable alternative, with the benefits of reduced impact to the neighborhoods outweighing the marginal increased cost. Design issues included permitting difficulties and removal of a service shaft leading to a single heading of almost 7.2 km (4.5 mi) of 3 m diameter (10 ft) tunnel resulting in constructability issues with ventilation. Contractual issues included payment terms based on rock quality and unit priced support elements. Design proceeded through to successful bidding of the project, with a low bid of $113 million, below the engineer's estimate of $130 million. Because the bid was 13% below the engineer's estimate, a detailed evaluation of the bidder's qualifications was performed. Construction NTP was issued to Renda, Southland, SAK Joint Venture (RSS JV) on August 3, 2009. Challenges encountered throughout the construction period included unexpectedly high anticipated water at a shaft location, very poor rock quality leading to severe overbreak behind the tunnel boring machine (TBM), a major TBM breakdown, other equipment

Table 1. Scope of work

Item	Description
Shafts (three total)	One primary working shaft for launching TBM on east heading and relaunch on west heading. Two end retrieval shafts for removing TBM.
Tunnel	8.5 km long (5.3 mi), 3 m diameter (10 ft), 27.4–85.3 m deep (90–280 ft) hard rock tunnel.
Pipeline	2.1 m (7 ft) diameter steel water main and backfill grouting.
Surface pipeline tie-in at two end shafts	Approx. 61 m (200 ft) of open-cut pipeline segments and vaults to tie-in tunnel to existing tunnel/surface piping transmission mains.
Pipeline relining	Approx. 122 m (400 ft) of surface pipeline relining beneath I-270.
Cathodic protection	Cathodic protection system for surface pipe at two end shafts.
Corrosion monitoring	Tunnel pipe corrosion monitoring system at five intermediate small diameter shaft locations (drilled and cased) and the three large diameter shaft locations.

breakdowns, and rearrangement of connection chamber pipelines. This paper reviews the planning, design, and construction issues and how they were resolved by a combination of clear contract documents and good communication between the WSSC and RSS JV teams. While there were early concerns that the bid price was 13% less than the Engineer's estimate, as of January 1, 2013, work was proceeding with less than 1% in change orders.

PLANNING

The Bi-County Water Tunnel project completes a main water supply link for two suburban counties (Montgomery and Prince Georges), north of Washington DC (see Table 1 for project components). The network connection points were fixed at the east and west ends of the tunnel, but there was some latitude for the horizontal and vertical alignment selection exercise that is all-important when planning linear infrastructure in such a densely populated area.

The basic objectives of the planning study included the following:

1. Identify feasible alignment alternatives for the 2.1 m diameter (7 ft) water supply main.
2. Detail the approach for developing, evaluating, and ranking the alternatives.
3. Through an extensive community outreach component, inform the public and solicit input.
4. Gather information to be used in assessing the magnitude of each impact.
5. Assess community, environmental, economic, engineering, construction, and regulatory impacts.
6. Identify a recommended alignment and construction method that takes into account above impacts.

Black & Veatch performed the alignment study and project final design. The alignment study was conducted in phases and presented results in four reports that summarized the results of various meetings and workshops. This phased approach allowed continuous input and review during the decision-making process between the WSSC and stakeholders. One stakeholder group was the Policy Review Group (PRG), consisting of WSSC, representatives for elected county officials and the Maryland-National Capital Park and Planning Commission (M-NCPPC). The second stakeholder group was the Citizens Advisory Committee (CAC), consisting of 32 volunteer representatives from homeowners and other stakeholder groups in the study area. The CAC was the

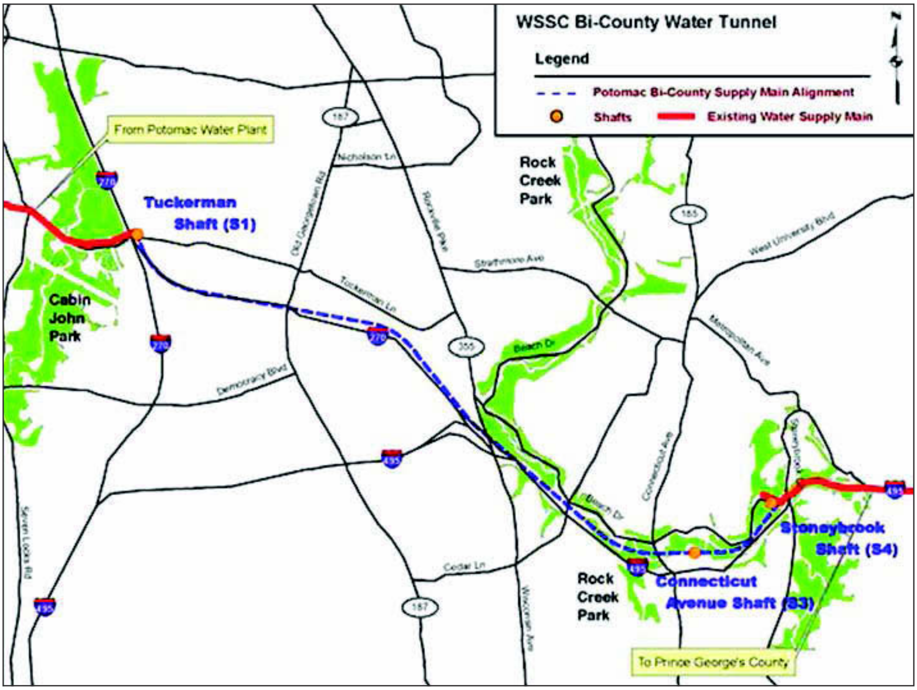

Figure 1. Location plan and tunnel alignment

"voice" of citizens in the study area and acted as an independent liaison between the project team and citizens during public meetings.

1. The **Initial Alignment Development Report** took a broad look at possible alignments to provide baseline information from which final alignment decisions would be made. It provided basic information on 15 possible alternatives and narrowed these to nine possibilities. These consisted of two open-cut, and four tunnel alignments, with three open-cut/tunnel combinations.
2. The **Refined Alignments Report** looked more closely at the nine alternatives. Evaluation criteria were cost, constructability, schedule/permits, environmental impacts, community impacts, and operations and maintainability. The decision-making process revealed that the tunnel alternatives outscored the others by a wide margin even though they did not have the lowest cost. As a result, this report carried forward the three most promising tunnel alternatives and (for comparison purposes) the lowest cost open-cut alternative.
3. The **Developed Alignments Report** further refined each of the decision elements that went into the decision model. Since the shaft sites were key considerations for establishing the tunnel alignments, a detailed evaluation was done for the preferred sites. Ranking criteria included site access, land ownership, community impacts, and environmental impacts. At the end of this phase, one tunnel alignment was clearly superior and became the final alignment recommendation. The recommendation was accepted unanimously by all stakeholders.
4. The **Final Alignment Report** provided more detail and a work plan for preliminary design including geotechnical investigation requirements. Figure 1 shows proposed alignment and shaft locations.

Public Outreach

WSSC recognized the need for extensive community outreach and involvement during all phases of the project, and especially during selection of the preferred alignment and construction method. This commitment was a key to the success of the project and its acceptance by stakeholders. Equally important was the presentation of information in a manner that did not pit neighborhoods against each other, but that objectively identified major factors that should be considered in the evaluation and decision making process. All potential alignments passed through highly developed suburban areas and/or environmentally sensitive urban parklands effecting both Prince Georges and Montgomery counties.

DESIGN

Geologically, the project site lies within the Maryland Piedmont Physiographic Province, northwest of the contact between the crystalline Piedmont rocks to the west and the younger Coastal Plain sediments to the east. The soil stratigraphy in the project area consists of fill and alluvial soils underlain by soils derived from in situ weathering of the underlying bedrock. These soils that are derived from bedrock weathering are present along the entire tunnel alignment. The bedrock weathering profile was separated into four distinct zones (per local practice). These are “saprolite” (ISRM weathering zone classification Grade VI), “transition zone” (Grade IV), “weathered bedrock” (Grade III), and “slightly weathered to unweathered bedrock” (Grades I and II) (ISRM, 1985).

The bedrock consists of metamorphosed sedimentary and igneous lithologies including the Sykesville Formation (metamorphic sequence of gneiss and schist), which has been intruded by the Georgetown Intrusive Suite (quartz tonalite with some schist and amphibolites) along central portions of the project alignment. Unconfined compressive strength (UCS) of all the rock types generally ranged from 34.5–137.9 MPa (5,000–20,000 psi), with the 90th percentile UCS observed at 206.8–275.8 MPa (30,000–40,000 psi). The rock was reasonably abrasive, with Cherchar test results averaging 5.0. Based on cores from borings, the rock is determined to be of good to excellent quality, except in isolated shear zones, where quality can be poor. Preliminary packer test results indicate most of the rock mass has low permeability (less than 10^{-5} cm/s), but there are zones as high as 10^{-3} cm/s. Foliation is the major structural feature of the rocks, and it dips to the west at an average of 59 degrees. Secondary joint sets dipping moderately to the east, and another set with gentle dips to the south exist in the rock mass. The vertical alignment for the tunnel was set to maintain a minimum of two tunnel diameters (6 m [20 ft]) of cover of slightly unweathered to weathered rock. This would minimize the potential for encountering mixed face conditions, and the need for heavy initial support or presupport. On this basis, three rock support classes were specified based on ground conditions encountered during the geotechnical investigation. The determination of which class to install was based on RMR values encountered during excavation (Table 2). Support Class III was to account for shear zones composed of crushed rock with little stand-up time, which was encountered in the geotechnical investigation.

Because of space limitations, the Connecticut Avenue Shaft (S-3) was the only one designated as a working shaft. It was required that all tunnel excavation be done from this shaft. The contract documents allowed the 1,250 m (4,102 ft) east reach to be mined with either drill-and-blast methods or TBM. It was required that the west reach be mined with a TBM. The shafts at either end of the alignment (S-1 and S-4) are both retrieval shafts and could be used for pipe placement and grouting. Another intermediate shaft (S-2) was removed from the project at a late stage of design. To avoid confusion with permitting authorities, the shafts were not renumbered.

Table 2. Initial support

Support Class	RMR Value	Suggested Minimum Initial Support
I	>60 (good to very good)	Four 1.5 m long (5 ft) rock bolts placed circumferentially in the crown and every 1.2 m (4 ft) along tunnel axis; friction bolts or mechanical anchorages acceptable.
II	40–60 (fair)	Same rock bolt pattern as Class I with the addition of welded wire fabric
III	<40 (poor to very poor)	Six 1.5 m long (5 ft) rock bolts extending below spring line placed every 1.2 m (4 ft) along axis with steel channels and welded wire fabric; rock bolts are resin grouted #8 deformed bars.

Pipe materials were evaluated using WSSC's criteria for identifying "Best in Class" materials. Steel pipe and PCCP pipe were evaluated, as well as various joint connections and lining and coating options. Based on this analysis, it was determined that steel pipe with butt-welded joints would be the best material. Steel pipe also required a protective mortar lining, either factory or field applied.

The design team summarized their interpretation of the geotechnical data in a geotechnical baseline report (GBR). So that quantitative baselines could be presented for soil, rock, and water, geological data gathered specifically for this project were considered, along with local experience tunneling in similar rock. The GBR was discussed in two 3-day workshops featuring design team leaders, representatives from WSSC, and three industry peer reviewers collectively known as the technical advisory committee (TAC) to make certain that all opinions were heard and considered. The presentation and determination of baselines were a collaborative process that considered the risk tolerance of WSSC and technical concerns and experience of the designer and TAC. The GBR was then aligned with specifications, drawings, and payment conditions, making sure that unit prices for additional support elements could be noted and paid for without dispute in the field. While the philosophy of the GBR was that baselines reflect the designer's opinion of conditions that would be actually encountered underground, there were selected baselines that were significantly different from what could be expected from analysis of the data alone, including groundwater and rock quality baselines. The GBR was written to serve as the baseline conditions for contractors to use in preparing their bids.

Groundwater Inflow

Tunnel water inflow was analyzed using Heuer's modified method (2005) assuming 10 diameters (10D) of cover leading to a predicted steady state inflow (*Qs*) of 8,060 Lpm (2,129 gpm) peak and a maximum initial heading inflow (*Qh*) of 681 Lpm (180 gpm). The steady state inflow estimate also assumed that the flow from the highest inflow zones (highest Heuer permeability category) would be grouted by the contractor to limit water treatment volume and maintain a favorable working environment. The final baseline (9,464 Lpm [2,500 gpm] steady state and 2,650 Lpm [700 gpm] peak per heading) also reflected water disposal permit limits and requirements, and historical information relating to groundwater inflow associated with fracture zones during construction of the adjacent East Bi-County Tunnel. The intent was to clearly define the maximum water treatment volumes at the surface and make these consistent with the maximum allowable volumes of water pumped from the tunnel in the permit. It was required that the specified capacity for the water treatment plant exceed the baseline steady state inflow volume (*Qs*). During construction, this proved to be a good risk-based decision as daily maximum and annual groundwater withdrawal permit conditions were not exceeded.

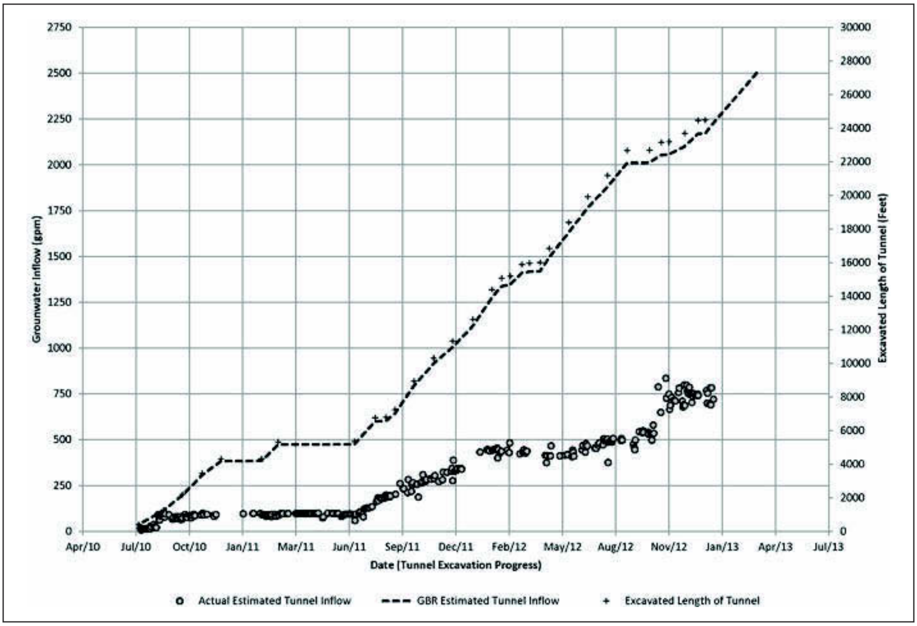

Figure 2. Baseline and actual groundwater encountered during excavation of the Bi-County Tunnel

Table 3. Ground classification

Ground Class	Contract Quantities from Bid Sheet, m (ft)	Observed Quantities Through Sta. 45+27, m (ft)
I	7,292 (23,925) 85%	6,690 (21,949) 92.9%
II	858 (2,815) 10%	471 (1,547) 6.5%
III	429 (1,407) 5%	41 (134) 0.6%
Total	8,579 (28,147)	7,202 (23,630)

Highest steady state inflows measured to date are 3,690 Lpm (975 gpm), with approximately 914 m (3,000 ft) to mine at time of writing (Figure 2).

Rock Quality

The amount and distribution of rock quality were deliberately biased towards Type II and Type III categories. It was acknowledged that the bid price would likely increase if the anticipated percentage of lower rock quality categories was increased. However, it was also believed that increasing these percentages would reduce the likelihood of an unbalanced unit cost for these categories and reduce the owner's risk of overpayment, particularly for the Type III rock quality category. During bidding this was proven to be a good decision as bid prices for the Type II and Type III rock quality categories were reasonably closely grouped, meaning that no unbalancing of bids on this pay item was experienced. See actual versus baseline distributions of rock quality categories in Table 3.

RISK MANAGEMENT

Risk management has been an ongoing point of focus for the project team from early in the design through project completion. The project risk register was first put together during preliminary design and was continued in different formats through final design and construction. Further discussion regarding the project risk register during the design phase can be found in Goodfellow et al. (2009). While the risk register was never considered as a contract document during bidding, a specific clause appeared in the general conditions of the contract that promoted the use of a risk register to identify potential risks during all phases of the construction effort. This clause referred to using a risk register as a tool during each progress meeting to discuss the current risks, how they could be avoided or reduced, what actions should be taken, who was responsible and what risks no longer applied. It was envisioned that the risk register would be used as an open forum for cooperation and communication between the owner and the contractor to manage risk. The risk register, alongside other tools of risk management such as the GBR have been used effectively to promote open discussion and communication about potential hazards seen by any project participant.

BID/QUALIFICATIONS

Bidding saw a reasonably wide spread of bid prices, with the low bidder, RSS JV, 13% below the Engineer's estimate and 22% below the second bidder. Interestingly, tunnel excavation prices generated from the unit prices were closely grouped. The variety of bid prices was seen mostly in the installation price of the steel pipeline. The qualifications contained in the contract documents demanded certain levels of experience from key personnel and the bidding contractor companies. A legal assessment of the low bid contractor's experience noted significant personal experience of RSS JV site management designated for the project. SAK Construction (part of the RSS JV) had sufficient tunnel construction corporate experience to meet the specified criteria. The second bidder placed a bid protest; however, after a review of the situation and further discussion of the qualification criteria, this protest was withdrawn. Construction NTP was issued to RSS JV on August 3, 2009.

CONSTRUCTION

Shaft Excavation

Construction started on the first of three shafts in September 2009 at the main S-3 site. Prior to excavation of each shaft, RSS JV installed shaft dewatering well systems intended to remove groundwater locally around the shaft as excavation proceeded. Dewatering wells were installed in the soil and transition zones. The main working shaft was excavated using liner plate and ring steel to support the soil and transition zones (Figure 3). Ring steel was spaced at 1.5 m (5 ft) centers to support the liner plate rings. The rock zone was excavated by drilling and blasting, with installation of rock bolts and shotcrete to support the rock. Typical round lengths drilled were 3 m (10 ft) in depth. Shaft excavation was completed in March 2010. Minimal groundwater was encountered during excavation of the main working shaft at S-3. Table 4 summarizes

Figure 3. Excavation of main working shaft

pertinent facts about excavation of each shaft. Construction of the S-4 Shaft, approximately 1.6 km (1 mi) east of the main working site/shaft, started in January 2010. RSS JV supported the soil zone using liner plate and ring steel as well and decided to carry the liner plate and ring steel all the way to the bottom of the shaft instead of using rock bolts and shotcrete because of the approximately 227 Lpm (60 gpm) of groundwater inflow into the shaft. The shaft was completed in October 2010.

During installation of the pre-excavation shaft dewatering well system at S-1, inflows were determined to be higher than anticipated. RSS JV submitted a differing subsurface condition (DSC) based on its field testing data. Following review of the DSC by WSSC and its project team, a settlement was negotiated. RSS JV proposed installation of a secant pile ring wall to support the soil and transition zones and socket the overlapping piles into the upper rock zone so that any potential groundwater inflows would be sealed off during shaft excavation. Open communication between the owner and the contractor during a contentious issue that seemed to linger longer than usual ended with minimal additional costs to the project while reducing overall risk to work going forward. This was a case where perhaps the ground support system selected for the fix was more than required, but was not cost prohibitive and provided less risk for the unknown. RSS JV did encounter some water issues at the transition from the secant

Table 4. Shaft excavation data

	S-3 Shaft (Main Working Shaft) m (ft)	S-4 Shaft, m (ft)	S-1 Shaft, m (ft)
Excavation Diameter	Soil: 11 (36) Rock: 9.1 (30)	Soil: 4.6 (15) Rock: 4.6 (15)	Soil: 7.6 (25) Rock: 6.1 (20)
Finish Diameter	Not Applicable	2.1 (7)	2.1 (7)
Depth to Water Table	9.8 (32)	3 (10)	1.8 (6)
Depth to Top of Rock	23.5 (77)	13.7 (45)	15.2 (50)
Depth of Shaft to Tunnel Crown	46.3 (152)	31.1 (102)	38.4 (126)
Depth of Shaft (bottom sump area)	50.6 (166)	35.1 (115)	42.7 (140)
Soil Support Method	Liner Plate, Ring Steel	Liner Plate, Ring Steel	Overlapping Secant Pile Ring Wall
Rock Support Method	Rock Bolts and Shotcrete	Liner Plate, Ring Steel	Rock Bolts
Final Shaft Lining	None, Backfilled	Steel Pipe with Backfill Grout	Steel Pipe with Backfill Grout
Planned Excavation Rate Average (to tunnel crown)	36 work days, 1.2 m/shift (4 ft/shift), 6.2 m/shift (20.3 ft/wk)	29 work days, 1.1 m/shift (3.5 ft/shift), 5.4 m/shift (17.6 ft/wk)	35 work days, 1.1 m/shift (3.6 ft/shift), 5.6 m/shift (18.4 ft/wk)
Actual Excavation Rate Average (to tunnel crown)	114 work days, 0.2 m/shift (0.8 ft/shift), 1.9 m/shift (6.3 ft/wk)	119 work days, 0.3 m/shift (0.9 ft/shift), 1.3 m/shift (4.3 ft/wk)	104 work days, 0.4 m/shift (1.2 ft/shift), 1.9 m/shift (6.2 ft/wk)
Production	Single, then transitioned to double shift operation, typical 10 hr work shifts	Single shift operation, typically 10 hr work shift (in residential area)	Single shift operation, typically 10 hr work shift (in residential area)

pile wall to rock and attempted to pressure grout the area to reduce inflows, with limited success. The secant pile wall, however, did provide a means to seal off most of the groundwater RSS JV expected to encounter. The secant pile wall was constructed during March/April 2011, and the S-1 shaft excavated during May through November 2011.

TBM Fabrication/Refurbishment

TBM fabrication and refurbishment were a joint effort of Robbins Co. out of Solon, Ohio; SAK of St. Louis, Mo.; and RSS JV. TBM #91-155-10 (the designated number and serial number of the Bi-County TBM) had been used on multiple prior projects, but underwent some significant changes for this project. Power was increased to 1,200 horsepower (4x300 hp drive motors, and matched to Flinder gear boxes). The TBM was equipped with new programmable logic controllers (PLCs) and variable frequency drives (VFDs). The size of the main bearing was also increased to 2,489 mm (98 in.), from the original 711 mm (28 in.). This size change mandated a 1 m (3.3 ft) extension on the main beam box section to accommodate the larger bearing and subsequent relocation of the main beam belt. The cutterhead was fabricated with six lifting buckets and twenty-two 483 mm (19 in.) disc cutters. SAK installed a new lubrication system on the TBM at its St. Louis facility and refurbished all of the TBM back-up components. The joint effort involved in rebuilding and refurbishing components of the TBM was successful in delivering a machine well suited for the project.

Tunnel Excavation: East Heading

Following completion of the main working shaft at S-3, RSS JV proceeded to excavate the starter and tail tunnels in preparation for assembly of the TBM and trailing gear. The starter and tail tunnels were excavated to a diameter of 3.7 m (12 ft) for a distance of approximately 61 m (200 ft). Both launching tunnels were excavated over a three-month period. Refer to Figure 4.

RSS JV installed a concrete working slab in the east and west starter tunnels as well as concrete side launch walls in the east starter tunnel and mobilized the TBM and trailing gear in the east starter tunnel (Figures 5 and 6). The TBM was launched near the end of July 2010. Early during the start of excavation, the No. 4 TBM drive motor malfunctioned and was removed from service. Despite the loss of one of the four drive motors, steady progress was made during excavation of the 1,250 m long (4,102 ft) east tunnel heading. The tunnel reached the eastern S-1 shaft late November 2010.

One Ground Support Class III area was encountered during excavation of the east tunnel that was 23.2 m long (76 ft), and five Class II areas were encountered that were 30 m long (98 ft). The TBM cutterhead was removed from the cutterhead support within the S-4 shaft, and transported back to the main working shaft at S-3. The trailing gear and remainder of the main beam assembly were backed up using the grippers and removed from the tunnel at the main working shaft. Maintenance was performed on the machine while on the surface at the S-3 site, and it was then mobilized into the tunnel for the start of the west tunnel drive.

Figure 4. Excavation of starter tunnel

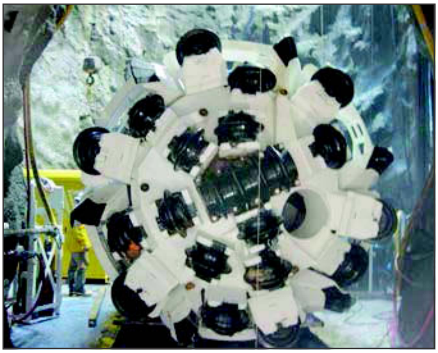

Figure 5. TBM cutterhead at bottom of S-3 shaft

Figure 6. TBM cutterhead and support lowered into shaft

Tunnel Excavation: West Heading

The TBM was relaunched in February 2011. After about one month of tunnel excavation, a major equipment failure occurred with no warning. An investigation determined that there was a metallurgy issue with the steel of the No. 2 pinion gear. A portion of the pinion gear tooth sheared off, causing complete destruction of the No. 2 gear reducers and the ring gear. Damage was also caused to the other three gear reducers. The TBM was removed from the tunnel and damaged components shipped back to Robbins in Solon, Ohio, for repair. The main bearing was not damaged, but the bull gear was damaged. However, RSS JV located a replacement bull gear that prevented additional delay to the repair effort. As a result of the breakdown, RSS JV experienced approximately 4 months of down time while repairs were made. During the down time, Robbins in coordination with RSS JV changed the Flinder gear boxes to more robust Lohmann gear boxes. This also required modifications to the motor mounting area on the back side of the cutterhead support. The WSSC viewed the incident as beyond the control of RSS JV and agreed to a noncompensable time extension to the contract that recovered the time lost because of the event.

The TBM was relaunched in July 2011. Less than a month later the TBM encountered ground that was unstable initially and resulted in ground caving in front of and onto the top and side shields of the TBM. Initial groundwater inflow in the area was 568 Lpm (150 gpm) that eventually reduced to 227 Lpm (60 gpm). The blocky ground encountered had to be excavated by hand to prevent damage to the TBM and mucking conveyors. The void created by the unstable ground was approximately 1.8 m (6 feet) above the top shield of the machine and extended about 6.7 m (22 feet) in length, RSS JV held the opinion that the unstable ground could not be supported using the support methods stipulated in the contract. Contract language indicated that if the contractor became aware of a differing subsurface condition, work was to stop and the owner notified. Through discussions between WSSC and RSS JV and based on verbal authority from WSSC, RSS JV proceeded to order 30 m (100 ft) of full ring steel sets and lagging. The WSSC and RSS JV teams considered the impact delays could have on the overall schedule if blocky ground conditions were encountered again and decided to reduce the risk by ordering additional steel sets to retain an inventory at the site. The sets were delivered in a week, and RSS JV proceeded to support the ground and move the TBM through the 11 m (36 ft) area. Following the claim investigation, it was determined that the ground encountered was a differing subsurface condition. A settlement was negotiated between WSSC and RSS JV using an actual cost reimbursement approach to determine the cost and time associated with the delay. Coordination and

Table 5. Tunnel excavation chronology and data

East Tunnel Heading	
TBM	Launch: July 23, 2010 Hole Through: November 23, 2010
Reach Length	1,194 m (3,917 ft) 1,250 m (4,102 ft) including starter tunnel
Excavation Rate	Planned: 23.4 m/day (76.8 ft/day) Actual: 14 m/day (46.1 ft/day) (schedule planned days: Mon to Fri)
Production Average	Two 10.6 hour shifts, maintenance as needed
Average Penetration Rate	Excavation: 2.2 m/hr (7.3 ft/hr) TBM Availability: 42%
West Tunnel Heading	
TBM	Launch: February 2, 2010 Hole Through: Not completed
Reach Length	7,276 m (23,873 ft) 7,329 m (24,045 ft) including starter tunnel
Excavation Rate	Planned: 23.9 m/day (78.3 ft/day) Actual: 16.7 m/day (54.9 ft/day); as of 12/31/2012: 6,143 m (20,153 ft)
Production Average	Two 10.6 hour shifts, then transitioned to Three 8 hour shifts, maintenance as needed
Average Penetration Rate	Excavation: 3 m/hr (10 ft/hr) TBM Availability: 45%

communication among team members resulted in a short delay, but were crucial and led to quick decisions that prevented additional delays to the project.

Excavation is still underway as of December 31, 2012. RSS JV encountered one other instance of blocky ground that required steel ring sets and lagging to support the ground and resulted in a one week delay. See Table 3 for ground class observations made through December 31, 2012. TBM maintenance issues that have resulted in schedule delays include, drilling of intermediate corrosion monitoring shafts, some of which were being used by RSS JV for tunnel ventilation and repair/replacement of inner and outer cutterhead support seals. TBM hole-through along the west tunnel heading is expected around the end of March or early April 2013. See Table 5 for tunnel excavation chronology and data.

Cutter wear during tunnel excavation has remained fairly consistent and within acceptable tolerances for the material being mined. However, lifting bucket wear has been significant because of the abrasive qualities of the material being mined. Four complete bucket changes were performed during excavation of the west drive. This may have contributed to the loss of the cutterhead support seals as the material not being picked up cleanly could contribute to pressure and wear on the seal retainer. Normal maintenance performed on the TBM entails frequent checks of all bolts, main beam, cutterhead, thrust cylinders, and belt frames. Lube oil and filters are changed every 250 machine operating hours as a standard, but some changes have been initiated sooner because of suspected contamination. Oil is sampled and laboratory tested. Gear box oil changes are performed when indicated, and this oil is also sampled and tested.

East Tunnel Pipeline Installation

Steel pipe installation in the east tunnel went as planned without any major issues. A hydraulic pipe carrier was used to move the pipe from the main working shaft into

Figure 7. 1,829 mm diameter (72 in.) pipe tie-in and valve installation at S-4

the east tunnel. Extensive quality control and quality assurance were required during pipe installation of the pipe to ensure it was manufactured in accordance with the project documents, welders followed approved weld procedures, the welding being performed was inspected for adequacy, and all nondestructive testing on initial and final welded joints passed testing. Following installation and welding of the pipe, RSS JV and its subcontractor backfill grouted the pipe using a 70% cement, 30% fly ash mix. Grout was pumped using multiple lifts to prevent floatation of the pipe. Grout filling was tracked via observation ports, and overall the grouting effort was completed without any issues. Contract documents required 6.9 MPa (1,000 psi) backfill grout at a 28-day strength, and actual grout cube compressive strength testing results were upwards of 41.4 MPa (6,000 psi) at a 28-day strength.

Main Transmission Line Dewatering and System Tie-In

As of December 31, 2012, one main pipeline tie-in was complete at the eastern portion where the BiCounty Water Tunnel ties into the existing tunnel water main system by way of a surface air release and valve vault system. Extensive coordination was required between RSS JV and its subcontractors to order the large-diameter gate valves and isolation joints, which required long lead times. In addition, work included coordination with WSSC to dewater existing water supply transmission mains so RSS JV could cut into an existing 2,438 mm diameter (96 in.) line and install the new 1,829 mm diameter (72 in.) pipe and 1,829 mm solid wedge gate valve. The new valve is housed in a surface valve vault located at the S-4 site. Work on the final western tie-in portion started during the summer of 2012. Figure 7 shows tie-in work at the S-4 site location.

Construction Schedule

As of December 31, 2012, work remained behind schedule. While actual tunnel excavation rates have not matched rates forecasted in the baseline schedule, RSS JV did adjust its planned resource allocations from a 2-shift, 5-day a week operation to a 3-shift, 6-day a week operation. Additionally, the current schedule forecasts pipe installation in the west tunnel during a single-shift, 5-day workweek. RSS JV was in the process of revising its baseline schedule to incorporate its shift/resource changes for pipeline installation to be done in a three-shift operation in the west tunnel. Substantial time should be recovered on the schedule when the change is incorporated and the baseline schedule revised. Figure 8 compares the original baseline dates to as-built dates. The project's scheduled completion date is January 9, 2014. Changes to the contract for the period from NTP to December 31, 2012, extended the project duration by 160 calendar days. Contract changes to date include power outages, storm events, a major TBM breakdown (120 days), and differing subsurface conditions (28 days).

CONCLUSIONS

The WSSC Bi-County Water Tunnel project is a successful example of urban tunnel planning, design, and construction. Although the project is not yet complete, significant

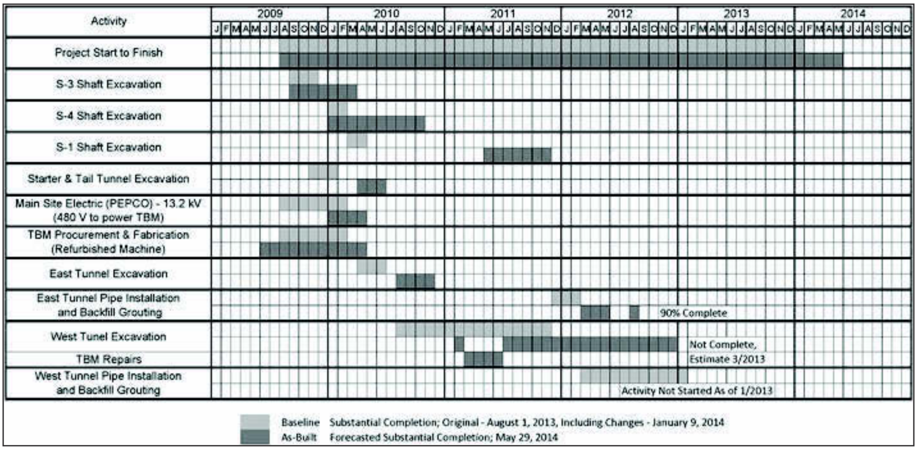

Figure 8. Comparison of baseline and as-built schedules

lessons learned have been garnered thus far and will inform the future work on this project as well as similar future projects. These lessons learned include:

1. Prudent interaction with the local community led to complete buy in for project and alignment decisions.
2. Risk management tools used during design and construction included a risk register and GBR that were successfully applied to the project:
   a. Baselines for percentage of rock mass quality in the tunnel and groundwater inflow were manipulated from purely geotechnical data interpretations to mitigate risks elsewhere in the design, namely to deter unbalancing of bid unit prices and satisfy environmental permit requirements.
3. The project is currently within budget but is experiencing schedule delays because of TBM excavation and equipment issues. A revised baseline schedule will be prepared and will incorporate resource changes for the pipe installation effort in the west tunnel to recover a substantial portion of the delay. Despite the problems along the way WSSC and RSS JV continue to look for ways to improve sequencing of work to meet the schedule end date.
4. Construction issues have been considered and dealt with promptly with a combination of a strong collaborative approach and excellent communication between the owner, designer, construction manager, and contractor.

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WANETA EXPANSION PROJECT— PENSTOCK TUNNELS

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ABSTRACT

The Waneta Expansion Project (WAX) is a $900 million, 335 megawatt, hydroelectric project currently under construction on the Pend d'Oreille river near the city of Trail, British Columbia. This project includes construction of a second power house and a 10 km transmission line to share hydraulic head created by the existing Waneta Dam. The WAX is currently two years into construction with a projected completion in mid 2015.

Redpath/FKCI Waneta Tunnelers (RFK) was sub-contracted by the heavy civil contractor, ASL-JV, an Aecon/SNC-Lavalin joint venture, to excavate two new, parallel intake tunnels each approximately 215 meters in length with an 11 meter finished diameter and inclined on a 17% grade. Once the penstock tunnels were excavated a 300 millimeter thick cast in place concrete liner was installed in each tunnel. This paper will provide details about the top heading and bench excavation method used to develop the parallel intake tunnels and describe the challenges and benefits associated with using a self-advancing tunnel form on a 17% slope.

INTRODUCTION

The Waneta Expansion Project (WAX) is located near the existing Waneta Dam site at the confluence of the Pend d'Oreille and Columbia Rivers approximately 10 km south of Trail, BC, Canada. The WAX owners consist of a partnership between Fortis Inc., Columbia Basin Trust, and the Columbia Power Corporation. This design build project was awarded to SNC-Lavalin in late 2010 and is expected to be operational in mid 2015.

The WAX consists of a new 335 MW powerhouse including two new Francis turbine units, each generating approximately 167 MW. The powerhouse is located downstream of the existing Waneta Dam and will make use of excess water which would otherwise be spilled during the runoff season. Water will be funneled to the turbines through an intake structure and two 10.5 meter diameter, concrete lined penstock tunnels. A 10 km transmission line will connect the powerhouse to existing electrical grids.

Redpath/FKCI Waneta Tunnelers (RFK) was subcontracted by the heavy civil contractor, the Aecon/SNC Lavalin joint venture "ASL-JV," to excavate the two penstock tunnels and install the cast in place concrete liner. RFK is a joint venture partnership between J.S. Redpath Limited, an underground mine contractor located in North Bay, Ontario, and Frontier-Kemper Constructors ULC, a heavy civil tunneling contractor based in Nova Scotia, Canada. Since penstock tunnel excavation was planned using drill and blast methods with significant grade, +17%, the combined experience of these two contractors was a perfect match.

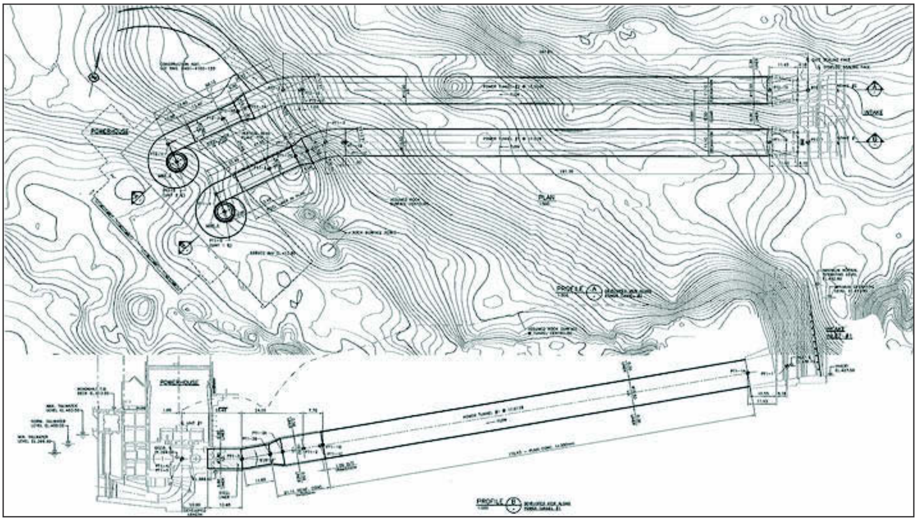

Figure 1. Tunnel plan and profile

SITE SPECIFIC

With the project site being located along two major rivers environmental awareness was, and remains, a major consideration to all work. Protected species such as the White Sturgeon, Rubber Boa snake, and Yellow Belly Marmot reside in areas surrounding the project site. All site contractors were required to adhere to strict environmental regulations especially regarding the use of concrete, explosives, and mobile equipment. An onsite water treatment facility was set up to treat all construction and runoff water which came in contact with the site prior to discharge.

The owner's requirements of zero fly rock and limited blast vibrations added to the excavation complexity. A single lane highway bridge and an existing railroad bridge, constructed during the 1940s, were located approximately 300 meters from the project site and were monitored during every blast. Blast vibrations were also an important consideration since powerhouse, intake, and tunnel excavations all took place within 500 meters of the existing Waneta Dam. All blasting was closely monitored to ensure there were no adverse effects on the dam, railroad and highway bridges. Blast vibrations were kept below 50 mm per second peak particle velocity.

ACCESS ADIT

In order to start the penstock tunnel excavation early, while powerhouse and intake excavations were underway, a smaller access adit tunnel was developed. The access adit was 6 meters wide × 6 meters tall, modified horseshoe in shape, and 135 meters long with a grade of –12%. Developing the access adit allowed RFK to excavate the penstock tunnels and remain off of the surface works critical path and out of the way of other site excavations. The access adit was also used for initial training for those workers not familiar with underground excavations. (See Figure 1. Tunnel plan and profile.)

Four meter round lengths were drilled using a 2-boom Tamrock drill jumbo. A typical round consisted of 75 holes, 45 mm in diameter with three 100 mm diameter relief holes in the cut. Hole spacing was approximately 800 mm × 800 mm. Explosives used were packaged emulsion, "stick powder," with Nonel detonators. All blasts were initiated using electric blasting caps which were tied into detonating cord. Ground support in the access adit consisted of a minimum 50 mm layer of 35 MPa, fiber reinforced,

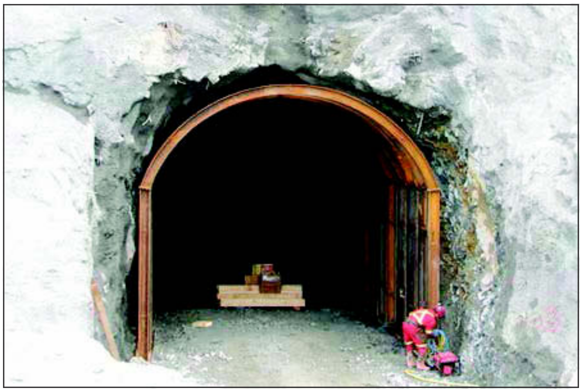

Figure 2. Portal and access adit development

shotcrete followed by a typical 2 meter × 2 meter pattern of 2.4 meter long expandable (Swellex) type rock bolts. The access adit had low cover and would be used as primary ingress/egress during the tunnel excavation and the concrete liner phase. Eight steel arch sets were therefore installed at the portal on one meter centers to provide additional ground support. All utilities and ventilation were routed through the steel sets and down the access adit. RFK was able to complete the 135 meter long access adit in 40 days, working five days per week with two 10-hour shifts per day**.** (See Figure 2. Portal and Access Adit Development.)

PENSTOCK TUNNEL EXCAVATION

Two penstock tunnels were designed parallel, 10 meters apart, horseshoe in shape, and approximately 11 meters in diameter. After excavation a 300 mm thick cast in place concrete liner would be installed for hydraulic efficiency purposes only. Due to the large diameter of the penstock tunnels RFK determined the safest and most efficient excavation method would be top heading and bottom bench. Seperating the excavation into top and bottom halves allowed RFK to keep tight control over ground support and minimize the amount of open ground at any time. Another deciding factor was the availability of smaller mobile equipment.

Based on the tunnel size and expected ground conditions initial planning suggested further dividing the excavation through the use of a split top heading method. In this scenario the top heading in each penstock tunnel would have two separate working faces side by side, one slightly ahead of the other, each approximately 5 meters wide × 5 meters tall. This method, similar to a pilot and slash, would have provided four working faces at any given time yielding greater flexibility in the excavation cycle.

Conversely, sequencing the excavation cycles between the four working faces would have required careful day to day planning and left little room for errors such as equipment breakdowns. Bottom bench excavation was always planned as full face using horizontal drilling and taken after top heading excavations were completed in each tunnel. (See Figure 3. Top heading and bottom bench excavation.)

As access adit excavation neared the intersection of the penstock tunnels RFK encountered better than expected ground conditions and determined that a full width top heading could be taken without compromising safety during the ground support cycle. Based on this information RFK chose to take a full face top heading, 11 meters wide by 5 meters tall, semicircular in shape. Doing so eliminated the flexibility of four separate working faces but allowed RFK to advance the entire top heading with one blast instead of two, making the operation more efficient overall.

Controlled blasting techniques were employed during top heading excavations to help define the excavation line and minimize overbreak in efforts to reduce the amount of concrete placed during the tunnel lining phase. Top heading drill patterns had approximately 98 blast holes with interior spacing averaging 800 mm × 800 mm. The perimeter of each round was line drilled with spacing averaging 450 mm and every other hole loaded during the blast. Four meter round lengths were drilled using the Tamrock 2-boom jumbo. Mucking was done using two CAT Elphinstone R1700G Load Haul Dump (LHD) scoops. Due to the wide top heading, 11 meters, RFK was able to use both LHDs to muck the face simultaneously, passing in the tunnel near the face. The use of two LHDs at once helped decrease the mucking cycle.

Figure 3. Top heading and bottom bench excavation

From the access adit intersection power tunnel excavation was uphill at approximately 17% for 190 meters and downhill at approximately –15% for 20 meters in both tunnels. Blasted rock was taken to the downhill side of one power tunnel which was used as a muck bay. The downhill side of the adjacent tunnel was used as a sump. Blasted rock was hauled to surface from the muck bay after the heading was completely mucked out.

Initial temporary ground support, identical to that installed in the access adit, was installed in the penstock tunnels to allow RFK to continually advance the heading. All shotcrete was placed using a Normet Spraymec robotic shotcrete machine. Shotcrete was transported from surface using a single, 7 cubic meter, underground remix truck. Permanent ground support, consisting of 22 mm diameter × 4 meter long, fully grouted rebar dowels were installed non-critical path behind the working face. All permanent ground support was installed based on a prescription by the onsite geotechnical engineer. Five separate rock classifications were defined and evaluations carried out daily after each blast was taken. Ground support classifications ranged from Class I, spot dowels only, to Class V, 100 mm of fiber reinforced shotcrete and lattice girders. Ground conditions in the penstock tunnels were good enough that RFK was only required to install Class I and Class II ground support.

To ventilate the underground excavations RFK used a fully reversible suction system. Two 1.3 meter diameter, 200-hundred horsepower ventilation fans were located on surface adjacent to the portal entrance. Two separate steel ducts of the same diameter were connected to the fans and advanced through the access adit. Once at the access adit/penstock tunnel intersection, one steel duct would split off to each tunnel. The steel ventilation duct always remained approximately 18 meters away from the working face in the penstock tunnels while a smaller 30 hp, 0.9 meter diameter booster fan was used to push fresh air directly toward the working face. The 30 hp booster fans were set up in each tunnel in a fixed location and used 0.9 meter flexible ducting, advanced with the heading. Average air flows in the penstock tunnels were approximately 5,500 cubic meters per minute. Other utilities such as compressed air and service water were carried in 100 mm diameter HDPE pipe hung along the walls of each tunnel. Electrical cables for tunnel lighting and equipment were carried on the opposite side of the tunnel as the compressed air and water lines.

RFK was able to excavate both top headings, 451 lineal meters total, in 139 days and completed the bottom benches in only 82 days.

Two disadvantages associated with the steep grade excavations were difficulty mucking uphill and increased exposure during ground support operations. RFK was able to mitigate the latter by using a remote shotcrete arm on the Normet Spraymec allowing an initial layer of shotcrete to be installed prior to bolting. Shotcrete accelerator was used to reduce cure times so that bolts could be installed during the same shift. A mechanized bolting machine was used to install the expandable bolts, again minimizing exposure to the operators. Excavating and mucking uphill was technically more difficult for operators but the chosen equipment was designed for underground mines where steep grades are more common. The slope was hard on tires, but with careful operation standard equipment was used effectively without requiring modification.

One advantage of the steep grade was that a dewatering pump was not required during ground support, drilling, and loading cycles to keep the face dry.

CONCRETE LINER

A smooth concrete liner was specified to minimize head losses through the penstock tunnels. Initial plans were for a modified horseshoe, or D shaped, tunnel cross section with finished inside dimensions of 10 meters × 10 meters. Prior to the start of excavation a change order was approved converting the finished tunnel shape to full round 10.5 meter finished diameter. With permanent ground support being installed during the excavation phase the liners would not be exposed to any ground loads. The maximum design load was thus the pressure differential caused by rapid dewatering. While the tunnels are in use, and full of water, the ground around them will become saturated with an equal hydrostatic pressure. In an emergency the intake gates could be closed and water would drain out of the tunnels in approximately 2 minutes. In this case of rapid dewatering the surrounding ground will maintain the full hydraulic head of up to 70 meters on the concrete liner until the pressure slowly dissipates through leakage. With this in mind the designers specified the use of 35MPa plain concrete to cast the 300 mm thick liner.

It is notable that no reinforcing bar or fiber was required in the final concrete liner. The tunnel excavations were fully supported before concrete placement, and in the circular shape plain concrete was able to meet the design requirements. Reinforcement could have been used to reduce shrinkage cracking of the liner, but minor cracking was actually preferred as it will allow drainage into the tunnel during a rapid dewatering condition. Construction joints similarly required no water stop or bonding agents which would hinder equalization of water pressures.

The design requirements did have restrictive finish specifications. The tunnel liner could not vary from line or grade by more than 12 mm or by dimension/shape by more than 0.5%. On the 10.5 meter diameter finished tunnel up to 52 mm of differential between the height and width were allowed. Liner finish quality had to meet the British Columbia Ministry of Transportation Class II requirements. This required all honeycombs over 25 mm diameter be filled, all bug holes over 5 mm diameter be pointed and the surface given a rubbed finish where more than 50 such voids occurred per square meter. Surface irregularities 3 mm high were allowed with restrictions on their size and number.

TUNNEL FORM DESIGN AND FABRICATION

In order to keep tunnel concrete works off the overall project's critical path RFK requested proposals from seven different suppliers for a self advancing concrete form which would allow a full pour cycle every 24 hours, safely operate on a 17% slope, and be able to meet the specified finish requirements. Ceresola Tunnel Lining Systems

(CTLS) of Switzerland (now Max Bögl Schweiz AG) was the chosen supplier and undertook the design and fabrication of a walking beam style steel form. It was agreed the form would be able to cast a 7.5 meter long full round section of the liner, walk itself through the tunnel on its own carrier, and be designed to operate on a 17% slope. The form used by RFK on this project was the largest full round form CTLS had constructed to date.

Since the tunnel form was only 7.5 meters long internal supports "spud pins" were not required. To prevent movement during concrete placement the front of the form was braced against the surrounding rock and the rear against the previous concrete pour by six large screw jacks on each end. Two screw jacks in the crown, one on each side, and two in the invert.

In an effort to reduce cycle times a steel framed, cantilever bulkhead system was developed which would fasten to the upstream leading edge of the steel formwork, eliminating the need to brace the bulkhead against the rock. In surface trials the provided cantilever bulkhead system proved difficult to fit around other installations on the end of the form such as walkways and hydraulic cylinders. The individual parts were also too heavy to efficiently assemble by hand on a regular basis. RFK decided to use rough cut 2×8 bulkhead material and support it using traditional wood 2×4 walers and stiff-backs. The wood supports were not designed to withstand concrete loads in cantilever so they were pinned and braced to the perimeter rock using 20 mm steel dowels.

Advancement was accomplished by designing the round form and the carrier to walk itself. Rollers were installed on the carrier beams to allow the form to slide back and forth with the carrier resting on the ground. The same rollers could be used to move the carrier when supported by the form. Not having wheels on the ground meant the form could be walked over mildly uneven surfaces, did not require a rail system, and was stable in the sloped tunnel. The form could also be shifted sideways by means of small cylinders in the feet of the carrier.

The tunnel form was fabricated and structural elements assembled for testing and inspection in Seveso, Italy. After workshop inspection the form was disassembled and loaded into 11 standard 40 foot shipping containers for transport to Trail, BC. On arrival the 15 full form elements (1.5 meter × 7.5 meter × 1.6 meter up to 4200 kg each), 10 half elements for the invert, handrails, and miscellaneous parts were skidded out of the shipping containers at a storage yard approximately 7 km from the project site. Each element was then individually transported to the assembly location at the project site as needed. (See Figure 4. Tunnel form inspection.)

TUNNEL FORM ASSEMBLY

On site the assembly location was on the surface near the head pond and adjacent to the 40 meter deep intake excavation. The carrier itself was assembled then five elements comprising the crown were bolted together. The crown was lifted as one unit and attached to the carrier. The side elements, or wings, were bolted together and hung from the crown while the invert was assembled in place under the carrier. After the major elements were bolted together the electrical, hydraulic, pneumatic, communications and water systems were installed. Overall, surface assembly took 11 weeks. Once assembled, the form was tested and modifications were made as needed before walking the form under its own power to the edge of the intake excavation.

A location for the shortest lift crane radius (15 meters) into the intake was cleared and leveled for placement of a 275 US ton lattice boom crawler crane. This gave a load limit of 48,000 kg at the self-imposed 75% capacity to avoid the restrictions of a Critical Lift. Fully assembled the form weighed approximately 135,000 kg. (See Figure 5. Tunnel form re-assembly in bottom of intake excavation.)

Figure 4. Tunnel form inspection

The form elements, concrete placer car, and feet were removed to allow the carrier to be lifted and set in the bottom of the intake excavation. The invert, carrier, crown and sides, each containing all installed utilities and sub-assemblies, were then hoisted to the bottom of the intake, in order. Each part was directly attached to the elements already in place, reattaching the invert last.

After full reassembly the form was shifted 20 meters sideways using 200 mm side adjustment cylinders and aligned with Tunnel 2. The form was then walked, as intended, to the downstream starting station of Tunnel 2 over the course of approximately one week. Although designed for the slope, a large moment was inflicted on the system as it was walked down the tunnel. When the carrier was raised and extended forward down the tunnel two screw jack feet on the lower end supported the majority of the system's weight; along with a moment from the extended carrier. An anchor pin assembly was provided on the upper end of the form to arrest longitudinal forces, but it was difficult to prevent load transfer to the vertical supports. Extreme care was required while walking downhill to prevent damage to the screw jacks. Advancing the form back up the tunnel did not have the same challenges. The carrier was advanced and set in place while the form was still enclosed in the previous pour's concrete. The tight encasement prevented any movement with loads being distributed into the concrete.

Figure 5. Tunnel form re-assembly in bottom of intake excavation

Adjustments had to be made after the first two pours to account for the rear of the form and carrier advancing into the poured liner. After the third pour a cycle was defined, although not close to the 24 hour target. A cycle consisted of fixing the form in place, building the bulkhead, pouring concrete, curing the concrete, striping the bulkhead, advancing the carrier, advancing the form, cleaning, oiling, and aligning the form for the next pour. After 10 pours the overall cycle was typically 48 hours (4 shifts). Of the 48 hour cycle, concrete placement took only 7 hours. The following shift stripped the bulkhead and moved the carrier forward. On the second day the form was advanced and reset. Night shift then started a new bulkhead and set up for the next pour. By the 15th pour the cycle had been reduced to 36 hours.

Several concrete mixes were submitted for approval to allow flexibility based on weather conditions and temperatures. Because placement happened during the spring and summer months a mix design with the highest water/cement ratio (0.42) and highest fly ash content (22%) was used. RFK determined this mix could be accelerated as needed to keep up with the pour schedule. Tight controls were necessary to ensure the concrete would perform as designed. Stripping strength of 4 MPa had to be reachable within 12 hours. Accelerator (typically 181ml/100kg cementitious) was added on site no more than 30 minutes prior to start of discharge. A minimum 180 mm slump was needed because of the limited access behind the form to place and vibrate. At 220 mm slump the concrete mix was too wet and began to segregate. The steep grade and use of high slump concrete helped to ensure all surface irregularities from the blasted rock were filled.

A Maturity Meter was used to monitor temperature production and rate of hydration of the placed concrete to allow stripping as soon as the concrete reached strength. Trial batches defined the rate of compressive strength gain to the Time Temperature Factor (TTF) output of the meter. When the TTF read 350 the bulkhead could be safely stripped (2 MPa), as soon as it read 450 (4 MPa) the concrete was self-supporting and the steel form could be advanced.

RESULTS

Design of the tunnel form was based on the goal of placing one section of the tunnel liner every 24 hours on a schedule of three shifts/day eight hours/shift. Actual durations resulted in an average of one placement every two days on the first tunnel (25 pours in 49 work days) and one placement every 1.5 days (23 pours in 37 work days) in the second tunnel. Actual production was based on 2 shifts working 7:00 PM to 5:00 PM and 6:00 PM to 4:00 AM. Timing of the shifts allowed for a hot change from day to evening shift when required, but hot changes were not possible from evening to day shift. Pours which could not be started by the beginning of night shift were pushed back to the start of the next day shift.

Throughout both tunnels the pumping time averaged a stable seven hours, the variables of the cycle were the time spent curing, breaking down, moving forward and resetting the form. After initial start-up kinks were worked out the first tunnel typically required a minimum of 2 full shifts to move and set up the form. If a pour started on Monday morning, the form could be set up to pour again Tuesday evening. Difficulties in securing concrete for evening pours, necessary conservative scheduling of start times, shift limitations and the learning curve extended the average cycle to a full two days. In the second tunnel, experience led to faster cycle times and tighter scheduling allowing pours as often as every 26 hours. Shift and material availability limitations as well as the continued learning curve pushed the average cycle to 36 hours.

An unexpected challenge of the concrete liner was excessive bug hole formation which required extra work to meet the finish requirement. In common concrete work a flat slab is placed with no upper form allowing entrapped air bubbles to escape into free air at the surface. A troweled finish would meet the finish specification with little or no follow up work. Vertical walls are vibrated in place allowing air bubbles to escape out the top of the pour, typically leaving only a small number of bug holes in the vertical surface depending on the concrete mix and placement practices. The exposed face of

Figure 6. Finished concrete liner

a vertical wall should require limited effort to fill bug holes and patch rock pockets in order to meet a Class II finish.

The concrete placed below spring line in this project was cast against the broad surface of the form. The shallow curve of the large radius did not allow air bubbles to slide along the steel and escape into free air. The number of bug holes experienced required a rubbed finish to be installed below spring line in the whole of both tunnels. A stiff sand and cement grout was troweled over the concrete, allowed to take initial set and "rubbed" to create a uniform flat surface. The number of man hours spent finishing the concrete were comparable to the number spent placing it.

Efforts to improve the finished concrete surface included monitoring and fine tuning placement procedures, testing various form release agents, and minor adjustments to the mix design. Variables associated with placement include injection ports, which ones and how often they are switched; and vibrator use sequence and duration. Several form release agents were tested to see if a water-based product could be found to further reduce bug holes. Oil based form release agents were not used as any residue would create a slip hazard on the tunnel slope. Adjustments to the concrete mix design primarily included minor changes to the slump, entrained air content, and various workability admixtures of the concrete. The fine tuning efforts had little effect on the rate of bug hole formation but did result in more efficient placing procedures and consistent concrete mix. The finish was also more consistent, though it still did not meet the finish requirements.

Regardless of the highlighted struggles, the methodology and use of the full round form proved to be an effective choice for this project. The challenging slope was managed and a round concrete surface meeting the structural, shape, and alignment requirements was produced. Very little remedial finish work beyond the patching of bug holes was required in any area. (See Figure 6. Finished concrete liner.)

PART 11

Major Projects

Chairs

Zhenqi Cai
Hatch Mott MacDonald

Marco Giorelli
Aecon Constructors

LARGE-DIAMETER DEEP SHAFTS FOR THE DEEP ROCK TUNNEL CONNECTOR PROJECT, INDIANAPOLIS, INDIANA

Stuart Lipofsky ▪ J.F. Shea Construction Inc.

Percy Townsend ▪ J.F. Shea Construction Inc.

ABSTRACT

The Deep Rock Tunnel Connector Project constructed by S-K JV, a joint venture between J.F. Shea Construction and Kiewit Infrastructure Co., is a $179 million project. The project is being constructed to address combined sewer overflows from outfalls along the White River. The contract includes a tunnel drive of 12,678 meters (41,593 lf), tunnel lining of 12,678 meters (41,593 lf) and two access shafts 82.6 meter (271 ft.) and 66.4 meter (218 ft.) in depth and both 10.6 meters (35 ft.) in diameter. The geology in the shafts consists of an average 30.5 meter (100 ft.) deposit of glacial sediment overburden then New Albany Shale and a bedrock combination of Limestone and Dolomite. This paper will present a general overview of the entire project and the construction methods for excavating two deep shafts, including slurry wall construction, drill and blast excavation methods, muck handling and shaft final concrete lining.

INTRODUCTION

Citizens Water Authority, located in the city of Indianapolis, Indiana, has undertaken an extensive project called the Indianapolis Tunnel Storage System to reduce raw sewage overflows into the Indianapolis waterways. The tunnel system will extend approximately 40.24 km (25 miles) at 60.96 m (200 ft.) below the surface, storing 250 million gallons of sewage during and after wet weather. This will allow the existing wastewater treatment plant to treat the wastewater as capacity becomes available. The $1.6 billion Long Term Control Plan is required to be completed by 2025 under the consent decree with the U.S. EPA and the Indiana Department of Environmental Management (IDEM). The Deep Rock Tunnel Connector Project will be the first phase of the overall tunnel storage and transport system, consisting of approximately 8 miles of tunnel. Project No. CS-038-010C, Deep Rock tunnel Connector Project, was officially awarded to S-K JV for $179,323,117 on November 17, 2011 and the Notice to Proceed on December 16, 2011. The project will be completed and in operation by later 2017.

GEOLOGY

The Launch Shaft is approximately 73.15 m (240 ft.) below existing grade wand will be excavated in two major stages, through soil overburden and through bedrock. The soil overburden thickness is approximately 28.35 m (93 ft.) and consists primarily of fill, sand and gravel outwash deposits with few, discontinuous glacial till layers. It was also note that sub-rounded cobbles and boulders may be encountered in the soil overburden. Since the final location of the shaft was moved after all the geotechnical data was obtained, three additional borings were taken prior to any excavation to confirm the elevation of the rock interface or shale layer. One boring was taken to full tunnel depth for verification of the shale and limestone rock, such that there would be on complete geotechnical boring at the launch shaft.

The top layer or fill material consists of miscellaneous materials, including topsoil, silty sand, gravel, and construction debris with a varied thickness of as much as 0.61 m (2 ft.). The outwash deposits thickness is approximately 27.74 m (91 ft.) and consists mostly of sand with silt and gravel, sandy silt, silty sand, silty gravel and sandy gravel with relatively thin till layers. The outwash sands and gravels are classified as medium dense to extremely dense. The glacial till layers occurring in the outwash deposits are mostly discontinuous low permeability gray silty clay or clayey silt that is very stiff to hard.

At approximately 28.35 m (93 ft.) below existing grade, a black New Albany shale layer with gray layers with an overall thickness of approximately 19.51 m (64 ft.). Most of the shale encountered was soft rock and had a low compressive strength at the top of the shale layer gradually increasing in strength with depth. The shale distinctively changed to a light gray limestone until about 47.85 m (157 ft.) below existing grade and the shaft continued to encounter limestone to the bottom of the shaft.

GROUND WATER INFLOW

From monitoring wells, the ground water at the Launch Shaft is indicated to be just below the surface of the existing grade. As we excavated different trenches throughout the site, we observed that the ground water table varied from a depth of 1.22 m (4 ft.) to 1.83 m (6 ft.) below existing grade, which is the depth at top of the clay layer. After disturbing the clay layer, an inrush of water would pour into the trench. Without a proper designed dewatering plan, it is impossible for a small horsepower single pump to keep up with the water inflow. For excavation of the Launch Shaft, a baseline water inflow quantity was not established from the geotechnical data except that the shaft would be excavated beneath the existing ground water table. The Geological Baseline Report did indicate that with a thick shale layer, there would be a very weak hydraulic connection between the outwash and bedrock aquifers.

SLURRY WALL

The Launch and Retrieval Shaft slurry walls were constructed using the same equipment and methods. The slurry wall depth at the Launch Shaft was 35.05 m (115 ft.) and the Retrieval Shaft depth was 22.86 m (75 ft.). The construction was performed by our Subcontractor, Bencor. Construction started with the installation of a 13.41 m (44 ft.) diameter slurry guide wall system. A 1.5 m (5 ft.) wide trench was excavated 1.2 m (4 ft.) deep. Styrofoam was cut 0.91 m (3 ft.) wide and placed in the center of the excavation. Concrete was placed around the Styrofoam to complete the 0.30 m (1 ft.) thick guide walls. A 0.30 m (1 ft.) thick concrete slab was poured in the center of the shaft to support the crane during slurry wall excavation. The styrofoam was removed as each panel was excavated.

The excavation was performed by a combination of a Leibherr HS 855 crane with a clamshell and Bauer BC 40 Hydromill cutter suspended on a Senebogen 6100 carrier as shown in Figure 1. A bentonite slurry plant was setup on the surface for mixing and 3 ponds were excavated for bentonite storage. A Sotres D450 desanding plant capable of desanding 450 cm/hour of slurry was installed near the storage ponds.

The work consisted of excavating 20 rectangular "bites" approximately 3.14 m (10.33 ft.) long by 0.91 m (3 ft.) wide to a depth of 30.48 m (100 ft.). The resulting structure extended 2.6 m (8.5 ft.) into bedrock and provided a support of excavation which prevented water flows into the area. The bites were split into 5 Primary Panels with 3 bites each and 5 Closing Panels with 1 bite.

The construction process for each bite began by using an excavator to remove the top 3.0 m (10 ft.) of soil. This step could be completed in about 1 hour. The Hydromill excavated 24.9 m (82 ft.) through silt, sand, and gravel to the top of bedrock contact. The speed of this process depended on the presence of boulders in the area. If boulders were present, the Hydromill would either slowly chip away at them and remove the spoils just like the rest of the overburden, or the boulder would get caught in the teeth of the machine and stop its rotation. If this happened, the operator would raise the machine to the surface and dump the rock. If no boulders were present, this excavation could proceed at up to 10.7 m (35 ft.) per hour. However, the average rate of progress was closer to 6.1 m (20 ft.) per hour due to these obstructions, Hydromill maintenance, issues with the slurry system, etc. The Hydromill excavated through 2.4 m (8 ft.) of shale bedrock at an average rate of 2.1 m (7 ft.) per hour. This meant a typical bite could be completed in 1.8 m (6 ft.) per hours start to finish. It should be noted that this value was volatile and a single bite could take multiple days to complete.

If a Primary Panel (Figure 2) was being excavated, all 3 bites would be completed simultaneously before moving to the next step. This involved "cleaning" the trench by running the Hydromill along the sides and bottom of the excavation to remove any intrusive material that still remained. A Koden monitor was lowered into the panel to check the verticality of the excavation walls and to detect the presence of over excavation or soil collapse. If this test revealed an acceptable area, then a rebar cage was lowered into the slurry mix with bracing on all 4 sides to guide it into place and provide clear cover. The rebar cage consisted of horizontal #6 rebar at 30.48 cm (12 in.) E.F. and vertical #7 rebar at 30.48 cm (12 in.) E.F. This rebar cage was fitted with 30 schedule 80 pvc pipes that extended to the bottom of the wall to allow toe grouting before the shaft excavation. Finally, tremie pipes were lowered to the bottom of the cut and 34.5 mpa (5000 psi)

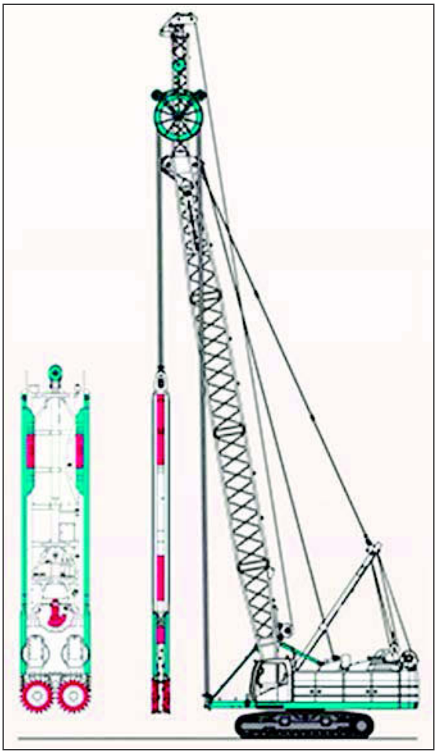

Figure 1.

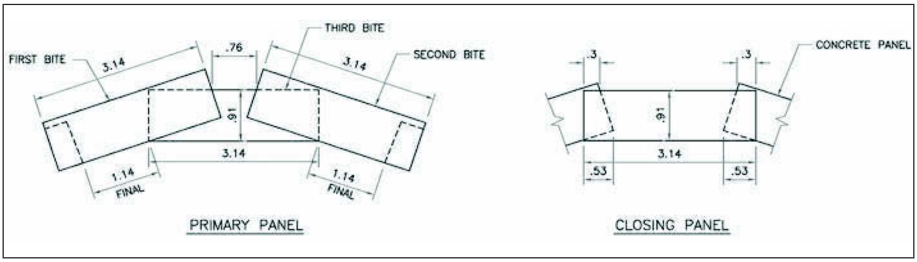

Figure 2.

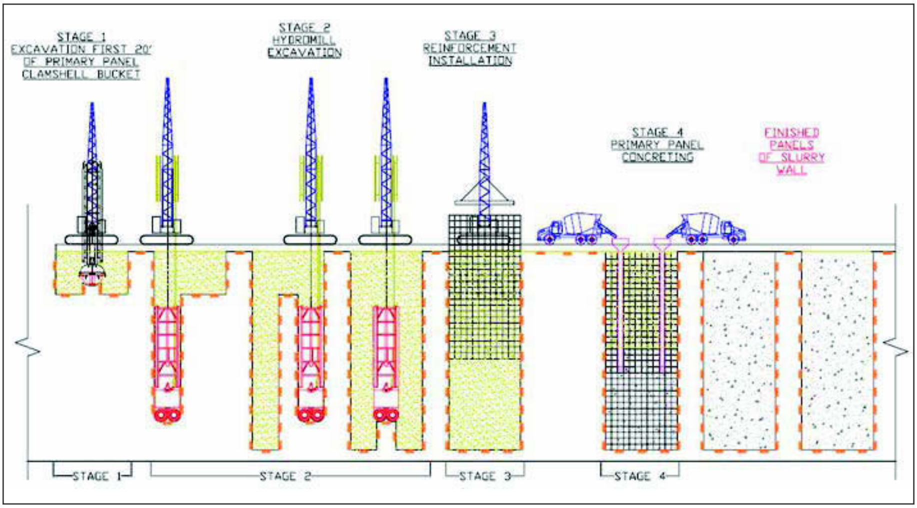

Figure 3.

concrete was pumped down as shown in Figure 3, displacing the slurry. This slurry was pumped to holding ponds and stored for later use.

When the slurry wall concrete cured, the grout pipes were categorized into three types; primary, secondary, and tertiary. Our subcontractor Layne Christensen drilled through the pipes with a ECM 590 drill 4.6m (15 ft.) down past the soil to rock interface. The grout holes were spaced 1.52 m (5 ft.) apart. Each hole was water tested then grouted. Most holes immediately refused grout while some accepted very low quantities.

Figure 4.

LAUNCH AND RETRIEVAL UPPER SHAFT LINING AND BACKFILL

The Launch Shaft upper wall was redesigned by S-K JV to a 50.8 cm (20 in.) wide decagon shape wall in lieu of a 91.4 cm (36 in.) wide circular wall. The wall height was modified from 5.49 m (18 ft.) to 3.35 m (11 ft.). The original design showed the Launch Shaft upper wall extending 2.14 m (7 ft.) below existing ground elevation to the top of the slurry wall. We extended the slurry wall up to existing grade and poured the Launch Shaft upper wall to final elevation. The upper wall was changed from a circular shaft to a decagon to match the shape of the slurry wall. It was formed with 3.35 m (11 ft.) Symons gang forms and included 4 blockouts for a future shaft cover and 2 blockouts for vents. Shaft rebar consisted of double mat #7 vertical rebar doweled into the slurry wall and #6 horizontal rebar at 0.30 m (1 ft.) on center. The shaft wall was poured in a single lift with 34.5 mpa (5,000 psi) concrete as shown in Figure 4.

The contract included a milestone which required the construction of a working platform around the Launch Shaft completed up to the 100 year flood level prior to the installation of the Tunnel Boring Machine. The platform was to be constructed of structural backfill which consisted of No. 8 or No. 9 crushed limestone complying with the Indiana Department of Transportation Standard. We utilize excavated sand and gravel material from the Levee Construction adjacent to the Launch Shaft. The material was placed in 30.5 cm (12 in.) lifts up to El. 670 as shown in Figure 5.

The Retrieval Shaft upper wall was completed per the contract design. We excavated 2.74 m (9 ft.) below existing grade to expose the top of slurry wall. The top 0.30 m (1ft.) of the slurry wall had to be chipped and cleaned to remove concrete that was contaminated by bentonite. A 10.16 cm (4 in.) concrete leveling slab was poured around the slurry wall to provide a level area for setting the upper wall concrete forms. The upper wall was formed using a 3.05 m (10 ft.) high 6.71 m (22 ft.) radius Symons form. Wooden blockouts were installed to form the 0.76 m (2.5 ft.) thick CIP concrete collar corbel which will support the W18×175 steel beams for the precast shaft cover. Shaft rebar consisted of double mat #7 vertical rebar doweled into the slurry wall and #6 horizontal rebar at 0.30 m (1 ft.) on center. The shaft wall was poured in a single lift with 34.5 mpa (5,000 psi) concrete. Sand and gravel material was backfilled around the upper wall in 30.5 cm (12 in.) lifts up to existing grade.

Figure 5.

Figure 6.

OVERBURDEN EXCAVATION AND RING BEAM

Prior to excavation, four holes were drilled 42.67 m (140 ft.) deep 10.16 cm. (4 in.) diameter around the perimeter of the Launch Shaft and pumped with grout to seal fractures in the shale layer. Overburden excavation consisted of 28.0 m (92 ft.) to 28.9 m (95 ft.) of sandy soils and gravel. The material was excavated into a 13.00 cubic meter (17 cu.yd) muck box with a CAT 321 excavator and hoisted with a Liebherr 895 crane. At 29 m (93.12 ft.) weathered shale was encountered. We excavated 1.5 m (5 ft.) into competent rock, approximately 31.7 m (104 ft.) below site grade and poured a 0.15 m (6 in.) mud slab for the placement of the ring beam forms. The ring beam reinforcement consisted of double mat #7 vertical rebar and #6 horizontal rebar at 0.30 m (1 ft.) on center. The ring beam reinforcement was color coded, assembled in sections, and lowered into the shaft. The 11.89 meter (39 ft) diameter, 1.22 meter (4 ft) tall form was assembled and braced. The 34.5 mpa (5000 psi) 3/8 concrete was placed with a 4.59 cubic meter (6 cy) bucket to a thickness of 0.76 meters (2.5 ft) locking in the bottom of the slurry wall as shown in Figure 6.

DRILL AND SHOOT ROCK EXCAVATION

A 30.48 cm (12 in.) diameter burn hole was drilled from the soil to rock transition down 42.67 m (140 ft.) to tunnel invert prior to drill and blasting. The Launch Shaft rock was excavated using conventional drill and blast techniques. S-K JV utilized Dyno Nobel as a blasting consultant to assist with the design of shoot plans. The shaft was excavated to 12.2 m (40 ft.) in diameter. The work cycle consisted of first drilling 63.5 mm (2.5 in.) diameter holes by 3.7 m (12 ft.) deep in six circular rings, using a 3-boom shaft jumbo. Smooth wall blasting techniques were utilized during the shaft exaction. The plan consisted of approximately 112 production holes being loaded with 7 cartridges of Dyno AP Plus 51 mm (2 in.) × 406 mm (16 in.) long and approximately 60 perimeter holes loaded with 2.4 m (8 ft.) of 25 mm (1 in.) × 366 cm (144 in.) Dyno Split AP. Blasting was performed using 25 and 42 Nonel EZ Drifter millisecond delays. The delay sequence was timed so only one hole per delay in the production holes and two holes per delay in the perimeter holes would be initiated. By shooting one production hole or two perimeter holes per delay, the vibration was kept to a minimum to the nearby existing structures. The specified vibration limitations are 12.7 mm (0.5 in.) per second in the ground for all nearby residential buildings and not to exceed 76.2 mm (3 in.) per second at any buried pipe or utility lines. The Launch Shaft is located 164.9 m (541 ft.) from an existing Enterprise Products Company (EPCO) high pressure pipeline. Blasting was conditionally allowed next to the EPCO line. EPCO was required to perform a review of the condition of their pipeline, review and approve the Launch Shaft blast plan. In addition, two seismographs were placed along the pipeline directly adjacent to the launch shaft location and all reports from each shot were transmitted to EPCO for their review.

Once the holes were all loaded and tied in, a steel blast cover was installed over the top of the shaft to contain the blast rock and to keep the air-over pressure below the 130 dB requirement of the specifications. The shot was then detonated and the drill and blast operation continued until reaching the crown of the Starter Tunnel at El. 452. The shaft lining was poured back up to the ring beam at the soil to rock interface. Blasting resumed until reaching the invert of the Starter Tunnel at El. 428.

LAUNCH SHAFT CONCRETE LINING

The Launch Shaft concrete lining began at El. 452. just above the crown of the Starter Tunnel and extended up past the ring beam at El. 575. A steel Everest shaft form having a diameter of 10.66 m (35 ft.) was used to form the pour. The lining was completed in 8 each 4.27 m (14 ft.) lifts and short 3.35 m (11 ft.) lift. We poured concrete and tied rebar on the day shift, tied and placed rebar on swing shift, broke/set forms and installed embedded items on the graveyard shift.

The concrete was a 34.5 mpa (5000 psi) 3/4 aggregate pump mix. It needed to reach 6.89 mpa (1000 psi) before breaking the forms and 13.79 mpa (2000 psi) before setting the forms on the inserts. To accomplish this we added 49.21 liters (13 gallons), 2% of accelerator, to each truck. After 8 hours the cylinders were picked up and broken at 9 hours. A double mat of #6 reinforcing bars at 0.31 meter (1 ft.) centers was assembled in sections of four using a template on the surface. Six coil loop inserts were imbedded into the top of the pour threaded with coil rods.

The 4.27 meter (14 ft.) tall form was held by the work deck with a total weight of approximately 36,000.00 kg (80,000 lbs.). Lasers were mounted at the top of the shaft to ensure verticality and a rotating laser was used on the bottom of the forms to verify plumpness. The first pour was 153 cubic meters (200 cy) placed by a 3.82 cubic meter (5 cy) bucket, lowered into the shaft and poured into hoppers and chutes behind the forms. The slickline and Reed C3050 concrete pump were setup for subsequent pours. Slickline with a swivel coupler in the center of the form work was used to place the concrete along the circumference as shown in Figure 7.


Figure 7.


Figure 8.

MUCKING, ROCKBOLTS, AND SHOTCRETE

The Launch Shaft overburden was excavated using a Caterpillar 321 Excavator and muck boxes. A Leibherr 895 was used to hoist the boxes to the surfaces and dump with a muck dump wall. The face of the slurry wall was cleaned with a pressure washer as shaft excavation continued. All areas along the face of the slurry wall that were not within ±1% verticality were removed with a hammer attachment on the Caterpiller 321 excavator. Once the overburden was removed and the ring beam was poured, the drill and blast operation began. The same equipment was used to muck the overburden and the drill and blast rounds. Shaft support in the rock consisted of FS-46 Friction Stabilizers 2.4 m (8 ft.) long on a 1.83 m (6 ft.) × 1.83 m (6 ft.) pattern. The 3-boom shaft jumbo (Figure 8) was also used to install the shaft support bolts before drilling the next blast cycle.

A 10.16 cm (4 in.) layer of 41.4 mpa 6000 psi fiber reinforced shotcrete was applied to the shaft walls using a Reed C50 SS pump and Meyco Oruga shotcrete robot to a depth of 73.4 m (241 ft). A Moyno pump was connected the Reed pump through a PLC for accelerator dosing to the robot nozzle. A coring and curing room was constructed on the surface to protect the shotcrete samples from extreme temperatures and mishandling.

CONCLUSION

The construction of any large diameter shaft always presents unique and difficult challenges. Choosing the correct means and methods for excavating through soft overburden soils as well as hard rock is crucial to a successful completion of any shaft. Specialized equipment for each step of the process is required to perform the work productively and, of course, safely.

With any slurry wall construction, there is always a concern of making a proper watertight or almost watertight seal at the slurry wall panel joints. Each panel must be excavated plumb and true. Verification by a Koden survey system or similar is highly recommended. As the excavation progressed, it was evident that the slurry wall was installed correctly as very little water seepage was observed. In addition, the bottom of the slurry wall must be set into the rock interface sufficiently. With the toe grouting taking almost no quantities of grout, it verified that the bottom of the slurry wall was successfully socketed into rock providing a necessary watertight seal, especially where the water pressure is the highest.

Blasting in such close proximity to a high pressurized gas line should try to be avoided whenever possible. The blast plans were designed to ensure the protection of the gas line with extreme caution to the conservative side. To minimize the peak particle velocity, which correlates directly to vibration, each plan was designed to use very few holes per delay along with a very detailed delay sequence to avoid cutoffs. With actual low peak particle velocity readings from seismographs, measured at the nearby high pressure line, the existing water treatment plant and the residential neighborhood, the blasting and excavation of this shaft was very successful.

Concreting a shaft by the pour up method is a common practice in today's construction techniques. Even so, the danger of moving and setting a heavy steel form hanging from the crane must be performed with engineered planning and caution. Pumping concrete into place with a unique pumping delivery system was very detailed. To ensure success, testing for initial concrete strengths to set the shaft form to pour every day had to be executed at late hours in the night.

Due to very detailed engineered planning and execution, the construction of this large diameter shaft was excavated and concrete lined successfully with very few problems.

CONSTRUCTION PROGRESS ON THE DC CLEAN RIVERS PROJECT

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Greg Colzani ▪ Jacobs Associates

Christopher Allen ▪ DC Water and Sewer Authority

ABSTRACT

On October 12, 2011, with federal, regional, and local officials present—and a headline banner running in lights across the display in Times Square—the US$2.6 billion DC Clean Rivers Project officially broke ground. Construction began on the 7,378 m (24,200 ft) long, 8 m (26.3 ft) diameter Blue Plains Tunnel. Within a year, construction began on a new overflow and diversion structure near RFK Stadium (Division C), a diversion tunnel system on M Street Southeast (Division E), a diversion tunnel beneath Interstate 695 at the 11th Street Bridge Project (Division G), and a diversion tunnel on Tingey Street (Division B). This paper discusses the status of DC Water's construction progress on the DC Clean Rivers Project.

INTRODUCTION

Construction on the District of Columbia's Long Term Control Plan (LTCP) has ramped up significantly over the last two years. The basic design of the program is the conveyance of most overflows to the Blue Plains Advanced Waste Water Treatment Plant via a deep tunnel system. The program is divided into discrete construction contracts, termed "divisions," on the DC Clean Rivers Project. Figure 1 is an overview of the system for Phase 1 of the program and shows the sections currently under construction, and discussed in this paper. These divisions are:

- Division A: The Blue Plains Tunnel. Traylor Bothers is leading a joint venture (JV) with Skanska and Jay Dee to build the first showcase tunnel.
- Division B: The Tingey Street Diversion Sewer. The 1,830 mm diameter (72 in.) tunneling work is being performed by Northeast Remsco Construction in close coordination with a developer, Forest City.
- Division C: Overflow Structure at CSO-019. Ulliman/Schutte is performing this work (no tunneling is involved) near RFK stadium.
- Division E: The M Street Diversion Sewer. Corman Construction is the prime contractor and Bradshaw Construction Corporation is doing the tunneling work.
- Division G: CSO-007 Diversion Sewer. Construction is by the DC Department of Transportation for DC Water, with the tunneling work being completed by Bradshaw Construction Corporation.
- Division H: The Anacostia River Tunnel. The second "big" element of Phase 1. Contract award for this division was pending at the time of writing of this paper.

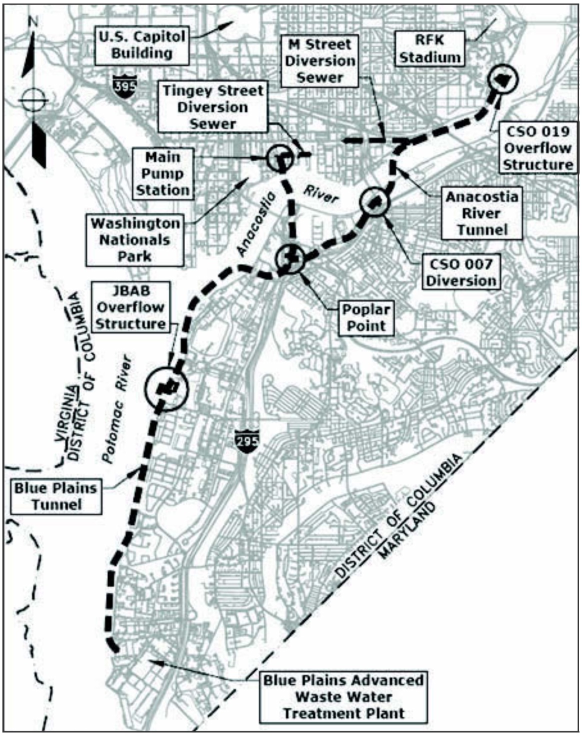

Figure 1. DC Water's Clean River Project active construction divisions

DIVISION A: THE BLUES PLAINS TUNNEL

The Blue Plains Tunnel (BPT) is the first of two "signature" tunnels that are the backbone for control of combined sewer overflows (CSOs) into the Anacostia River. The tunnel is being constructed by the Joint Venture of Traylor Brothers, Skanska, and Jay Dee. Shaft construction at the Blue Plains Advanced Waste Water Treatment Plant was the biggest activity in 2012 for the DC Clean Rivers Project. Steel reinforced slurry walls constructed by Bencor provided excavation support for the shafts. The BPT includes a 7,378 m (24,200 ft) long, precast segment lined tunnel, and five shafts. The 8 m (26.3 ft) diameter tunnel boring machine (TBM) was manufactured by Herrenknecht and was launched in early 2013.

DIVISION B: THE TINGEY STREET DIVERSION SEWER

The Tingey Street Diversion Sewer is being constructed in an area of the District that is currently undergoing significant renovation. Revitalization of the area started over five years ago with the new Nationals Park baseball stadium anchoring the development. The diversion sewer project, managed by a local developer, Forest City Development Company, will convey two CSOs (#13 and #14) to west and the Blue Plains Tunnel (Division A). The tunnel was originally designed to be 1,675 mm in diameter (66 in.), but the tunneling subcontractor, Northeast Remsco Construction, elected to build a finished diameter (1.830 mm [72 in.]) tunnel because of equipment availability. Table 1 summarizes pertinent information regarding the Tingey Street Diversion Sewer.

Tingey Street was once part of the Washington Navy Yard and is in an area that once included a munitions factory and other naval support buildings. Commodore

Table 1. Tingey Street diversion sewer

Tunnel Length: 335 m (1,100 ft)	MTBM: Herrenknecht AVND 1800 AB
Excavated Diameter: 2,260 mm (89 in.)	Machine Type: Slurry
Finish Diameter: 1,830 m (72 in.)	Construction Contractor: Northeast Remsco
Depth (surface to invert): 9.1 m (30 ft)	Design Contractor: CDM Smith

Tingey was commissioned to build the Washington Navy Yard in 1800, and old maps of Tingey Street indicate that some of it was once part of the bay on the north shore of the Anacostia River. Approximately 100 m (330 ft) of the tunnel alignment were wetlands filled in by early builders. The filled area is expected to contain very poorly consolidated soil conditions.

To mitigate these conditions, the contract documents require jet grout support columns beneath the sewer pipe to provide long-term support of the pipe. Additionally, a wide range of unknown manmade materials have been encountered in recent excavations adjacent to the tunnel—including timber, artillery shells, steel debris, concrete blocks, timber piles, and rubble. Fortunately these materials were generally above the tunnel horizon.

The tunnel is also in the vicinity of two brick-lined sewers older than a hundred years. Although the sewers have been rehabilitated and lined with shotcrete in the past few decades, their overall structural integrity was a concern. The contractor is required to stabilize one of the sewers by using permeation grout underneath the sewer in the vicinity of the new tunnel. Another sewer that is near the jacking shaft was analyzed using finite element analysis (FEA) for the effect of potential jacking loads on the sewer before tunneling began. The sewer will be monitored during tunneling.

Because the tunnel alignment passes over the top of the twin Washington Metropolitan Area Transit Authority (WMATA) subway tunnels, it requires a detailed review of the design by WMATA. The subway tunnels were built in the last few decades, and the separation distance from the crown of the subway to the invert of the new tunnel is approximately 13.7 m (45 ft). Ultimately, crossing over the subway is anticipated to have minimal impact on the twin tunnels.

Tingey Street is a rapidly changing corridor that has seen significant construction by Forest City Development Corporation. New apartment buildings, a grocery store, mixed use retail, and office space are all scheduled to open by the end of 2013. To avoid additional third-party conflicts, DC Water contracted with Forest City to manage the construction of the tunnel so that construction of the sewer could be managed by a single entity, thus allowing concurrent sewer construction with other development construction. Figure 2 is a picture of Tingey Street, taken in late 2012.

The DC Clean Rivers Project schedule did not require the Tingey Street diversion sewer to be completed as early as it was. However, the developer, Forest City, had a vested interest in accelerating the work to facilitate the opening of several new facilities in the area. A public-private partnership between DC Water and Forest City Development resulted in award of a guaranteed maximum price (GMP) design-build contract to Forest City.

Northeast Remsco and CDM Smith were subcontracted as the design-build subcontractor for Forest City. The final design work for Division B commenced in the summer of 2012. Forest City established a substantial completion date of October 2013. Jet grouting is scheduled to begin in the winter of 2013, and tunneling is set to occur in May and June 2013 (TBD). The concrete diversion structures and remaining work will be completed in the summer and fall of 2013.

Figure 2. Tingey Street development

Figure 3. Overflow structure at CSO #19

DIVISION C: OVERFLOW STRUCTURE AT CSO #19, RFK STADIUM

A large, three-barrel, CSO trunk sewer handling sewer and storm water from the northeast part of DC is managed by a pump station near RFK Stadium. During rain events, large overflows to the Anacostia River occur through what is known as CSO #19. A large diversion and overflow structure is being built at this location to replace the old system. The CSO #19 location is at the northernmost terminus of Phase 1 of the DC Clean Rivers Project. It is required to be completed by March 25, 2018 in order to comply with Phase 1 of the consent decree. Although Division C included construction of a very large concrete structure, as seen in Figure 3, it does not have an underground construction element, and it is not discussed further in this paper. The Division H Anacostia River Tunnel mining shaft will be located on this site as well.

DIVISION E: THE M STREET DIVERSION SEWER

The M Street Southeast (SE) Diversion Sewer was the third division to begin construction when Corman Construction of Annapolis Junction, MD, was issued a Notice to Proceed from DC Water on March 30, 2012. The M Street SE project is scheduled

Table 2. M Street SE diversion sewer tunnel characteristics

Parameter	9th Street to 12th Street	12th Street to 14th Street
Length:	308 m (1,010 ft)	400 m (1,313 ft)
Excavated Diameter:	1,865 mm (73.5 in.)	12 ft (3.7 m)
Excavation Equipment:	Akkerman Model WM 60-C TBM	Akkerman Backhoe/Shield
Ground Support:	Ribs and Boards	Ribs and Boards
Ground Modification (Grout) Zone:	19 m (63 ft)	53.3 m (175 ft)
Finished Diameter:	1.2 m (4 ft)	2,185 mm (86 in.)
Finished Pipe:	Fiberglass–Resin	Fiberglass–Resin

Figure 4. M Street open cut and Division H shaft location

to finish in the summer of 2014. Corman's tunneling subcontractor is Bradshaw Construction Corporation of Eldersburg, MD. The primary characteristics of the M Street Diversion Sewer are outlined in Table 2.

The project includes two tunnels being excavated through mixed face ground conditions of clay, saturated alluvium, and fill. Ground modification, by injection grouting prior to tunneling, was specified for two different sections of the diversion sewer.

A location directly beneath an overpass of new Interstate 695 at 12th and M streets needed to be stabilized with grout prior to mining. The initial plan was to drill from above and jet grout from the M Street SE roadway. However, Corman Construction and its geotechnical contractor Hayward Baker devised a plan to grout horizontally from the shaft location at CSO #16. Since the tunnel zone requiring grout is only approximately 19 m (63 ft) long, the horizontal grouting concept has a very good chance of success.

The second zone of ground modification was jet grouted from the surface. This zone was about 53.3 m (175 ft).

After completing the tunnel drives, the tunneling contractor will install fiberglass pipe and fill the annulus with grout. The M Street Diversion Tunnel will join with the Anacostia River Tunnel through a drop shaft at M Street SE and 14th Street. Figure 4 shows the first stages of construction at this location. The Division H drop shaft will be situated where the excavator is located in Figure 4. In addition to the tunneling, Corman's scope includes diversion structures at CSOs #15 and #16; an open-cut tunnel from CSO #17 to a drop shaft at the future Anacostia River Tunnel (ART, Division H); installation of a new liner in the old, brick-lined Eastside Interceptor Sewer (ESI); and

Figure 5. Typical access shaft on M Street during ESI rehabilitation work

relining of a portion of the Southeast Water Relief Main, a 0.9 m (36 in.) potable water line beneath M Street. Figure 5 shows an access shaft that was constructed by Corman in the summer of 2012 to enable rehabilitation of the ESI.

DIVISION G: DIVERSION SEWER ACCESS STRUCTURES AND TUNNEL AT CSO #7

The diversion sewer at CSO #7 is located on the south side of the Anacostia River in the immediate vicinity of Interstate 695, where three multiple lane bridges cross the river near the historic Washington Navy Yard. The diversion sewer structures and tunnel were constructed by the DC Department of Transportation (DDOT) in a unique arrangement between DC Water and DDOT. CSO #7 is directly beneath the 11th Street Bridge, and DDOT is performing a large reconstruction effort at that site from 2009 to 2013. The DC Clean Rivers Project and DDOT negotiated a contract requiring DDOT to manage the construction. DDOT issued a change order to Skanska/Facchina, the 11th Street Bridge design-build contractor, to build the structures and tunnel. Facchina Construction of Northern Virginia was the primary subcontractor for the concrete structures, and Bradshaw Construction performed the tunneling and pipe jacking for the 11th Street Bridge Project.

The access structures for tunneling were started in the spring of 2012, and Bradshaw pipe jacked the 1,370 mm (54 in.) reinforced concrete pipe (RCP) over an approximately two-month period during the summer of 2012. Two pipe-jacked reaches were required. Reach one crossed beneath Interstate 695 and was approximately 55 m (180 ft) long. Reach two crossed beneath a local traffic ramp and extended to a future interface location with the Anacostia River Tunnel. Reach two was approximately 67 m (220 ft) long. Although the project was successfully completed in early 2013, the diversion sewer tunnel will not be placed in service until the Anacostia River Tunnel is completed. The consent decree milestone for completion of the Anacostia River Tunnel and Phase 1 is March 25, 2018. Figure 6 shows the diversion structure where CSO #7 is being diverted to the new tunnel as it was being built in the summer of 2012.

DIVISION H: THE ANACOSTIA RIVER TUNNEL

The Anacostia River Tunnel (ART) is the final major tunnel component of the CSO program that must be completed to meet the March 25, 2018 consent decree milestone. Procurement of the Division H contractor commenced in early 2012. The procurement process included a significant qualification process, leading to a shortlist of three firms selected to submit proposals and costs to construct the project. The three teams selected by DC Water to develop detailed proposals for the Division H contract are:

- Impregilo/Healy/Parsons
- Kenny/Shea/Obayashi
- Traylor/Kiewit

The selected design-build teams participated in numerous confidential meetings with DC Water. The goal was to achieve the best product design, at the best price for DC residents, and in the court-mandated time frame available. A stipend will be paid to the two losing firms for their participation in the process.

The 3,810 m (12,500 ft) long tunnel is designed with an inside diameter of 7 m (23 ft). As with the Blue Plains Tunnel, it will be constructed using an earth pressure balance TBM. The tunnel will be driven south from the CSO #19 site near RFK Stadium to the Poplar Point junction shaft, constructed by the Division A contractor (shown in Figure 7). The tunnel alignment is approximately 30 m (100 ft) below the surface and

Figure 6. Diversion structure at CSO #7, Division G

Figure 7. Poplar Point Junction shaft location, Fall 2012

Table 3. DC Clean Rivers Project divisions under construction

Division	Description	Approximate Value
A	Blue Plains Tunnel	$340,000,000
B	Tingey Street Diversion Sewer	$12,000,000
C	Overflow Structure at the Eastside Pumping Station	$25,000,000
E	M Street SE Diversion Structures and Sewer	$30,000,000
G	Diversion Sewer and Tunnel at CSO #7	$5,000,000

crosses beneath the Anacostia River, CSX railroad tracks, new Interstate 695, and the WMATA Green Line. Division H will construct six shafts of varying sizes, four diversion structures, and three connecting adits of varying length. The tunnel drive will finish at the Poplar Point junction shaft, which was completed by the Division A contractor.

CONCLUSION

The DC Clean Rivers Project has officially broken ground, and contractors are busy building Phase 1 of the program. The divisions that started construction in 2011 and 2012 and their approximate contract values are summarized in Table 3. The Consent Decree deadline of March 23, 2018 for completion of Phase 1, as established by the District Court for the District of Columbia in 2005, is now only five years away. DC Water is on track to meet the completion deadline for Phase 1 and is confident that the deadline for Phase 2, March 25, 2025 also will be met.

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FACTORS OF SCALE—PLANNING, DESIGN, AND TBM CONSIDERATIONS FOR LARGE-DIAMETER BORED TUNNELS

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ABSTRACT

Large diameter TBM driven tunnels are becoming ever more frequent options for consideration in achieving owners requirements. This paper will look specifically at the challenges that large diameter tunneling has from three aspects. Planning, Design and TBM requirements. Factors of "Scale" when compared to the more standard TBM projects is the common denominator, yet in each phase of the project this common element presents unique challenges to be overcome. The paper will set out these unique challenges and seek to inform future planning efforts through experiences on some of the world's largest TBM driven tunnels.

INTRODUCTION

Large diameter bored tunnel projects in an urban environment are increasingly being considered in order to create cost effective infrastructure with minimal impact upon the population and the environment. The Asian economies have embraced the available technology and China is leading the way in tunnels greater than 13m (42.65 ft) ID, either planned, under construction or completed. In Europe, the current world leader in terms of diameter is Sparvo in Italy where 13.6m (44.6 ft) ID twin bored tunnel s, accommodates two lanes each is nearing completion. The Russian Orlovsky tunnel when built will lead the race for the largest TBM at 19.25m (63.16 ft) bore diameter.

The accepted design rules for the planning and implementation of these mega projects needs to be looked at and challenged in order to understand if past assumptions and rules are still applicable for the increased diameters. The "Scale Factor," increases as a function of the face cross section area. Increase in diameter, leads to specific requirements for the design of any project with large diameter TBM's. This needs to be fully understood and coordinated through the project life in order to make the project a success. Preliminary design, awareness of planning needs, risk and contract procurement is all affected by the "Scale Effect." Detail design requires an understanding of the TBM's installed logistics and available power. The TBM design and construction must consider the risks associated with ground loss and the need for managing ever increasing volumes of overcut around the machine.

The Scale Effect can be beneficial, with greatly increased internal working space. This helps with safety, gives the ability to more easily install control and monitor system components and allow design innovation utilizing the extra space. Downsides can be potential increased impact on surface, increased support logistics, both in tunnel and further down the supply chain and fabrication and transport challenges (Figure 1).

PLANNING CONSIDERATIONS

Right of Way Considerations

When planning for urban infrastructure the ever increasing cost of surface Right of Way plays a significant part in the design and ultimate configuration of the tunnel alignment. Right of Way procurement differs widely from country to country with some legal systems requiring tunnels to remain within existing city or public right of way and others allowing the purchase of Right of Way beneath buildings. A single bored tunnel has significantly less ROW cost over an equivalent scheme with twin bores. As a consequence the single large diameter bored tunnel for a highway or rail tunnel can have significant benefits to an owner. Elimination of the need for cross passages between twin bores, at approximately 600 ft spacing for a highway tunnel, can further reduces the financial burden. A double deck highway tunnel also optimizes the construction cost verses twin bore tunnels of the same length. As an example, a double deck highway tunnel within a single bore can reduce the surface Right of Way costs by up to 50%. The elimination of complex and sometimes risky cross passages further reduces the project risk. Construction costs on a like-for-like comparison of a twin bore versus a single bore four lane highway of length of 2 miles has been shown to be in the region of $600m.

The SMART project, Storm-water Management and Road Tunnel in Kuala Lumpar, Malaysia successfully incorporated both twin deck highway and a flood relief tunnel within the structure efficiently minimizing the Right of Way cost and the future impact from tunneling. The Alaskan Way Viaduct (AWV) project in Seattle, had several alignments and tunnel configurations considered through its design period, and ultimately opted for a single large bore twin deck roadway with two lanes in each direction over twin bores with cross passages. The commercial benefits of a single bore coupled with the industries acceptance of the technical viability of a world record breaking TBM to create a 52 ft id bored tunnel, led to the present configuration being presented as the base case in the Request for Proposal and being taken forward into construction (Figure 2).

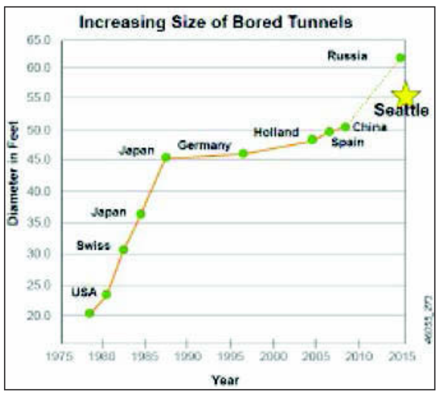

Figure 1. Graph of tunnel size with time

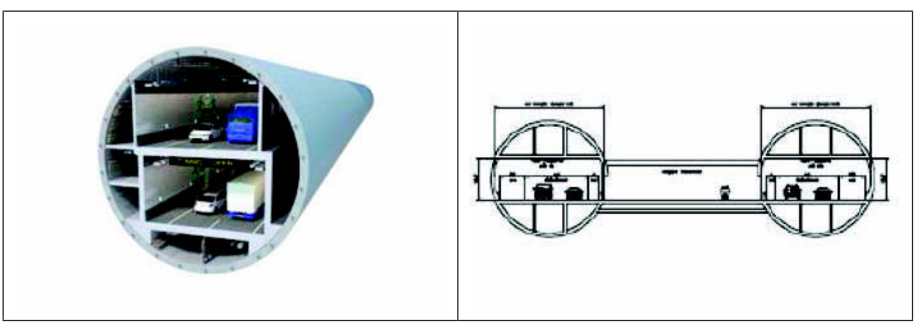

Figure 2. Comparison of single v. twin bore two lane highway (52 ft ID v. twin 32 ft ID)

Table 1. Comparative dimensions of a 4.35m and a 17.48m TBM

Parameter	TBM Diameter		Scale Factor
	Reference TBM	Mega TBM	
Excavated diameter (m)	4.35 (14.3 ft)	17.48 (57.35 ft)	4
Excavated area (m^2)	14.6 (160 ft^2)	240 (2583 ft^2)	16
Shield gap(mm)	19 (in)	15 (0.6 in)	0.8
Shield gap volume (m^3/m)	0.26 (0.1 cy/ft)	0.82 (0.32 cy/ft)	3.2
Tailskin gap (mm)	76 (3.0 in)	205 (8.1. in)	2.7
Grout volume per advance (m^3)	1.2 (0.47 cy/yd)	22.1 (8.75 cy)	18.4

Key Scale Factors

Within an urban environment large diameter tunnels, greater than 15m, are still very much in their infancy. By contrast metro tunnels are commonplace and the impacts of these tunnels on surface, is well understood and techniques are employed to maintain impact to acceptable levels. Owners and Contractors have reached a level of comfort with managing the construction risks of tunnel construction in the 5–8m (16–26 ft) internal diameter range.

In Seattle, the AWV at 17.5m (57.3 ft) bore and 16.46m (54 ft) internal diameter certainly pushes the boundaries in terms of excavated diameter As an urban tunnel crossing beneath piled building foundations and sensitive urban infrastructure, it is also requires an increased level of awareness of the potential to impact surface settlement. The significance of the primary TBM factors that impact ground movement require consideration in the design phase, and is clearly highlighted when a 4.35m (14.3 ft) and 17.5m (57.4 ft) ID tunnel are compared in Table 1.

Management of Ground Movement

Whilst the diameter and excavated area increase in line with the normal mathematical functions, the gap around the shield and the void between the segment and the ground increase by a factor of 0.8 and 2.7 respectively. Similarly the increase in grout volume is 18 times greater than in a smaller TBM. Unless specifically addressed in TBM design through the injection of compensating fluids of bentonite and grout the potential for unacceptably large ground movement exists. This can be controlled through TBM design and must be closely monitored to manage the consequences.

An assessment of ground movement in the planning phase is often carried out using a simple inverted Gaussian curve based upon the work by Boscardin, Cordin and Rankine et al. [1998] In a comparative study of two theoretical tunnels of 4.38m and 17.48m od, with tunnel extrados at the same depth, in the same ground conditions and with a 0.75% face loss assumed for comparative purposes, the criticality of the control of ground movement on large diameter TBM's can be seen (Figure 3).

The increase in TBM diameter up to the diameters currently under construction can theoretically have a significant impact upon surface movement which could be in the region of 10 times that which would be deemed acceptable in a smaller machines operating in the same urban environment. Metro and urban rail tunnels are setting the standard for the magnitude of ground movement which is acceptable in an urban environment and which if exceeded can be controlled though mitigation measures such as compensation grouting or building structural enhancement. A single bore large diameter TBM at an excavated diameter of 17.48m needs to be able to control face loss to a figure of approx 0.04% if it is to yield a surface settlement value which is equivalent to that of a 4m diameter tunnel with the same depth to the tunnel crown. Such a stringent expectation on the TBM operations, the Contractor and ultimately on the Owner can only be achieved through an acceptance through the planning, design, procurement

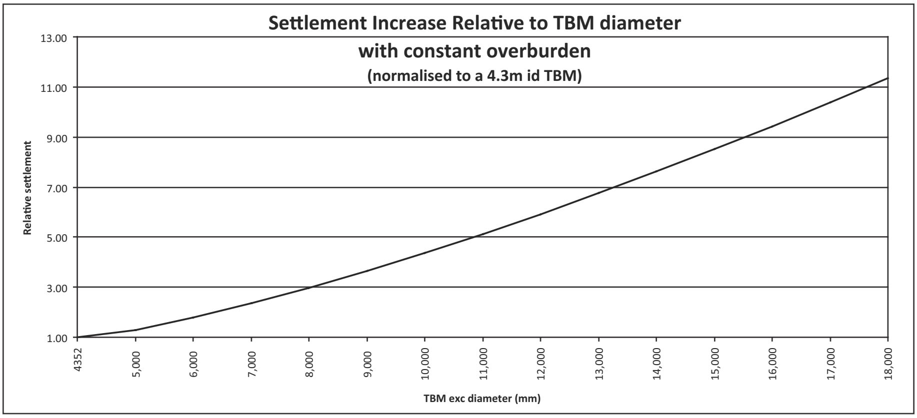

Figure 3. Potential settlement increase with TBM diameter normalized to a 4.3m id TBM

methodology and expertise in construction that the limiting goals of ground movement can be achieved. All parties involved in the project development process need to recognize the magnitude of the task in controlling settlement from the outset and maintain the commitment throughout the project. Establishment of the required culture is important and this can be set up in the early stages of a project.

Large diameter TBM's, as has been shown above, have the potential to have a significant impact upon ground settlement. However, due to the increase size of the machines the possibility of greater control and management of factors influencing face loss is feasible. The scale of the machines allows for the technical requirements to be specific on the needs for such features as:

- In spoke free air cutter replacement—reducing the need to reduce face pressure for a manned intervention, reducing stoppage time and improving safety. Thus reducing the risk of ground loss caused during interventions.
- Multiple TBM body injection points—reducing potential annular loss around the TBM body.
- Multiple tail skin injection points—utilizing A+B grout, rapid set and 100% redundancy of ports to ensure complete filling around the segmental lining at all times.
- Belt scales and volume control—with built in redundancy.

Whilst these items are used and operated by the Contractor, the Owner needs to ensure that such items are fully included within his stated Technical Requirements, not only to ensure that the owners risks are fully addressed but also to ensure that all proposals are made on the same basis. Details of specific TBM features and the impact of scale on these are covered later in this paper.

Risk Management

Risk management and the applicable apportionment of Risk between the parties through contract provision is an important aspect of enabling larger diameter tunnels to be constructed. It is the Owner who has the ability set the Risk sharing model, and an educated owner will do so through discussion with the Contractors who are capable of undertaking such large scale projects. Early contractor Involvement, even if carried out through discussion rather than through any formal contractual process will allow the "appetite" of the industry to be gauged and due consideration given to the key issues and risks that impact upon a contractors perception of the contract and ultimately as to whether he will submit a qualified and competitive proposal. Target Cost contracting which is becoming the norm in Europe, allows the Owner and Contractor to set risk levels, the basis for change to out-turn cost and to limit the Contractors upside financial exposure at an early stage. Target Cost contracts may not be allowed through legislative regulation in all States, but contractual measures can be established to equitably manage Risk through measures such as:

- Owner to cover all or in part the additional cost due to inflation
- Owner to cover premiums for Bonds and insurances
- Shared contingency funds for primary risks, e.g., intervention work and Differing Site Conditions
- Shared contingency fund for the impacts of ground movement on property

Details of an approach to establish an equitable Risk Management approach for large diameter tunnels is set out in the following reference; Contracting for the SR 99 Bored Tunnel, Seattle, Washington RETC 2011.

STRUCTURE DESIGN CONSIDERATIONS

Innovative use of the interior space created by a bored tunnel grows with the tunnel diameter. Except for the design of the connections between the interior structures and the tunnel lining, the design approaches for the interior structures are not drastically different from those used to design above-ground structures (buildings or bridges). As a result, this section will focus on the Factors of Scale affecting the design of the tunnel lining.

For bored tunnels in soft ground, the primary purposes of the segmental tunnel lining are to support the ground, limit the groundwater inflow, and to facilitate the tunneling and follow-on operations. As the tunnel diameter increases, the following Factors of Scale become more influential in the design and analysis of the segmental tunnel lining:

- Mixed face condition: The probability of encountering multiple geologic units within a tunnel cross section increases with the tunnel diameter. Transverse to the tunnel axis, locally high bending moment in the lining can be expected near the transitions between stiffer and softer geologic units. Therefore, the design of large diameter lining must therefore consider the non-uniform pattern of ground load and bedding stiffness that correspond to the spatial distribution of the geologic units. Relevant soil properties are the coefficient of lateral earth pressure (Ko or Ka, depending on the degree of ground disturbance), soil modulus (E or G), and Poisson's ratio for each geologic unit. It is also prudent to consider the variation of these soil properties within individual geologic unit.
- Nonlinear ground response and boundary effects: The extent and magnitude of nonlinear ground response increase with the tunnel diameter. Further, as the tunnel diameter increases, the relative distance between the tunnel extrados to the ground surface and lateral boundaries decreases. The approaches previously used on smaller tunnels for estimating the ground response to tunneling and the interaction between the lining and the ground must be applied with caution to larger tunnels as these conventional approaches generally assume negligible influence from the boundary conditions and, to a lesser implication, linear ground response.
- Flotation, where the groundwater table is high: The self-weight of the lining is generally less than the buoyant force exerted on the submerged tunnel. The difference between the buoyant force and the lining self-weight increases with the tunnel diameter. Sufficient ground cover must be provided to resist flotation. However, there is a practical limit to how deep tunnels could be constructed,

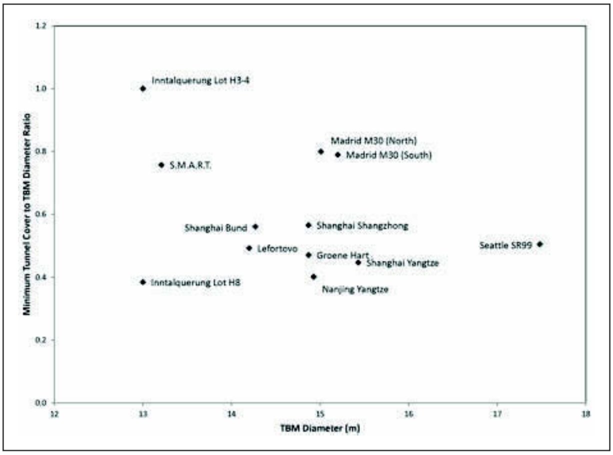

Figure 4. Minimum cover for large diameter bored tunnels

particularly at the portals and shafts. This is due to considerations such as the continuity of the vertical alignment beyond the tunnel portals, cost, complexity of the earth retention system and groundwater control, and safety. Figure 4 shows the minimum cover for bored tunnels with a diameter of 13m or larger. For the preliminary design and planning purposes, the data suggest that a minimum cover of 0.4 to 0.5 times the tunnel diameter should be provided. For traffic tunnels, the flotation resistant requirement strongly influences the portal location and the roadway vertical alignment.

- Large TBM thrust: The TBM thrust increases almost quadratically with the tunnel diameter in order to resist the face pressure and overcome the shield friction. For soft ground pressurized face tunneling, the TBM thrust is reacted against the tunnel lining which must be accommodated through the design and detailing of the lining. Sometimes, but not always, the TBM thrust governs the thickness and concrete strength of the segmental lining. Figure 5 presents the TBM thrusts from recent large diameter bored tunnel projects. Figure 6 presents the same data as Figure 5 but the thrust has been normalized by the face area, an approach used in the Japanese Standard for Shield Tunneling. For the planning and preliminary design purposes, the TBM thrust data appear to suggest 1,300 to 1,600kN/m^2 of for slurry TBMs and 1,800 to 2,100kN/m^2 for EPB TBMs.
- Large TBM torque: The TBM torque increases cubically with the tunnel diameter in order to excavate the ground and overcome the friction between the rotating cutter head and the ground. For smaller tunnels, the TBM torque has not been an important factor in the segmental tunnel lining design as the torque could generally be counteracted by the friction between the shield and the ground. However, for large diameter bored tunnels and with the advent of shield gap injection for the purpose of reducing the ground loss and surface settlement, there is a potential need to react a portion of the total TBM torque against the segmental lining. This creates a new loading scenario that has not traditionally been considered in the tunnel lining design process. Additionally, this consideration could impact the detailing of the circle joints and the joint connectors. More research is required in this area to better understand the need and inform the future design effort. Figure 7 presents the equipped TBM torque from recent large diameter bored tunnel projects.

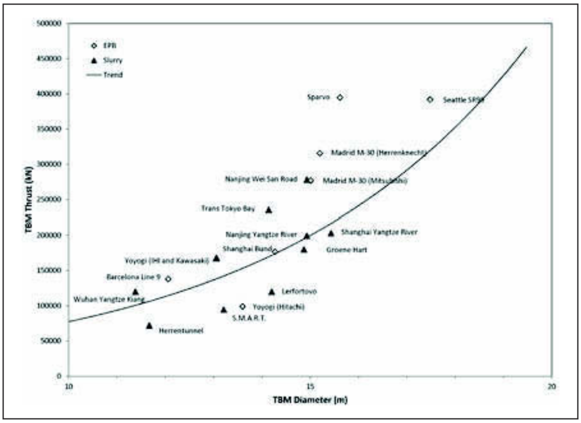

Figure 5. TBM thrust for large diameter bored tunnels

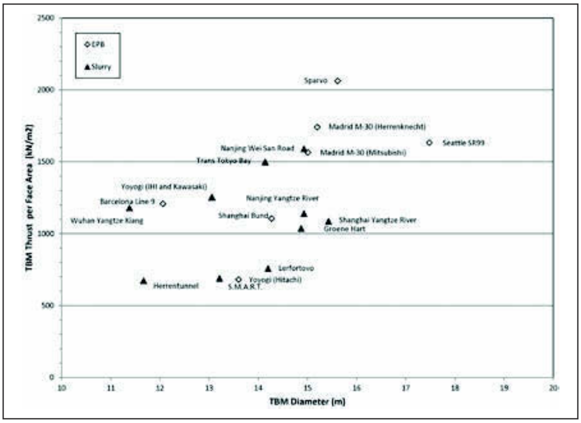

Figure 6. TBM thrust normalized by the face area

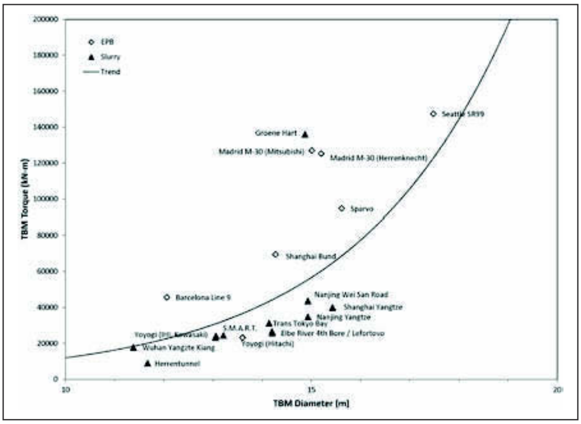

Figure 7. TBM torque for large diameter bored tunnels

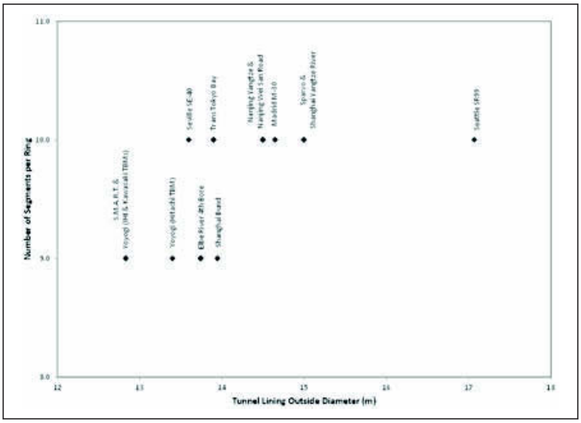

Figure 8. Number of segments per ring for large diameter bored tunnels

- Number of segments per ring: The weight restriction for handling and transporting the segments from the fabrication facility to the TBM portal often dictate the number of segments comprising a full ring. Consequently, the number of segments per ring grows with the tunnel diameter as evident from the recent data in Figure 8.

The simple yet effective approach to estimate the reduced ring stiffness due to the presence of radial joints proposed by Muir Wood (1975) continues to be widely applied to the design of large diameter segmental lining. An emerging trend is to perform limited verification of the results from the Muir Wood's simplified approach with a more sophisticated three-dimensional shell element model. While the aim is to more accurately capture the geometry of the lining segments, the deformation characteristic of the segment connectors and coupling between rings, there has been no wide report of significant changes to the segmental lining design solely as a result of the use of the three-dimensional shell element models as compared to the use of the Muir Wood model.

The loosening pressure approach proposed by Terzaghi more than 60 years ago continues to be popular for estimating the vertical ground loads on relatively deep tunnels. Where the tunnel is relatively shallow and the lining cost increase is insignificant, the trend is to design the lining for the full overburden condition.

It is difficult to single out one tunnel lining analysis and design approach that is preferable across the globe. Rather, engineers continue to adapt the approaches that they have been familiar with and used successfully on smaller tunnels to the design of large diameter segmental lining; for instance, the popularity of the bedded beam approach in Germany and Japan, the closed-form elastic plate with lined hole in the UK, and the use of continuum model (FEM or FDM) in the US and China. An emerging trend is to use multiple analysis and design approaches and envelope the results.

TBM DESIGN CONSIDERATIONS

Scale

"Scale" pushes the limits for TBM design, demanding creative solutions to address increased risks of surface settlement, ground loss, along with other machine design issues. Following is a general review of design issues for a large diameter EPB machines using precast segmental lining at (56 ft. OD × 54 ft. ID × 6.5 ft width).

Growing tunnel sizes present environmental challenges for muck disposal, transport, onsite handling and treatment, cooling, and water discharge facilities. "Scale" pushes the limits of logistical supply lines. Consider a single precast segment liner ring approaches one third of a million pounds. The size and weights of precast segments challenge manufacture, storage, and transport to the site. This places huge demands on crews, handling and special equipment to transport them to the TBM, where unloading and delivering to the segment erector is provided in the TBM design. Considering the time to assemble such massive rings, one might consider Semi-automatic initial positioning, dual segment erectors, using the extra space available, and allowing faster and more efficient assembly of the liner ring.

Of interest, even with this massive segment liner ring weight, is the large diameter buoyancy, nearing 100,000 pounds per foot, pushing upward on the crown of excavated soil. Could this buoyancy help control ground settlement when compared to that in smaller tunnels? It appears statistically that large diameter TBMs have less percentage volume loss or better control of the ground than one would expect.

As diameter goes up everything is magnified: Muck volumes, machine component size and weights, along with transport, handling and assembly issues. Consider electrical power source limitations, size of transformers, substations, and tunnel power cable voltage and size, utility pipes, ventilation ducts and fans. Scale demands enormous horsepower, torque, and thrust. Consider the number of hydraulic pumps, motors, cylinders, and their hydraulic and lube oil distribution systems. There is the need for enormous reservoirs whose capacities exceed that of several tanker trucks. For example, hydraulic oil volumes approach 13,200 gallons. Back-fill grout volume for an 8 inch annulus around the segment surface is about 29 cubic yards for each ring. This 8 inch liner annulus is the result of a 5 inch thick tail shield structure, plus segmental clearance using four row wire brush tail seal system. The shield length of about 64 ft. having just ⅝ths inch annular gap around the shield, demands 21 cubic yards of pressurized slurry for supporting the ground around the TBM until the liner and grout is placed. This gap could account for considerable ground loss or surface settlement if not addressed. Further, this shield annulus must support the soil for up to two or three days, assuming 30 ft/day progress, until segment rings are placed and grouted.

Water Supply and Cooling

Water is needed for cooling electrical drive motors, variable frequency drive systems, planetary gear boxes, the hydraulic and lube oil systems. Efficient use of thousands of gallons of water is a key issue in the design. For example, if 1,400 gpm is provided to the TBM, up to 67% can recycle back to the surface. There it is stored while it cools and then is reused. Some water after cycling through heat exchanges on the TBM is directed for high pressure nozzles on the cutterhead, and to the EPB soil conditioning system. This water is mixed into the muck and exits the tunnel by conveyor for transported to the disposal area.

Control of Face Loss

The primary focus is controlling over excavation or face loss during excavation. The Secondary focus is preventing potential surface settlement caused by ground moving toward the machine for whatever reason. Finally applying mitigation measures to address voids or difficulties as they occur. This requires provisions to detected or discover issues before they develop into issues resulting in settlement at the surface.

Suggested are concepts for specifications requesting machine designers incorporate design features and concepts to address this most difficult of all issues presented as diameters increase. Dealing with large volumes of muck can mask the effects of over excavation and surface ground settlement which may be more easily identified with smaller volumes. Focusing on concepts to minimize and control ground loss and surface settlement is paramount.

The machine design as a minimum should contain many features to address construction and ground control issues. Examples being:

- Provision of both axial and inclined probe drill through numerous ports
- Operational and redundant soil removal monitoring systems
- Automatic face recovery system
- Two component rapid set back fill annulus grout system
- Secondary backfill grouting system
- Two variable speed inline screw conveyors with redundant close-off gates
- Redundant tail seal system with emergency seal
- Redundant soil cutting tooling to reduce interventions
- Shield articulation joint to allow reduced annular gap
- Multiple wear detection systems with real time monitoring
- Design Innovation—Ability to change cutters in "Free Air" from within the cutterhead.

EPB Principles

Understanding the "EPB" principle follows a simple basic concept. That is, to provide bulking fluids at the point of excavation, filling voids between soil particles as soil bulks, while mixing and conditioning soil into a "Soil Plug," which can be metered and prevent uncontrolled flow. It is important to provide bulking fluids; foam, polymers, bentonite, water and other conditioning agents, as close to the face as possible as material is cut from the face and begins to bulk. Then in the mixing chamber as it continues to bulk, and then the screw conveyors as need. Bulking fluid conditioners is supplied as needed during the process of mixing excavated soil into a stable "Soil Plug." This simple EPB principle allows the ability to maintain the correct face and annular pressure to prevent water or soil from flowing toward the TBM. Failure to achieve this basic principle, will allow uncontrolled flow toward the TBM that can hydraulically mine ahead and above or even behind the machine causing ground loss and settlement.

Scale presents several challenges to achieving the "Simple EPB Principle." Large diameter requires increased cutterhead structural spans. To maintain strength and stiffness, the cutterhead spoke's depth and width increases. These factors increase the muck path through the cutterhead, adding potential risks for cutterhead plugging. Applying high pressure water jets may help in this regard.

Peripheral velocity of a cutterhead is limited by several factors. As TBM diameters increase the RPM necessarily decreases. Traditionally smaller TBMs have mixing paddles mounted on the back side of the cutterhead working in conjunction with fixed paddles mounted on the mixing chamber bulkhead. Large diameter TBMs lose mixing ability in the center area because of their lower RPM. This issue is further magnified by the exponentially larger volumes of muck they need to mix. This mixing of huge volumes of soil becomes a major limiting factor on the rate of advance if not addressed. Some machine designers have provided solutions by using powered mixing paddles within the mixing chamber or by adding an independent and separate center cutterhead rotating at a faster RPM. Either method when counter rotating increases mixing efficiency, and provides counter torque to help control roll during a push cycle.

Ground Stability

The wider spokes of large diameter machines has the potential to create pressure pulses within the excavated material that destabilizes the outer areas of face. Intuitively, fluid dynamics shows that the higher the velocity, the greater the turbulence effect on

Table 2. Current versus future TBM sizes

	Current Typical Internal Diameter	Future Potential/Need
Road tunnel (2 lane)	13 (43ft)–14m (46 ft)	
Road tunnel (3 lane)	15m (49 ft)	
Road tunnel (4 lane)		17m+ (55.8 ft+)
Rail tunnel (high speed single)	12m (39 ft)	
Rail tunnel (high speed twin track)	No current precedent	Twin track 15m+ (49 ft+)
Water tunnel	Varies up to 14m (46 ft)	TBM limited… 17m+(55.8 ft+) (?)

soil stability at the periphery of the cut. As diameters increase so too does the percentage of the soil volume coming from the outer portion of the diameter where the effect is greatest. Perhaps some research, focusing on hydro dynamic effects, may allow a better understanding on why and where ground loss is most likely to occur in the face. Such research could lead to improved spoil entry shapes or other considerations in the cutterhead design Lower RPMs or deeper penetration per rev, and more active powered mixing concepts could be considered.

Large Diameters present safety issues relating to fall from height potentially reaching that of that from off a five story building. Consideration of fall protection, overhead shielding, man access, automated material handling, ducted ventilation, emergency egress procedures and fire protection needs to made. Safety issues increase with scale, demanding safety measures such as: Increased fire suppression systems, air and gas monitoring systems, including the use of flame retardant hydraulic fluids and lube oils, flame retardant conveyor belting, hoses, and cabling. The reduction and control of fuel sources for fire present opportunities to reduce fire risks.

CONCLUSION

Large diameter tunnels are becoming increasingly economically viable as infrastructure needs of modern cities increase. Technological advances in TBM manufacturing have allowed Owners to investigate the potential for projects that up to this point in time have not been viable. The larger diameter TBM has allowed alternative solutions to be proposed and if economically viable, considered as the primary contender for new infrastructure. In the road tunnel sector, tunnels in the range of 13m (42.6 ft) to 14m (46 ft) are becoming common place in China and increasingly more so in the USA and Europe. Sparvo in Italy is leading the way for a major two lane highway which was openly tendered. Single bore two lane, limited height roadways such as SMART and the proposed Istanbul straight crossing tunnel are offering traffic solutions that are matched to the local need. The St Petersburg project, approximately (17.25m (56.6 ft) ID) will take the world record after the Alaskan Way Viaduct Replacement tunnel (16.4m (54 ft) ID) is completed. The next step in the increase in Road tunnel diameter will be to the 17m + (55.8 ft) ID range which will allow 4 lanes of traffic or 3 lanes plus an emergency lane. Technically the TBM and civil design challenges are being overcome, however the ability of the industry needs to be matched to the need, risk and commercial aspects of such a ground breaking project.

Rail tunnels have less potential for an increase in diameter with twin track high speed rail tunnels being the largest diameter that can currently be considered, at a size of approx. 15.5m (50.9 ft) ID. Water tunnel diameters, however, are purely limited by design, economic and operational considerations and tunnels larger than the Niagara machine (14.4m (47.2 ft) OD) could conceivably be proved economic for long water transfer tunnels (Table 2).

PLANNING AND DEVELOPMENT OF THE WATERVIEW CONNECTION TUNNEL

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ABSTRACT

The New Zealand Government's Roads of National Significance (RoNS) programme of 7 key projects, involved a commitment to upgrade transport infrastructure, particularly where such projects would promote economic development and efficiency by removing bottle necks and assisting mobility. The largest of these was the completion of the Western Ring Route (WRR) which constitutes the largest and most complex highway project yet undertaken in NZ. The centrepiece of the Western Ring Route is the Waterview Connection tunnels that provide the missing link between SH20 and the existing SH16. The 2.5km long twin 3 lane tunnels to be excavated with the largest Earth Pressure Balance tunnel boring machine in the Southern Hemisphere has been planned for over 15 years, and was originally proposed as a surface motorway with resulting significant property acquisition and community affects. This was changed to a longer 2-lane tunnel with active traffic management before the current 3-lane configuration which facilitated free flow conditions became preferred. The project while still in the planning phase when the Global Financial Crisis hit in early 2009 was fast-tracked as an economic stimulus project. This paper describes the recent history and planning of the project, and the unique challenges presented by the concurrent procurement and planning approval process required to meet a very ambitious procurement timetable.

INTRODUCTION

As a small, sparsely populated country distant from world markets, New Zealand relies on a robust transport network to move people, goods and services safely and efficiently. Around 92 per cent (by weight) of all freight within New Zealand is moved by road. An efficient freight industry is vital to the competitiveness of New Zealand businesses. With less time and money spent transporting goods, more investment can be made in productive assets and increasing wages which continue to fuel economic expansion.

The Government has identified seven state highways that are linked to New Zealand's economic prosperity, and are based around New Zealand's five largest population centres. The focus is on moving people and freight between and within these centres more safely and efficiently. The Roads of National Significance (RoNS) programme represents one of New Zealand's biggest ever infrastructure investments and is key to the country's economic growth. It also represents a major departure from previous road planning by investing to encourage future economic growth rather than waiting until the strain on the network becomes a handbrake on progress.

This paper describes the centrepiece of the largest RoNS project, Waterview Connection which forms part of the Western Ring Route, and in particular the project development phase that saw a surface motorway change to a long tunnel. The

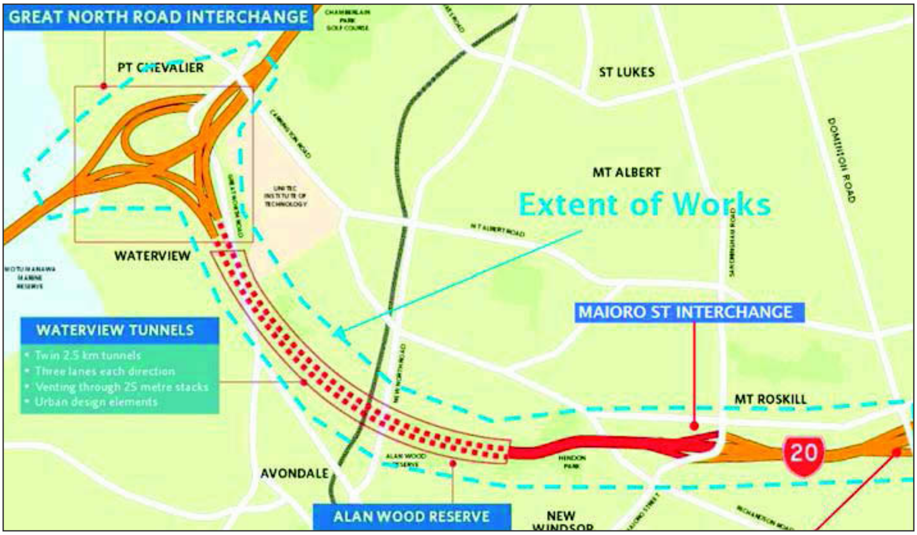

Figure 1. Waterview Connection project scope

planning approval process is also discussed as well as the challenges presented by a concurrent procurement phase.

BACKGROUND

Auckland has grown quickly to its current size of 1.5M people. While cities of this size are unremarkable on the world stage, it is not often that they contain more than a third of the population of an entire country. The resulting limits on public spending have created immense challenges for Auckland to enable its strategic road network to deliver improved economic prosperity and enhance Auckland's reputation as one of the world's most liveable cities.

Auckland's geographic challenge is that it lies on a narrow isthmus, in places only 2km wide, between two large harbours. This squeezes the road network into a constrained north-south pattern which is further limited by a single bridge crossing of the Waitemata Harbour although planning is underway for a second harbour crossing in the next decade.

PROJECT DESCRIPTION

The Waterview Connection ($1.2B) is the missing 5km link between two existing state highways. It will complete SH20 and connect it to SH16 via a complex service level interchange. At the centre of the Waterview Connection are the twin 2.5km driven 3-lane motorway tunnels that pass under an established residential area and a major local arterial road. Refer to Figure 1 for the project layout.

The new motorway interchange at the northern end of the project will create free flow links for all moves between the motorways while also maintaining connectivity for the local interchange.

The tunnels forming the centrepiece of Waterview Connection project consist of three 3.5m lanes and a 200mm offset to barriers to give a 10.9m roadway width. The posted vertical clearance is 4.6m, with 4.9m clearance provided. Cross passages are at 150m centers, and ventilation for smoke control to provide tenable conditions for the designed 50MW fire includes both longitudinal jet fans and axial fans provided at a ventilation stack at each portal. Other active fire systems include a deluge system as is customary in Australasian tunnels. The tunnels will be constructed using a 14.53m OD Earth Pressure Balance tunnel boring machine. The TBM will be the ninth largest in the world and the largest in the southern hemisphere. Excavation will be predominantly through East Coast Bays Formation (ECBF) which occurs throughout the Auckland region and which comprises shallow dipping alternating beds of extremely weak to weak sandstone and siltstone. Within this formation are occasional interbedded lenses of Parnell Grit, a weak to moderately strong sandstone. The Waitemata Group sediments were deposited during Miocene times (around 26 million years ago) during the mid-Tertiary submergence when Auckland was entirely underwater. This sandstone/siltstone was deposited underwater in a shallow basin, and has been uplifted and subjected to faulting since.

Figure 2. Western ring route location

At the northern end the tunnel passes through mixed face conditions including Pleistocene material derived from the ECBF rocks that was deposited in a shallow marine environment to form the firm to stiff clay of the Tauranga Group.

There has been prior experience with EPB tunnelling within Auckland, with 6km of small bore wastewater tunnel construction over the last four years. However there has been nothing approaching the scale of the proposed Waterview tunnels. The major identified risk with the tunnels is the low (9m) cover to the arterial road at the northern end. While it is proposed to mitigate the face stability risk in the soft ground conditions with closed mode EPB operation, it is also proposed to temporarily divert the traffic while excavating the tunnel in order to reduce the consequences of any face control issues.

PROJECT DEVELOPMENT PHASES

Overview

The Waterview connection project will provide the missing link in the Western Ring Route (WRR) and will then provide a 48 kilometre motorway alternative to the congested Auckland central motorway network comprising State Highway 1 (SH1) and the Auckland Harbour Bridge. It will bypass the city to the west and link Manukau, Auckland, Waitakere, and North Shore regional centres via State Highways 20 (the Southwestern Motorway), 16 (the Northwestern Motorway) and 18. Refer to Figure 2 for the Western Ring Route location.

Waterview Connection was first mooted in 1996, with the contract awarded in November 2011. It is forecast for completion in early 2017 meaning it has been a 21 year period from project inception to opening. This is not uncommon for tunnel projects, and like many other contemporary urban road tunnels, it was 10 years before a proposed surface route was investigated as a tunnel. Following confirmation of the tunnel

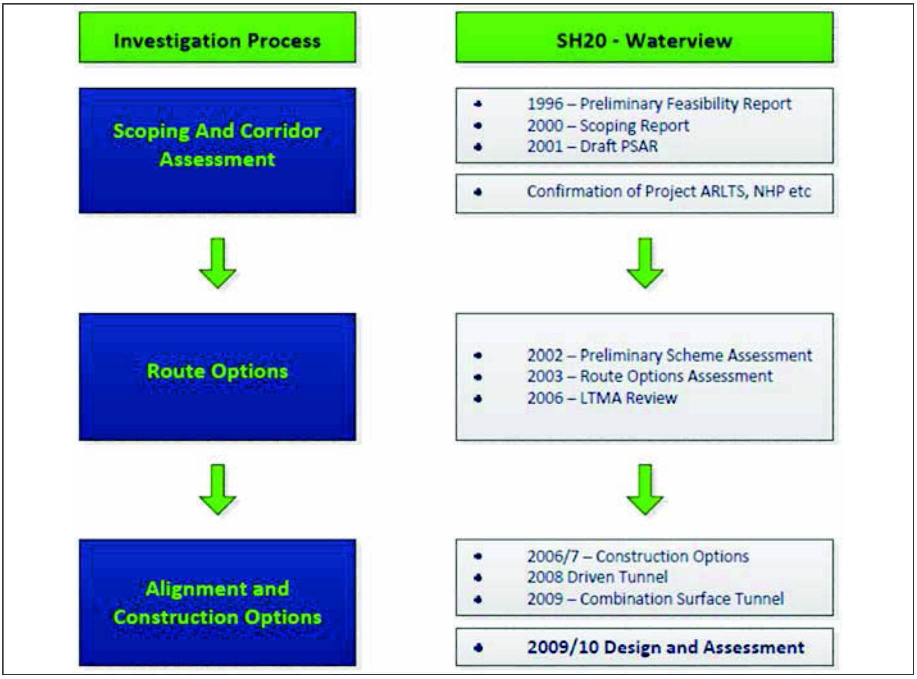

Figure 3. Project development phases and timetable

route, it was only 2 years for planning approval to be obtained and a contract awarded, which is a very short timeframe when compared to other similar projects.

The road corridor was not in place, and therefore the planning process also needed to include the designation of the motorway corridor.

The project development phases and timetable are shown in the Figure 3.

Constraint Mapping

Constraint mapping was undertaken initially, and included desktop analysis, examination of the existing environment, and consultation with the community to identify sites and areas of 'environmental' significance (including social and cultural value). The following data layers were compiled in the constraints analysis:

- **Topography and Geology**—physical constraints and opportunities in the study area, including areas of faults and mapped instability hazards.
- **Land Uses**—property information, zoning and existing land uses and designations.
- **Transport Networks**—roading and other transport modes (both existing and planned).
- **Ecology**—vegetation (terrestrial and aquatic), avifauna, fish and aquatic fauna.
- **Landscape**—identified landscape and visual amenity areas, view shafts and other key visual catchments.
- **Archaeology**—sites and areas of archaeological interest (e.g., sites registered with the New Zealand Historic Places Trust).
- **Population**—population distribution, dwellings and growth projections.

- **Sites of Social Value**—resources, sites, trees, schools and other locations were identified as areas of perceived high value by the community.

This constraints map was used as the basis for generating and assessing the corridor options. It is worth noting that although the local environs included many constraints as is typical for an urban tunnel, there are no constraints that would normally mandate a tunnel, such as steep terrain or a watercourse. Urban tunnels are increasingly required due to a lack of community acceptance of the impacts associated with a surface motorway. This is further discussed below within the section on community consultation.

Corridor Assessment

The corridor assessment included the development of a long list of 20 route options. A phase of stakeholder and community consultation was undertaken, and the long list of options were screened using environmental threshold criteria, which resulted in a reduction to 12 routes over two alternative corridors. A detailed option assessment and technical evaluation was then used to produce a short list of route options over the two corridors as shown on Figure 4.

The shortlist of route options was then subject to a detailed evaluation against the following criteria:

- Relative traffic performance
- Connection flexibility
- Ease of future connections
- Staging

Figure 4. Waterview connection corridor options

- Potential physical environmental impacts
- Potential social environmental impacts

Following this evaluation the AW1 corridor was selected, which then allowed detailed route assessment to commence. At this stage the project did not include a tunnel.

Route Assessment

Following corridor selection the next phase of project development was an assessment phase. An initial alignment option for the AW1 route was developed and released to the public and stakeholders for comment in March 2006. The alignment included potential sections of 'cut and cover' tunnelling through Owairaka and Mt Albert (approximately 1.2km in length), as the alignment crossed the existing rail corridor and several major arterial roads.

Following a review that identified various construction difficulties associated with the cut and cover section of tunnel, in September 2006, 10 years after project inception, the NZTA announced it would also look at different construction options for further undergrounding. The options developed included an option combining open road and cut-cover sections, an option with extended cut and cover tunnelling and a driven tunnel option. These construction options were the subject of technical and environmental assessments between 2006 and 2007.

In 2007 an evaluation framework was developed to consider these options. The framework had regard to the Project objectives, strategic objectives of NZTA, and statutory obligations and practices. The evaluation built on previous option evaluations undertaken on previous phases of the Project.

In total, six evaluation criteria were developed for the evaluation:

- **Cost**—construction cost, property cost, operation cost (whole of life and average annual cost) and incremental net present value;
- **Traffic Effectiveness**—traffic benefits, accident savings, security of transport system, integration with other transport modes, improvements to access and mobility and the contribution to the WRR;
- **Physical Environmental Impacts**—land stability/geotechnical stability, sites/areas of geological interest (e.g., basalt flows); coastal marine area and receiving environment; groundwater; natural habitats and fauna (coastal, terrestrial and streams), coastal processes as they contribute to natural character of coastal environment; landscapes; and contaminated sites;
- **Social Environmental Impacts**—sites of cultural significance, community linkages and connectivity, population impacts/displacement, health and well-being, community services and facilities, recreation and reserve areas, urban amenity and business and economic opportunities;
- **Timeliness**—Project Approval process timeframes and construction programme; and
- **Sustainability**—energy efficiency (includes vehicle energy use & the operating costs), land transport integration (supporting regional growth), future proofing (capacity within the facility and opportunity for change in mode), agglomeration and intensification potential; and opportunities for travel demand management.

The option evaluation process at this stage did not identify a 'preferred option' but rather sought to inform the project decision makers on the relative 'benefits' and 'disbenefits' of the options being considered. The purpose of this process was to assist decision makers in confirming a preferred option, and used as the basis for stakeholder and community consultation.

Consultation Outcome

In February 2008, the NZTA Board confirmed the driven tunnel option as the preferred construction method for the Project and sought community and stakeholder feedback (February to June 2008) on that option. The purpose of this round of consultation was to inform stakeholders and the wider community of the NZTA's draft preferred option, to seek support, opposition, or otherwise for the preferred option, and to receive input on issues and matters considered significant in moving forward with the Project.

Of the 747 public responses received, 76% were in support and 18% were in opposition to the driven tunnel option. In addition there was a petition (with 72 signatures) received from the Waterview Kindergarten Parent Committee in opposition to the tunnel. The remainder of responses (6%) indicated they 'did not mind' or did not express any preference. The key reasons given in support of the driven tunnel included:

- That the NZTA have listened to community concerns;
- The ability to retain open space/protect the Oakley Creek environment;
- The need for fewer homes to be taken;
- The tunnel would be less disruptive to community values;
- Improving traffic congestion in Auckland; and
- Containing the effects (e.g., noise, visual) of the motorway underground.

The key reasons given in opposition to the driven tunnel included:

- Community disruption and effects during construction (especially noise, dust and traffic impacts);
- Impact of loss of houses on Waterview Primary School/Kindergarten and 'sense of community';
- Concern that the dispersion of emissions to air from the ventilation stacks could have health effects on local residents and school (including Waterview Primary School and Kindergarten);
- Cost—that $1.9 to $2.3 billion is excessive for one project and 'the tunnel money' could be better spent on multiple transport projects/public transport, and the cost of operation and maintenance;
- That future fuel price increases make this the wrong investment for New Zealand;
- That too much political influence has informed the Project; and
- Impact on property values.

This feedback neatly summarizes the arguments for and against urban tunnels, with the majority of the community supportive of a tunnel due to the impacts associated with a surface motorway, but with residual concerns regarding air quality in the vicinity of ventilation stacks, particularly on sensitive receivers such as a kindergarten and school. This issue became the major dissenting issue during the planning approval process.

Reassessment of Route Options

In January 2009, following a change of government, the Minister of Transport requested the NZTA to investigate 3-lane alternatives to the proposed longer 2-lane Driven Tunnel option of that time. In particular, the Minister was concerned that the scheme as developed at that time was not affordable and the 2-lane tunnel did not provide for the capacity sought for a 'balanced network.'

In early 2009, NZTA carried out a review of route and scheme options. The previously evaluated options were reviewed and new direct surface options were considered. From this review, three route options were developed for further consideration (Figure 5).

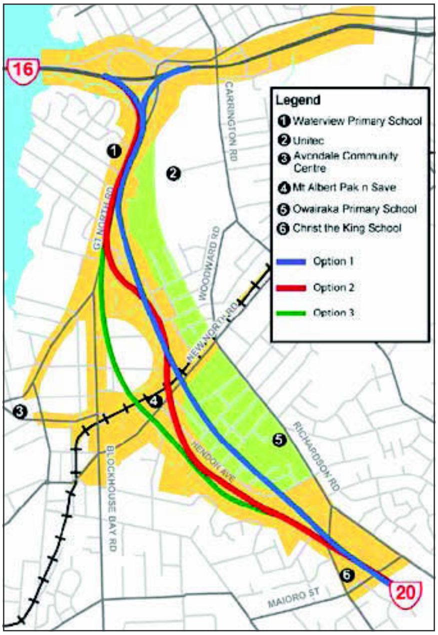

Figure 5. Reassessment of route options

All three options presented connected SH20 to SH16 at the Great North Road Interchange. Each option was described in terms of horizontal alignment (e.g., surface or below ground) but also highlighted areas where progressive levels of mitigation could be provided through scheme design (e.g., to allow sections of each of the options to be built in cut or in cut and cover or in driven tunnels).

For each option the NZTA considered levels of appropriate mitigation related to construction as well as the costs, social and environmental impacts and traffic performance. As a result the NZTA concluded that the most appropriate route option was the Option 3 alignment, with the section through Avondale Heights constructed in a tunnel. This was referred to as the 'Combined Surface Tunnel Option' as it introduced 1km of surface works through the rail corridor, and reduced the length of the driven tunnel to 1km with a 1.3km length of cut and cover tunnel separated by 200m on open motorway in cut. The reasons for this option preference included: the balance of value for money and improved capacity of the project combined with the reduced social and environmental effects compared particularly with the surface alignments. The tunnel option also required 113 less residential properties to be acquired, including reducing the acquisition of social housing properties in the area.

In light of the above assessment, the NZTA released an alignment option based on the combination of surface and tunnel construction, for community and stakeholder comment (May 2009).

During this reassessment phase a protest group was formed in opposition to any new non-tunnel option ('Tunnel or Nothing'). Refer to Figure 6.

Combined Surface Tunnel Option

Following the May 2009 consultation, the feedback received from stakeholders and the community, and the environmental assessments, identified concerns about the effects of the open section of motorway between the driven and cut-cover tunnel sections. As a result, further geotechnical investigations were undertaken to identify options for lowering the alignment to enable the two tunnel sections to be joined, thus removing the short section of open cut. There were geological constraints with this connection and the proximity of Oakley Creek raised issues with respect of potential impacts on this sensitive environment. As an outcome of this work, the geotechnical investigations identified the opportunity to move the alignment to the east, so that the alignment was positioned in material more compatible with tunnelling. This presented the opportunity

Figure 6. Protest group in favor of a tunnel

to extend the length of the bored tunnel section, with a subsequent reduction in the length of cut and cover tunnel, which also had the benefit of a reduced impact on Great North Road and the adjoining properties. A detailed assessment of these alignment and design alternatives was undertaken, including consideration of the Project objectives, costs and environmental considerations. It was concluded that while the revisions proposed to the alignment did impact on different properties from the earlier alignment (albeit only with respect to subsurface effects), there was generally a reduction in the adverse effects on other properties and the receiving environment.

On the basis of this assessment, a revised alignment was identified and in December 2009, the NZTA Board confirmed that it intended to proceed with this as its preferred option for the Project.

PLANNING APPROVAL PROCESS

Amendments to the Resource Management Act (RMA) which covers planning approval for projects in NZ provides the Minister for the Environment with specific powers in relation to applications for Resource Consents and Notices of Requirement that are part of a proposal of National Significance. This legislation allows the Ministers for the Environment and Conservation to make a direction that the Notices of Requirement and resource consents be referred to a Board of Inquiry(BoI), conducted by an Environment Court judge. The BoI process essentially goes straight to the Environment Court rather than a traditional planning approval followed by an appeals process. The Board of Inquiry process is legislated to have a fixed duration of 9 months and was designed to avoid major projects of National Significance being held up by an extended planning approval process.

The Project required designations under both the Auckland City and Waitakere City District Plans and resource consents under the Auckland Regional Council's Regional Plans. The NZTA is a Requiring Authority and as such the NZTA has a mandate to seek to designate land for the State highway network in accordance with its functions. For those activities not covered by the District Plan, resource consents are also required to enable construction, maintenance and operation of the Project. In particular, regional consents are required for works in the coastal marine area (e.g., for occupation and discharge), for works to divert and discharge surface water, stormwater and groundwater, for the disturbance of contaminated sites, for works within watercourses (e.g.,

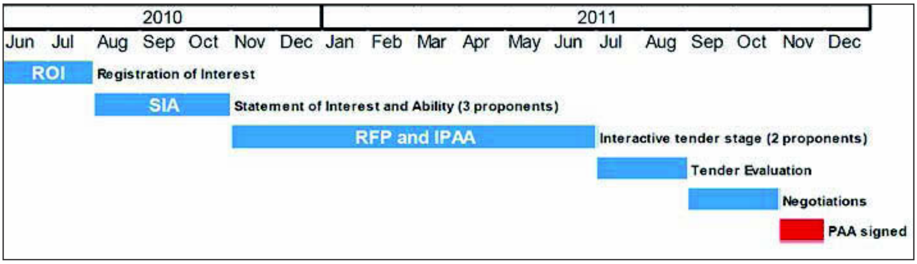

Figure 7. Procurement program

reclamation, occupation and discharge), and for land use activities (e.g., earthworks or land disturbance activities).

The NZTA's application for designation and resource consent to construct the Waterview Connection was lodged with the Environmental Protection Authority in August 2010 and referred to a Board of Inquiry soon after. Hearings to consider the application began in February 2011, where the Board considered all the evidence and submissions on the project from the NZTA, local authorities, the community and other interested parties.

The Board of Inquiry's draft decision was made in early June 2011 and some changes were made to the NZTA's proposed set of conditions. These changes, which include moving the location of the ventilation buildings and stacks and additional open space requirements, reflected the Board's considerations of the effects on the community.

The BoI process involved considering detailed investigations and evidence provided as part of the NZTA's application as well as taking into account the community's concerns.

The NZTA had 20 days to review the draft and provide feedback to the Board, before the Board released its final decision on 30 June 2011.

CHALLENGES WITH CONCURRENT PROCUREMENT AND PLANNING APPROVAL

In response to the global financial crisis, the Government wanted to fast track the RoNS projects as part of the economic stimulus programme. The final route alignment was determined in December 2009, and the government was seeking to have a contract awarded 23 months later in November 2011. At that stage the environmental assessment required for planning approval had not been completed, the planning approval process had not commenced, and there was no procurement documentation or a detailed timetable for the procurement. The NZTA was able to utilise the BoI planning approval process which saved 12-24 months programme, but it was also necessary to commence the procurement prior to achieving planning approval. NZTA did not want to compromise the procurement methodology which may have jeopardised the objectives of minimising risk and achieving value for money. The procurement programme was 19 months in total from Registration of Interest to the signing of the Alliance agreement. Refer to Figure 7. Further details of the procurement methodology can be found in Spies and Ireland (2013).

So in order to achieve the Government's timetable of contract award by November 2011 it was necessary to undertake the procurement in parallel with the planning approval process.

This is an unusual approach but saved 12 months on the pre-contract phase programme. The project was procured using a Competitive Alliance, and the actual competitive tender design and pricing phase of the procurement commenced in November 2011 with tender submission scheduled for early June 2011. So when the competitive phase of the tender commenced the planning process had already been running for 2 months. The parallel programmes were aligned so that the tender would not close until the final consent conditions were available, as NZTA considered that the competitive pricing would be compromised without finalised consent conditions that formed part of the requirements to be met by the Alliance. One of the benefits of a BoI process is that there is certainty of the timeframe, as the planning approval recommendation is provided by the Environment Court, and therefore no appeals (except on legal process) are permissible. This timeframe certainty meant that concurrent programmes were possible without restrictive interdependencies.

Another benefit of the BoI process is that the consent conditions are nominated by the Proponent, and then modified based on the submissions received, the hearing process, and the caucusing of experts. So although the consent conditions were being modified throughout planning approval process, there was also a set of conditions that could be used for construction planning and tender design. Various other measures were put in place to allow each of the tendering consortia to keep up to date with the planning approval process, so that the compliance risks could be included in the tendered price. These included:

- Invitation to attend the Hearing days,
- Technical consent meetings held every 2 weeks with the NZTA's advisors where the tenderers could be updated on the planning process, and
- Access to NZTA's legal advisors for the planning process.

With these measures each of the consortia were satisfied that consent compliance scope and risks were well understood and could be priced with certainty.

The main challenge presented by the concurrent planning approval and procurement, was an outcome of the planning process whereby the ventilation stack at the northern portal was moved away from the tunnel portal. Although NZTA's planning team was aware that the northern ventilation stack was a contentious issue, there was no pre-warning that a condition of consent for the project would require the relocation of the stack away from the northern portal. This change reflected the Board's considerations of the effects on the community, and specifically moved the stack further away from the existing school and kindergarten, and also reduced the visual impact of the stack.

The tenders were due to close one week after the draft consent conditions were received, and the impact on moving the stack on one consortia's design was significant. So rather than significantly delay the procurement programme, the NZTA accepted the risk for that consent condition, whereby if agreement could not be reached to maintain the stack adjacent to the portal then the works to relocate the stack to the BoI position would be a variation, which was the outcome in the end. As the Alliance contract is open book, and NZTA pays all direct costs, this outcome was not adverse when compared to the procurement time savings.

CONCLUSION

Waterview Connection was first mooted in 1996, with the contract awarded in November 2011. It is forecast for completion in early 2017 meaning it has been a 21 year period from project inception to opening. There was 10 years of project development before a proposed surface route was investigated as a tunnel. Although the local environs included many development constraints as is typical for an urban motorway, there are no constraints that would normally mandate a tunnel, such as steep terrain or a watercourse, however urban tunnels are increasingly required due to a lack of community acceptance of the impacts associated with a surface motorway.

In response to the global financial crisis, the Government wanted to fast track the RoNS projects as part of the economic stimulus programme. Following confirmation of the tunnel route, it was only 2 years for environmental assessments, planning approval and contract award, which is a very short timeframe when compared to other similar projects. This was achieved by undertaking the planning approval process concurrently with project procurement without any compromise of procurement methodology which may have jeopardised the objectives of minimising risk and achieving value for money. This measure, along with the BoI planning approval process, saved over 2 years in the project development program.

ACKNOWLEDGMENT

The authors thank Tommy Parker for contributing to this paper.

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EXPERIENCE GAINED AT NEW SUBWAY LINE U5, BERLIN

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ABSTRACT

The existing U5 underground line in Berlin currently ends at Alexanderplatz station in Berlin-Mitte. It is planned to close a gap of approximately 2.2 km in the section between the existing tunnels at Berliner Rathaus and Brandenburger Tor station. The line is set to run through downtown from the new Berliner Rathaus station to be built below the Spree, underneath the future Berliner Schloss (Berlin Palace) and the Spree Canal, and along Unter den Linden to Brandenburger Tor. The line includes three new stations (Berliner Rathaus, Museumsinsel and Unter den Linden). Two parallel tunnels each approximately 1.6 km long are to be built using a slurry pressure balanced TBM. The passage under the retaining walls of the banks of the river Spree, integration with the construction of the future “Berliner Schloss,” and the link to the existing Brandenburger Tor station present major challenges in terms of the design and execution of the shield tunneling. Construction work started in April 2012.

PROJECT OVERVIEW

The existing U5 underground line connects the village of Hönow, located at the eastern border of Berlin, with the Alexanderplatz station in the centre of Berlin. The first construction section of this line, between Alexanderplatz and Friedrichsfelde station, was opened in the year 1930.

The idea to extend this line from Alexanderplatz station into the west through the Mitte and Wedding districts up to Tegel airport was picked up again short after the German reunification, although the new plans were restricted to the section between Alexanderplatz station and Berlin main station.

The existing U5 underground line in Berlin currently ends at Alexanderplatz station. The new section is intended to close the gap by connecting the existing tunnels at Berliner Rathaus with Brandenburger Tor station. The tunnel section (underground line U55) from the central train station (Hauptbahnhof) to Brandenburger Tor was previously completed. The new link between the U55 and the planned U5 will connect Berlin-Mitte with the central station.

The U5 project in Berlin comprises the construction of three new underground stations and one connecting twin-track tunnel to be built using the shield-tunneling method. The project is divided into two construction lots, or contracts. Lot 1 covers the track cross-over (GWA), Museumsinsel station (MUI), Unter den Linden station (UDL), the link to Brandenburger Tor station (BRT), and the tunnels connecting these stations. Lot 2 comprises the construction of the new Berliner Rathaus station and the link to the existing tunnel toward Alexanderplatz.

The alignment of the new tunnel system beneath the City will run from the new Berliner Rathaus station to be built below the Spree, underneath the future Berliner Schloss and the Spree Canal, and along Unter den Linden to Brandenburger Tor station. The length will be approximately 2.2km. (Figure 1).

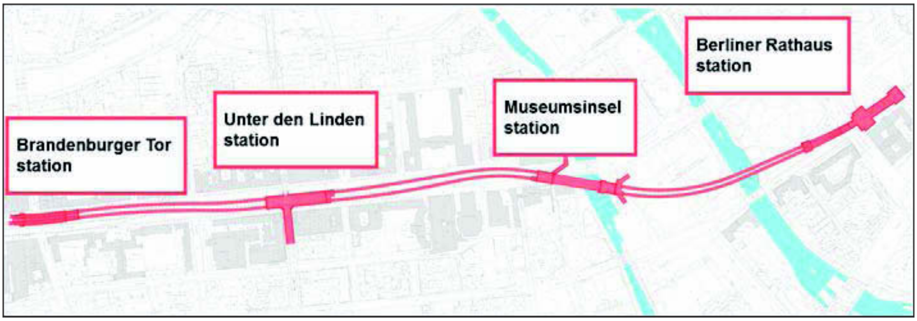

Figure 1. Alignment of the new underground line U5

Two parallel tunnels are planned, each approximately 1.6km long. The tunnels will be bored using a slurry pressure balanced TBM.

Three new stations are to be constructed using both the cut-and-cover and the cover-and-cut methods. Berliner Rathaus station will be constructed close to the Berliner Rathaus using cut-and-cover. The station includes the link to the currently operational tunnels (sidings) on the existing route of the U5. A track cross-over, partly constructed by cut-and-cover and partly by top-down, is planned for the station area as well. This is where the TBM will be launched.

The Museumsinsel station will be in the area of the Spree Canal. The construction of the station will require the excavation and support of two shafts with an underground excavation for the platform area between the two shafts. Due to the anticipated ground conditions and to mitigate potential surface impacts, the project specifies the ground within the platform area to be frozen prior to its excavation.

The existing U6 underground line and the new U5 will cross at Unter den Linden station. To allow the crossing structure to be built, the existing tunnels will have to be demolished underground and rebuilt to accommodate the interchange facilities.

Preparatory work started in April 2010 and the underground works began in April 2012. The link between Alexanderplatz and Brandenburger Tor is scheduled to be opened in 2019.

Various design considerations are outlined below.

GEOLOGY

Berlin-Mitte is located in the Berlin glacial valley running east to west. It was formed as part of the Warsaw-Berlin glacial valley at the end of the last ice age and is characterized by massive deposits of sand and gravel that act as a ground-water reservoir. Locally, the sands are overlaid by organically permeated sands or peat and organic silt that are extremely thick in some places. In the area between the Spree and the Spree Canal in particular, these highly organic sand and silt deposits extend as deep as the planned tunnel excavation.

Construction of Museumsinsel station will require the foundations of the structure to be sunk into the layers of marl below the sand strata.

The results of the exploratory drillings indicate that large cobbles and boulders will likely be encountered when constructing the diaphragm walls and during shield tunneling. The sandy ground is considered highly abrasive on account of its quartz content.

On the average, the ground-water table is located at approximately 3.0m below ground level. The abundant glacial outwash consisting predominantly of sands form a large, contiguous aquifer in and around the area explored.

CONSTRUCTION PRINCIPLES

Due to its central location in the inner city and the above mentioned ground and groundwater conditions, the planning for the new construction of the U5 underground line between Alexanderplatz and Brandenburger Tor stations are based on the following ground and construction principles:

- The tunnels are excavated mechanically by a slurry pressure balanced tunnel boring machine. The two tunnels are constructed one after the other from east to west with only one TBM. After the first drive, the machine will be dismantled at Brandenburger Tor station (BRT) and transported back through the tunnel to the start-up shaft. There it will be prepared for the second drive.
- The construction pits will be built mainly by cut-and-cover method with double-phase slurry walls, deep jet grouted slabs and bracing grids. Further excavation and the construction of the stations will be continued under a stiffening concrete cover.
- With the slurry pressure balanced tunneling method, the construction pits of the Museumsinsel (MUI) and Unter den Linden (UDL) stations will be driven through before the pumping out and the excavation. By doing so, the driving into and out of the pits under high water pressure differences is avoided, with the exception of the start and the finish procedures.
- All diaphragm walls will be reinforced with fiber glass reinforcement (at the location of penetration) to allow TBM driving through the walls.
- Jet grouted blocks, which have both a load-bearing and waterproofing function, will be completed by a second, redundant sealing system (injection, ground freezing etc.), so that a potential defect in the water tightness of the jet grouted block does not directly lead to an erosion and, as a consequence, to a loss of structural safety.
- The Museumsinsel (MUI) station platform hall will be excavated in a three-cell cross section under protection of a freeze mass. A road header will be used for excavation after the TBM has driven through twice and after the initial lining of the tunnels. The freeze mass has both a load-bearing and waterproofing function (Figure 2).
- Unter den Linden (UDL) station will be built within a cross-shaped construction pit below the crossroads of Unter den Linden-/Friedrich Street with the top-down method. First, the upper platforms of U6 underground line which run north-south below Friedrich Street are completed. The U6 is planned as a bridging structure on the slurry walls of the construction pits running east-west (Figure 3). Only after this section has been completed and the U6 underground line is operating again, the shield drives running east-west below it and, after this, the complete excavation of the platforms of the U5 line will be done. With this construction method, the closure of the U6 underground line can be limited to a period of about 16 months.

DRIVING THE TUNNELS

Tunneling with Slurry Pressure Balanced TBM

The boring commences from the shaft at the track cross-over and ends before Brandenburger Tor station. The tunnels will be bored utilizing a slurry pressure balanced tunnel boring machine. The tunnels will have an inside diameter of 5.70m.

The final lining for the tunnels will consist of precast reinforced concrete segments. A segment ring with a thickness of 35cm and a length of 1500mm is planned (Figure 4). The segment joint is sealed using a closed elastomeric gasket embedded in a channel.

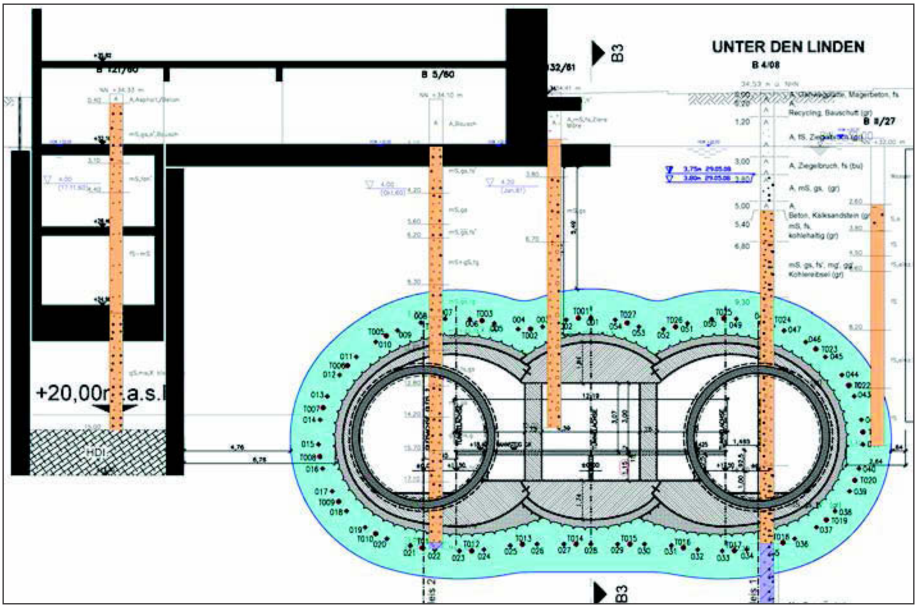

Figure 2. Freeze mass at Museumsinsel station

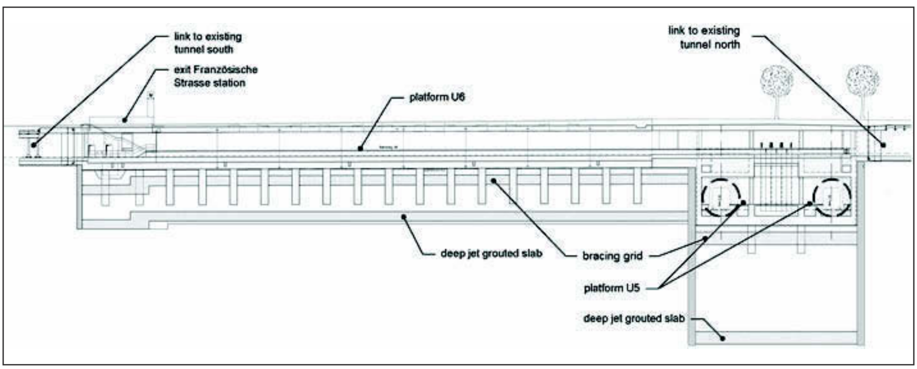

Figure 3. Unter den Linden station, U6 long section, U5 cross section

The sealing is required to accommodate a maximum ground water pressure of 3.0 bar. It is planned to arrange the drilling holes in the segments in such a way as to allow secondary grouting of the tail void.

The tunnels will be excavated consecutively. The TBM will be started from the shaft in the area of the track cross-over. The construction shaft and the track cross-over will be constructed using the diaphragm wall method. The diaphragm wall will be reinforced with fiber glass reinforcement at the location the tunnel penetration.

A redundant sealing system consisting of a launching chamber with lip seals, inflatable emergency sealing, and a jet grouted block installed in front of the diaphragm wall is envisaged for the start of tunnel excavation.

The tunneling will run under the Spree, the site of the former Palace of the Republic on which the future Berliner Schloss will be constructed, the Spree Canal, the Bertelsmann building, the Linden Tunnel, the statue of Frederick the Great, and the

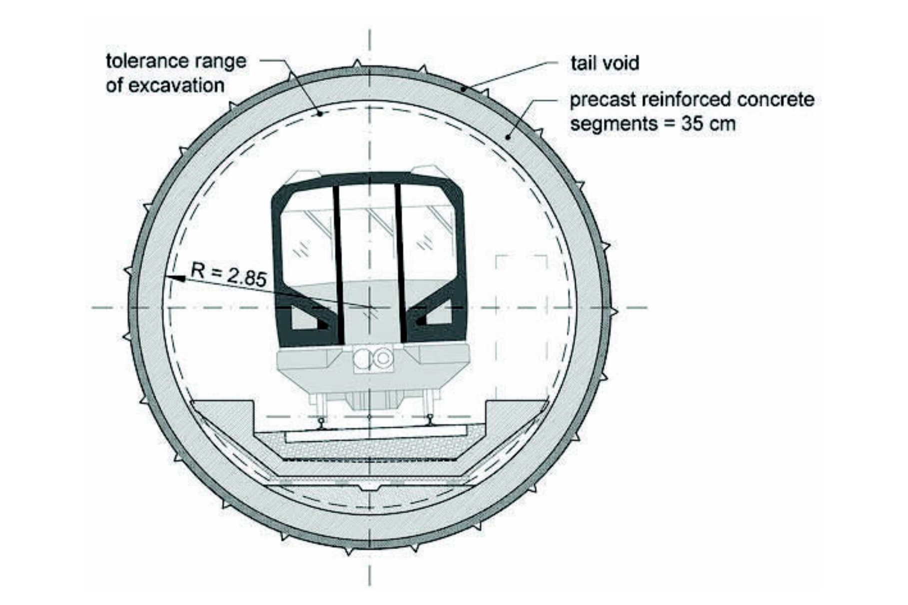

Figure 4. Standard cross section

suburban-rail tunnel for the north-south train line in the area of Unter den Linden. The underground crossings of the river and canal present special challenges for shield tunneling on account of the geology and the depth of approximately 6.0m below the river bottom. The river bottom will need to be ballasted with steel plates or heavy concrete blocks to help prevent ground loss as well as to help prevent migration of slurry fluid to river. The ballast has to be limited in height to ensure that shipping is not impaired during construction.

Monitoring

The boring of the tunnels downtown will require the construction work to be comprehensively monitored. To mitigate negative impacts due to ground movements, deformation values were specified for each structure and for each third-party facility located in the area of the projected settlement trough, and from this alert and alarm thresholds were derived.

To check compliance with the deformation values, a monitoring program complete with specified effective cross-sections, monitoring equipment and monitoring frequencies, and the objectives of the monitoring evaluations are defined as minimum requirements in the tender documents. Both effective cross-sections to monitor ground movements associated with the shaft and station excavations and to monitor and analyze the settlement trough resulting from the TBM mining will be required.

CHALLENGES ALONG THE STRETCH

The designer and the contractors have been faced with various challenges regarding the construction along the tunnel stretch, some of which are described in extracts below. The underground construction works already started and will be shown in more detail in our oral presentation.

Harbor

In order to avoid mass transports through the inner city to a large extent, a new dock has been built in the river Spree directly beside the start-up pit and the site installation area. Due to the large traffic on the river, the navigable cross-section of the river at this place had to be enlarged. The construction of this harbor was awarded as a separate contract and is already being realized. The evacuation of the material resulting from the shield drives and the delivery of the concrete segments for the tunnel construction can be realized completely by water transport after the harbor has been finished.

Sheet-Pile Wall in the Cross Section of the Shield Drives at the Bank of River Spree

About 110 m after the start-up pit, the shield drive crosses below river Spree. The low overburden between tunnel crown and the river bed requires a ballasting in river Spree. At the western bank of river Spree, a sheet-pile wall, which was used for the construction of the Palace of the Republic as bank wall and support, extends into the cross-section of the shield drive. Initial plans to recover this wall from the dismantling chamber by means of ground freezing technique were given up, because the western bank of river Spree is temporarily accessible now because the Palace of the Republic has been demolished. According to recent plans, the sheet-pile wall is to be removed out of a temporary construction pit in the river Spree.

Crossing Under the “Palastwanne” a nd the Berliner Schloss Construction Pit

When the Palace of the Republic had been demolished, the massive foundation slab had to remain in the ground, as, otherwise, a considerable lowering of groundwater would have been necessary (which would not be tolerable today). The old so-called “palace basin” will be integrated into the new structure when the future Berliner Schloss is built. The construction of the new Berliner Schloss started in May 2012 with the awarding of the “construction pit” lot and will be realized at the same time as the construction of the U5 underground line. Due to this, the “palace basin” and the Berliner Schloss construction pit have to be undercrossed by the shield drives. This results in various dependencies between the two projects regarding technology as well as schedule.

Bridge “Schlossbrücke”

About 450 m after the start-up shaft, the tunnel alignment with the Museumsinsel (MUI) underground station crosses under the river Spree in a very acute angle to the bridge “Schlossbrücke.” The Schlossbrücke connects the east end of “Unter den Linden” boulevard with the isle “Museumsinsel.”

Below the Spree canal, the platforms of the future train station will be located. The exits will be situated east of the Spree canal in front of the Berliner Schloss on the isle Museumsinsel and west of the Spree canal. Natural stones line the reinforced concrete arches of the actual structure. The substructures consist of quarry stone brickwork and concrete. The abutments are based on wooden piles. During the shield drivings, horizontal drilling works for ground freezing and the driven enlargement of the station concourse, it is possible that a few piles of the bridge “Schlossbrücke” are met and cut. The impact on the quality of the wooden pile foundation and on the deformation of the bridge to be expected from this has been investigated in expert reports. A deformation prediction builds the basis for a comprehensive monitoring system to be applied by the contractor.

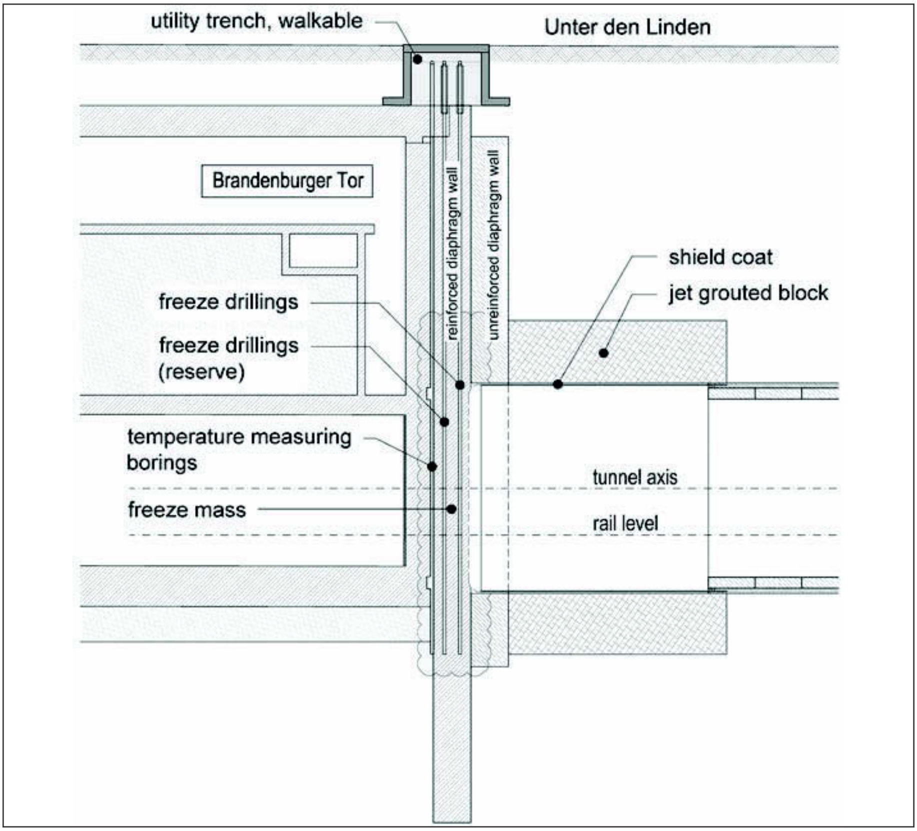

Figure 5. Approaching Brandenburger Tor station, longitudinal section

Anchors at Unter den Linden Station

The Linden boulevard situated at the south-east corner of the Unter den Linden/ Friedrich Street crossroads was completed in 1997. The concreted construction pit which was necessary for the construction at that time was anchored with temporary anchors. These anchors are spread along the Unter den Linden boulevard as well as along Friedrich Street up to the area of the Unter den Linden (UDL) underground station to be built. In order to enable a construction of the slurry walls and the tunnels without any obstacles, all anchors along the alignment of the walls and tunnels next to the Linden boulevard are removed.

Approaching Brandenburger Tor Station

The last great challenge for the two shield drivings is the arrival at the existing Brandenburger Tor (BRT) station. This was constructed by using the protection of a freeze mass out of a head construction pit—similar as now Museumsinsel station. The shield drives of the new-built U5 will end at the front side of the former (eastern) end of the existing Brandenburger Tor station. The reinforced slurry wall of the construction pit was constructed at that time with the protection of an unreinforced diaphragm wall (Figure 5). This is where the shield drive now is supposed to end. The TBM will be dismantled and transported back to the starting shaft. The shield casing will remain in the ground. The remaining tunneling to penetrate the diaphragm wall and the slurry wall of the construction pit wall will be performed manually under the protection of the freeze mass.

There are remaining uncertainties regarding:

- the quality of the slurry wall and the diaphragm wall
- a potential water inflow through the joints between slurry wall and diaphragm wall
- a potential water inflow between slurry wall and the wall of the structure
- safety requirements for entry and exit procedures

This results in the necessity for an additional freeze mass in the area of the joints and the connection of shield casing and diaphragm wall.

FINAL REMARKS

Closing the gap in the U5 underground line is a technically demanding construction project. The construction of the U5 underground line Berlin-Mitte attracts a great deal of interest from the public. The acceptance of the construction project, the construction works which will last several years and the restrictions resulting from them will mainly depend on whether the parties involved in the project are able to realize the project within the estimated construction time (until 2019) and the budgeted cost (approx. 433 million Euro).

The construction work at the stations started in April 2012. The TBM assembling will start in April 2013, TBM drive is foreseen from summer 2013 onwards. In the oral presentation the first experience gained especially with the deep excavations and TBM assembly work will be described.

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PART 12

New and Innovative Technologies—I

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TUNNELING IN BELGIUM

Klaus Rieker ▪ Wayss & Freytag Ingenieurbau AG

ABSTRACT

Wayss & Freytag Ingenieurbau AG is currently constructing three major infrastructure projects in Belgium. The paper will highlight some of the construction challenges of the projects and the sometimes quite unique construction methods developed and successfully implemented on those projects. Some of the challenges were: tunneling under the live runways of Brussels International Airport; construction of a railway tunnel underneath an existing road tunnel in the heart of Brussels utilizing hand-dug diaphragm walls and pipe-jacking and finally slurry TBM tunneling under the harbor of Antwerp with minimal cover.

"DIABOLO" PROJECT AT BRUSSELS INTERNATIONAL AIRPORT BOUNDARIES

Overview

Zaventem Airport in Brussels can be reached by road and rail. However, the volume of traffic on Brussels' motorway ring is very high, leading to severe delays in journeys to the airport. From outside Brussels, the airport could not be reached by direct rail link, i.e., without changing trains. Air passengers had to be prepared for a wait at one of Brussels' main railway stations.

The "Diabolo" Project included the construction of a railway line from the terminus station at Brussels airport below the airport runway to Belgium's new high-speed railway, routed north of the airport along the central reservation of the E19 motorway. Included as well was the modification of a junction on the E19 and re-routing of roads. The two parts of the infrastructure project, which were linked with each other, were the first two privately pre-financed projects (PPP projects) in Belgium.

The project was awarded in October 2007 to a 5-way joint venture, led by a project director of Wayss & Freytag. Shortly after award, work started at various places. The main tunnel drives were executed during 2009, one after the other using a Herrenknecht slurry TBM. The runway was crossed two times without any significant settlement and during full operation of the runway. All works were completed on time. Handover took place in February 2012, with the official opening of the train services by the king of Belgium on June 7, 2012. The project value totaled at about USD430 million.

Project Details

The railway link with a total length of 5,170 m was divided into several subsections (Figure 1), combining several construction methods.

The connecting structures for the new railway line between Brussels and Antwerp, engineering structures KW09 from the southerly direction (Brussels) and KW10 from the northerly direction (Antwerp), were constructed using the cut and cover method. Various reinforced concrete constructions were used for the ramp structure. Whereas the entrance area of the ramp section was built using open reinforced concrete sections, closed box sections were constructed in the further course of the work.

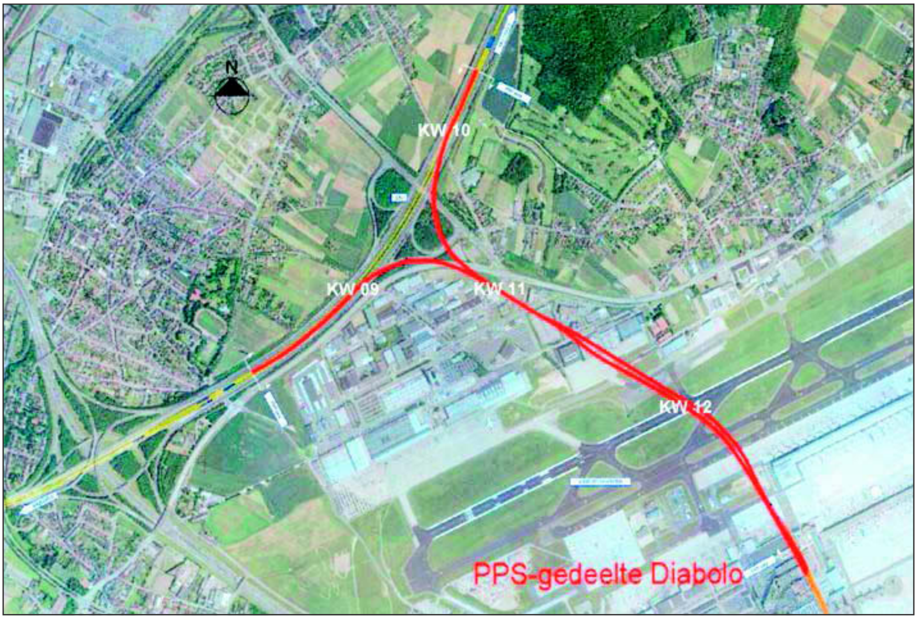

Figure 1. Schematic showing engineering structures KW09–KW12

After passing under the E19 motorway, these two ramp structures join and cross underneath the Vilvoorde-Machelen-Melsbroek junction. To construct the box section in the ramp area for the airport flyover over the E19 motorway, diaphragm walls—so-called “beschoeide sleuven”—were manually built to a depth of 23 m in conjunction with lowering of the groundwater table. This construction method was chosen as maintenance of the existing traffic capacity had to be guaranteed throughout the entire construction period. As a result, several traffic diversions were required to maintain traffic on the existing four lanes. The very cramped conditions did not permit the use of heavy machinery so that sections of the diaphragm walls had to be built by hand. This construction method is well established and in common use in Belgium.

Contract sections KW09 and KW10 are connected to a 480-m-long tunnel. Section KW11 “Tunnel Zone Cargo” was constructed using the cut and cover method. The tunnel crosses the access road to the freight airport. Unrestricted access to the existing freight centers had to be guaranteed during its construction. On account of the nearby buildings many of the existing utility or supply lines had to be moved in advance of the construction of the diaphragm walls.

Reinforced diaphragm walls were built at depths between 25 m and 30 m. On completion of the diaphragm walls the concrete slabs were constructed at ground level. Dry excavation of the soil was carried out below the concrete cover through logistics openings. The groundwater table was lowered to a level below the bottom slab between the diaphragm walls. At the end of the cut and cover section the TBM launching shaft for the bored tunnels was built. To permit the exit of the TBM from the launching shaft and its entrance into the target shaft the reinforced diaphragm walls in the area of the TBM exit and entrance openings were reinforced with glass fibers.

At the heart of the Diabolo project were two machine-driven tunnel tubes with a length of around 1,070 m, which were driven using a mix shield with slurry support. The two tunnel tubes have an external diameter of 8.01 m with a segment thickness

Figure 2. Animation of the tunnel route on the site of the Zaventem Airport, Brussels

of 0.35 m. The maximum groundwater level is 15 m above the tunnel floor. The tunnel tubes are situated completely below the water table.

Geology

Cone penetration tests and exploratory drilling showed that the subsoil at the airport site is soft ground, consisting of backfill material composed of mix-grain fine sands with isolated vegetable remains and embedded stones. The backfill overlies mix-grain layers of sand of the Brussels formation. Clayey and silty components are found in this foundation. However, gravelly to stony fine sands are also encountered at isolated points. The gravel and stones in the fine sand matrix consist primarily of limestone fragments. In the fine sands of the Brussels formation, there are inclusions of limestone conglomerates or limestone benches over the entire route of the tunnel. These limestone benches differ in thickness, some of them more than a foot thick. The mix-grain fine sands above the tunnel roof are loose to medium dense (4 MN/m^2) and, with increasing depth, medium dense to very dense (8–20 MN/m^2). The enclosed limestone conglomerates and limestone benches encountered are firm to hard (20–40 MN/m^2).

Tunnel Construction

Figure 2 shows the tunnel tubes constructed on the airport site. Two aircraft maintenance hangars belonging to the companies “Lufthansa Technik” and “Brussels Airlines” can be seen in the background. The tunnel route runs below the hangar on the left. The two tubes were driven with a shallow cover between the tunnel crown and the foundations of the buildings. There is only a clearance of 3.5 m between the tunnel roofs and the pile foundations of the hangar. The maximum allowable settlement was limited to 15 mm over the entire tunnel route after the two drives. To reduce settlement at the building and to protect the tunnel tubes, jet grouting was carried out at the level of the pile foundations. The grout bodies installed were to transfer the loads to the sides of the tunnel tubes.

The tunnel route runs under aircraft parking bays, several taxiways and the airport’s most important runway for take-offs and landings. The airport operator did not permit the closing of the runway during tunneling operations so that the joint venture had to take appropriate measures. Provision had to be made for settlement-sensitive supply and disposal lines located along the tunneling route. Prior to tunnel driving, these lines were documented and inventoried and adequate safeguard measures taken in consultation with the supply companies.

In accordance with the safety concept for the tunnel, the first cross-passage was constructed after around a quarter of the drive length. After half of the tunneling route an evacuation shaft was to be constructed at the center between the two tunnel tubes. This shaft lies between the taxiway to the cargo zones and the runway. An escape tunnel leading to an existing underpass below the runway ensures that the passengers will not be endangered by air traffic in the event of evacuation of the rail tunnel. This escape tunnel was constructed using prefabricated components. The timeframe for the construction of the diaphragm walls of the evacuation shaft and escape tunnel was very tight. This work was completed during a shutdown of the runway in mid-2008, prior to the start of the tunneling works.

Another cross-passage was constructed between the evacuation shaft and the target shaft. The cross-passages and the connections between the tunnel tubes were driven using ground freezing techniques to support the ground.

The tunneling route ends on the airport site. The construction of the target shaft and the dismantling of the TBM were subject to the airport security regulations, which required planning well in advance and close collaboration with the airport security authorities and the airport operator. All employees working on the airport site had to be in possession of an access permit, which was only issued by the airport operator on application. This affected the planning and execution of work. Construction site logistics had to be adapted to the fact that only escorted transports to the construction site set-up area were permitted. Simulation calculations with the airport control system were performed for the use of heavy machinery (cranes, diaphragm wall excavators) in this area to ensure that air traffic safety was not impaired. The maximum permitted boom heights of the machines were limited.

These preconditions also applied to the construction of the approximately 400-m-long transfer tunnel. Diaphragm walls were installed for the construction of this tunnel. Temporary reinforcement had to be installed during soil excavation, which was then removed on completion of the bottom slab. For the construction of this bottom slab, the groundwater level in this area was lowered below the level of the bottom slab. The bottom slab and the roof were designed as special hollow box sections.

The target shaft and part of the transfer tunnel were located close to a former fuel store. In this area part of the soil was contaminated so that during construction additional measures had to be taken, both for the excavation work and the lowering of the groundwater level.

The gap between the underground station airport station and the transfer tunnel was closed by converting the terminus to a through-station. The three existing rail platforms were lengthened to around 100 m so that they can be used by high-speed trains. Conversion of the station took place below the existing terminal buildings. The diaphragm walls necessary for this purpose had to be built in extremely cramped conditions. Most of the existing utility lines (electricity, water supply and wastewater disposal) had to be moved and new lines laid, but without restricting the use of the terminal and an existing hotel.

SCHUMAN-JOSAPHAT RAIL LINK

Project Details

The project (USD300 million) is to link two railway lines running north-south through Brussels. This new 1.2-km-long east-west tunnel link between the European business district (Schuman) and Josaphat will, together with the Diabolo project, enable circular train services between the airport and city center and should promote public transport rather road traffic.

These infrastructure works are taking place entirely within the dense urban environment of the city center. The solutions for carrying out the works required the least

Figure 3. Simulated view of the new multi-modal interchange

possible disturbance, minimal land take and purchase of buildings. These circumstances played a part in selecting techniques that are sometimes manual and laborious, with relatively slow progress of work. Most of the works are underground using conventional methods such as full face excavation, beneath buildings and other structures. There had been a limited amount of mechanized excavation, i.e., pipe jacking. Furthermore, the urban environment involves limiting the amount of access shafts, thus complicating the supply of construction materials and spoil removal.

The contract was divided into two work packages. Lot 1 comprised widening of Brussels-Schuman station and turning it into a multi-modal, multilevel train-metro interchange (see Figure 3). This interchange is partly located beneath the old buildings of the Résidence Palace, where a number of Belgian ministers were housed. Another part of the interchange lies beneath the tunnel under the Rue de la Loi, a busy street, and then connects to the foot of the Berlaymont building, the headquarters of the European Commission. Furthermore, a new building for the European Council will straddle the new rail box tunnel.

Lot 2 comprised a 1,250-m-long tunnel, connecting the Schuman interchange to Josaphat. This lot had two sections to be carried out using different method, depending on whether the tunnel is beneath roads and buildings or beneath the existing Cortenbergh Tunnel, an important road access for the Brussels district. In addition, in some places the existing road tunnels are in a poor state and required renovation works.

Geological Conditions

The local geology is relatively favorable for underground works. The soil is fine-grain tertiary sand with beds of sandstone of variable thickness. However, these sandstone beds are generally quite small and easily excavated although there is a considerable

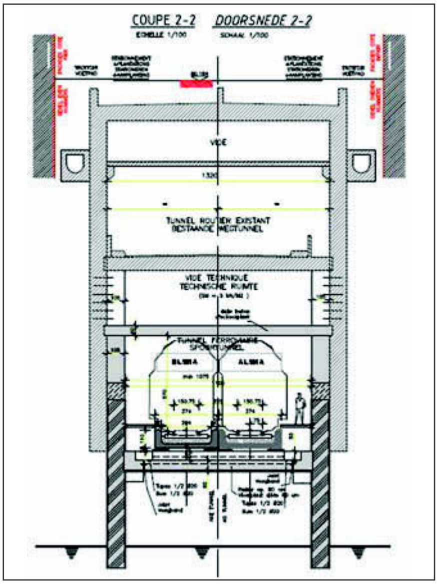

Figure 4. The Cortenbergh Tunnel

Figure 5. Manual diaphragm wall excavation

presence of blocks of sandstone in some places. Occasionally, sandstone decalcification has taken place, leading to cavities beneath these horizons. The groundwater layer is situated much lower than the underground structures although there are some higher aquifers. The absence of groundwater has enabled the use of underground trench techniques, described below. The sandy layer is 35 m thick. The sand located immediately above the sandstone bed is much less compact.

Lot 1: New Station and Route beneath Buildings and Roads

The existing Brussels-Schuman station is located beneath Rue de la Loi, in front of the Berlaymont building. The new tunnel will be connected, the station will serve as a multi-modal interchange (see Figure 3). Constructing the station involved closing it to road traffic for three months in summer. Metro traffic continuity had to be guaranteed beneath the railway lines at all times. The top and bottom slabs of the existing tunnel were replaced by a grid of beams. These beams rest on side beams running parallel to the road, 40 m long, fitted with post-tensioning cables. Ultimately, the railway tracks will run above the metro station within the new underground space created.

The new metro station is shown in Figure 3. The beam grid can be seen, as can the two continuous skylights allowing natural light into the complex. Closure of Rue de la Loi took place in summer 2012. Lot 1 also included the construction of a number of additional entrances to the multi-modal interchange.

Lot 2–Section 1

This lot consists mostly of a 920-m-long tunnel running beneath the road tunnel under Avenue de Cortenbergh and the link to Lot 1, which involves passing beneath five buildings.

Figure 4 shows the new composition of the Cortenbergh Tunnel. Galleries were excavated beneath the bottom slab of the existing road tunnel. A support slab level was built, with full face excavation being carried out from this level. This excavation

Figure 6. Microtunneling machine shield face

Figure 7. Driving transverse tubes

was in order to deepen the side walls and thus increase their load-bearing capacity. Once the excavation was deep enough support slabs were poured. The new walls were anchored chemically to the old ones.

Figure 5 shows the conditions of manual labor for full face excavation. Excavations were carried out with pick and shovel for a height of between 0.40 and 0.60 m. Spoil was loaded into a container which was hauled to the surface using a hoist. Immediately after excavation, the reinforced concrete pre-fabricated slabs were installed and the steel stays placed between the panels. The reinforcement had to be made using short-length rebars since the stays took up space in the working area. Once the reinforcement was in place, the concrete was poured into the trenches. This method nonetheless enabled a fairly respectable rate of progress to be achieved.

In early 2012 the new tunnel was completed although requirements in terms of noise and disturbance in an urban environment were very restrictive. No unacceptable settlement occurred and there was no major damage to neighboring properties.

Lot 2–Section 2

This 420-m-long section is similar to the previous one, but without the benefit of being underneath a road tunnel. The part beneath Avenue Eugène Plasky was built in a single stretch without access to the surface. The method used is similar to that shown in Figure 5. However, there was no slab present; it had to be built first .Two microtunnels, 3 m in diameter, were bored either side of the roadway, between the start and finish shafts. These side tunnels served as work galleries, from which adjacent pipes perpendicular to the galleries were driven. These pipes form a roof slab for the new tunnel. The side mircotunnels were also used as work tunnels to build the side walls by means of manual excavation. Figure 6 shows the microtunnelling machine.

A total length of 775 m of main galleries was bored using these machines. For the transverse tubes, Figure 7 shows full rings made of steel or reinforced concrete with a steel sheath being driven in. These are welded together in sections, the length of which depends on the width of the galleries. A total length of 1,135 m of secondary pipes 2.1 m in diameter was driven in this manner. This part of the worksite was completed at the end of 2011. The maximum settlement logged for this section was 26 mm, with differential settlement limited to 1/450.

Passing Beneath the Block of Housing on Rue Victor Hugo

A block with three-storey apartment buildings is located at the end of Avenue Plasky. The tunnel has to pass beneath this block and indeed become larger in diameter in

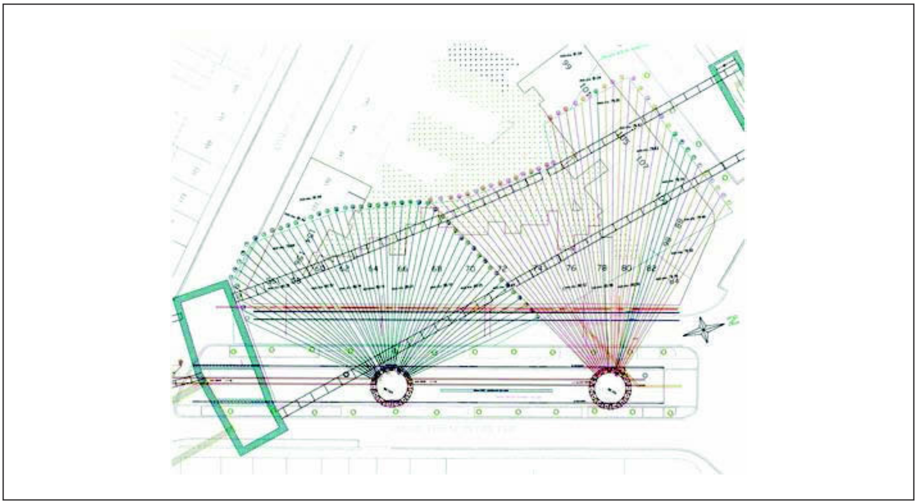

Figure 8. Location of TAMs for compulsory injection

order to encompass the former masonry tunnels and enable the new tracks to connect to those of the eastern Brussels loop. As a result, there were considerably more difficulties in terms of both design and execution at this point.

The first difficulty was that the span of the cross tubes increased from 15 to 25.5 m. The maximum bending strength for a diameter of 2.1 m was not enough for this span. The diameter could not be increased due to the limited size of the microtunnels, so an alternative solution had to be found. This consisted in replacing the slab by parallel galleries with height clearance of 2.5 m. These galleries were connected lengthwise along the tunnel by a roof of adjacent tubes.

Furthermore, due to the greater weight of the roof slab, the amount of settlement of the housing exceeded permissible margins. This settlement was the result of structural deformation rather that any increased stress within the soil. Initially, deformation was to have been offset by jacks placed between the longitudinal tubes and the galleries.

However, a number of drawbacks with this solution were envisaged and therefore compensation grouting was used instead. The layout of the sleeved pipes is shown in Figure 8. The cellars of the apartment blocks are made of brick masonry arches, so there was a danger of breakthrough due to the compensatory pressure. Since access to the cellars was possible during the project, it became possible to implement this solution in part. In addition, permission had to be obtained to make circular shafts. Figure 8 shows the location of these shafts in the central berm on Avenue Plasky. The buildings in this block were fitted with an automatic settlement measurement system. Settlement values of 1.2 mm maximum were measured adjacent to the largest part of the tunnel.

Connection to the Existing Arch

The end of the previous section is a specific piece of civil engineering. A diagram of the structure is shown in Figure 9. An access of limited dimensions was built in the central berm of Avenue Plasky. A gallery was built from this access. The roof of the gallery consists of sheeting driven in horizontally and adjusted with wooden shims. This operation is shown in Figure 10. The gallery was reinforced with steel frames. It also served as an arrival point for the microtunnelling machines described above, as well as the

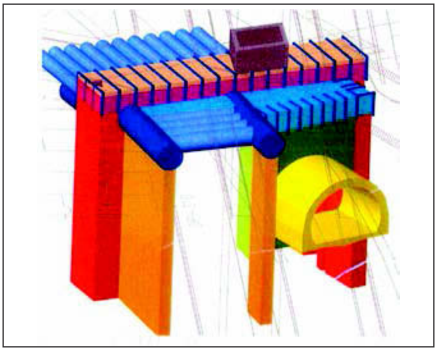

Figure 9. Structure connecting to the arch

Figure 10. Work gallery

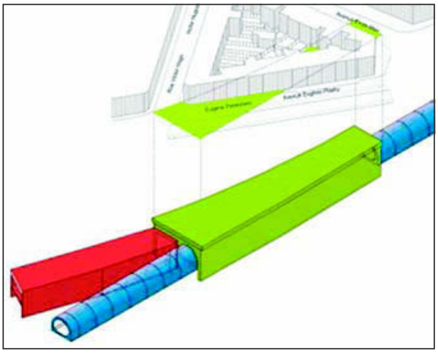

Figure 11. Overview of the connections

start point for the vertical diaphragm wall excavation. A second series of galleries allowed a cap above the old arched tunnel to be built. The connection was completed with a covering wall that closed off the space between the arch and the new frame. This cover constituted the end of the project, with the two rail lines meeting beneath the city block of flats. The whole of the underground structure is shown in Figure 11. The "box" construction is 120 m long. Additional measures were taken to minimise the noise generated by the worksite.

Works for this part of Lot 2 were completed in June 2012 and handed over.

CONSTRUCTION OF THE LIEFKENSHOEK RAIL TUNNEL IN ANTWERP

Overview

Since end of 2008 the biggest infrastructure project in Belgium has been under construction as a Public Private Partnership project in the Antwerp seaport area. The construction works are ongoing; however, the first major milestones of the project were passed successfully on the way to connect the railway freight transportation from the left and the right bank of the River Scheldt in 2014.

Figure 12 gives a schematic overview over the harbor installations with the new S-shaped Liefkenshoek railway link. In addition to the fact that trains will no longer have to leave the harbor territory, the operational costs for the trains will be lower due to the fact that the distance by rail between the major locations on the left and right bank is reduced by approximately 22 km or 45 minutes of travel time.

For a major part the new rail way line runs parallel to the alignment of the existing highway R2, including the immersed Scheldt highway tunnel, which was built in the 1980s. Furthermore, the restricted gradients of the slopes for railway tracks, which are much shallower than for roads, had to be taken into account.

This infrastructure project of approx. 16.2 km total length for railway freight is a Public Private Partnership project (PPP-project) with a maturity period of 38 years, ending in 2051 with the final handover to the Client Infrabel NV. After 2 years of tender,

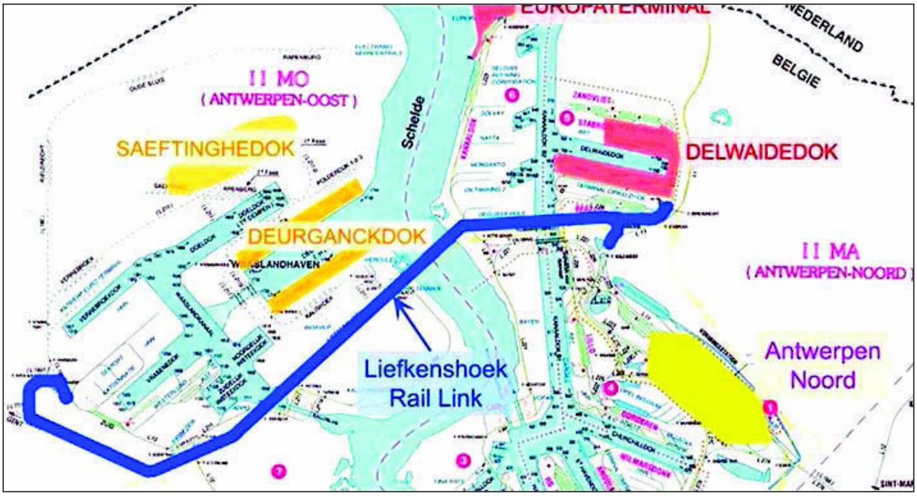

Figure 12. Overview of the harbor area of Antwerp

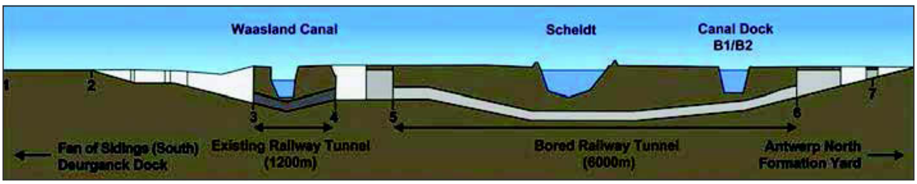

Figure 13. Schematic longitudinal section

the DBFM contract was awarded to a Special Purpose Company. The total value of the Design-and-Build-contract is USD900 million and executed by a four-way joint venture.

Project Details

The project is divided into 13 construction parts, among others two culverts of a length of 280 m, an aqueduct across the track, three street and railway track crossings, the refurbishment of the existing 30-year-old Beveren Tunnel below the Waasland canal, which has never been used, 4,300 m of water retaining slurry walls, 430 m of diaphragm walls, two shield-driven tunnels of almost 6 km length as well as eight evacuation shafts (ES) to be connected to the two tunnels and 13 cross-passages (CP) between the tunnel tubes (Figure 13). In total almost 3 million m^3 of earth moving had to be done, 400,000 m^3 of concrete and approx. 40,000 tons of steel had to be applied.

The execution of the project started in November 2008 on several parts of the jobsite, with the first milestones, the completion of the launching shaft to enable the installation and the start of the two Mixshield TBMs, each 100 m long. After successful assembly of the two TBMs from Herrenknecht, the North TBM was launched successfully in February 2010. TBM South followed at the end of March 2010, 7 weeks later.

In this article the focus will be set on the tunnel related works as well as the tunnel drive itself, the works in the Canal Dock B1-B2, to be done before passing with the TBMs, the works on the cross-passages (CP) and finally the connection galleries between the tunnels and the evacuation shafts (ES).

KW10—The 5.970-m-Long TBM Tunnels

The two tunnel tubes have an external diameter of 8.1 m and are constructed using two hydro-shield TBMs with a diameter of 8.4 m. The thickness of the prefab tunnel segments is 0.4 m and the segments have a width of 1.8 m. The tunnel drives have a maximum inclination of 1.25%. The minimum soil cover along the tunnel alignment is in the area of the Canal Dock with approximately 3 m, which requires special measures in this area. The maximum overburden is approximately 33.6 m; the maximum water column above tunnel invert is approximately 40 m.

The geology along the tunnel consists of several tertiary sand layers of different local formations containing fractions of clay as well as glauconite, and the Boom Clay, a rigid, overconsolidated and fractured tertiary clay, as sealing layer beneath. Most parts of the tunnel alignment are situated in the tertiary sands, but the clay, which has a strong tendency to clog, reaches up to a maximum of 40% of the tunnel cross-section for approx. 800 rings, which is equivalent to 1.44 km.

The 13 cross-passages between the tunnels had to be built during the tunnel drive, which had a big influence on the advance progress of the northern tunnel, from where most of the CP works were started.

Passing the Boom Clay and Crossing of the Scheldt River

After a successful launch of both TBMs the performance in the tertiary sands quickly reached a satisfying level of up to 75 rings per TBM per week. After approx. 800 rings the TBMs started touching the Boom Clay in the invert zone. The advance speed dropped down to sometimes 1 or 2 rings per day due to the high degree of clogging of the cutting wheel.

At ES07, the fourth evacuation shaft along the tunnel drives, situated just in front of the Scheldt dyke, a maintenance box to exchange the cutting tools of the two TBMs and to prepare the TBMs to pass the 10-m-long Scheldt River had been built. TBM South, which had overtaken TBM North due to the CP “stop-and-go-modus” left the box after 3 weeks of intensive maintenance and started to pass the Scheldt River. A hundred rings with clay at the maximum level had to be passed before the advance speed started to increase again. Finally, Boom Clay was left after 1,700 rings.

TBM North arrived in the maintenance box at the end of October 2010. During the preparation to start the maintenance works under atmospheric conditions in the dewatered box, a failure in the sealing block in front of the cutting wheel was found. A cavern of 1.8 m length, 0.8 m width and up to 7 m height in front of the cutting wheel with flowing sand was discovered. Intensive repair was done to refurbish the block and to start the maintenance works after a delay of 1 month. Finally, TBM North left the maintenance box by end of December 2010. With the experiences collected from TBM South regarding the manipulation of the clay and the crossing of the Scheldt River, TBM North advanced with additional 3 rings/day compared to the performance of TBM South in the Clay.

The Scheldt crossing was characterized by the low overburden of 9.7 m at the minimum and a river bed containing silt sedimentation and thick layers of disturbed sediment soil. In combination with the high water pressure of the Scheldt this led to a very small gap between the minimum slurry confinement pressure and the blow-up pressure, with 0.35 bar at its most critical cross-section. On top, the water level variation of the Scheldt, linked to the tides of the North Sea, had to be taken into account. Every 6 hours the level in the Scheldt changes from minimum to maximum level and back. The peaks shift by ¾ of an hour per day. The regular variation of the water level is between +6.5 m and –1 m, but also a spring tide with 2 m in addition had to be considered. As a consequence of the small gap between the decisive pressure levels and the quick change of water pressure, the slurry confinement pressure to support the front face had to be adjusted vey frequently by the operators.

Our tunneling experts worked out a special procedure to ensure a safe crossing of the river even with the very small safety margin. The Scheldt River was divided into green, yellow and red zones, related to their level of severity and special measures were linked to the level of severity. At mid February 2011, TBM North also successfully finished the Scheldt crossing by reaching the safe zone on the right bank, which was reached by TBM South in the second week of January 2011.

Passing of the Canal Dock B1-B2

The second very critical part of the tunnel drives was the passage below the Canal Dock B1-B2 due to the very low overburden of 1.1 m only, combined with the client's requirement to keep the Canal Dock free for traffic all the time. The upward incline of the alignment of the rail tracks was designed as steep as possible to keep the tunnels as deep as possible. Nevertheless, the cover between the bed of the Canal Dock and the tunnel crown was reduced to approx. 3 m. Due to the fact that the soil in the Canal Dock consisted of a silt layer, which reached down to the axis of the tunnel, this soil had to be substituted.

During detail design a spectacular alternative design of the soil substitution combined with a 2.1-m-thick steel-fiber-reinforced underwater concrete slab to reduce temporary backfill to a minimum section and to increase the safety level during passage was proposed. In early 2010, two rows of sheet piles were installed besides the outer edges of the tunnel alignments in a depth of 18 m below water level. The silt inside the sheet pile pit of 270 m length and 35 m width was excavated under water and substituted by 30,000 m^3 of a low-strength mortar. The pouring of the mortar was done in 4 days all day and night at a rate of 400 m^3/h, keeping four batching plants and 120 mixers busy. After the substitution of the silt a 2.1-m-thick steel-fiber slab was cast in October 2010 successfully on top of the mortar as a cover. The very sensitive steel fiber concrete was also cast during 4 days at an overall average rate of 290 m^3/h. Both pouring operations were done via a pontoon fed from the banks of the dock, half from one side and half after turning the flowing pipes to the other side. Finally, a temporary backfill with additional weight on top was installed in the sloped bank to the steel fiber slab. The first TBM started the crossing of the Canal Dock at the end of March 2011 after 4.9 km of tunnel had already been driven.

Cross-Passages

As already mentioned, the CPs were constructed parallel with the tunnel drive works. This required special solutions and high requirements on the organization and coordination of the works to reduce the interferences to an acceptable minimum. All CPs had to be constructed within a freezing body. At each CP a steel platform of more than 50 m has to be erected to start the works at the CPs and enable a permanent passing of tunnel logistics. The northern tunnel had to be blocked regularly to install the platforms and execute the lower drillings of the freezing tubes. A special drill mast was developed together with the drilling subcontractor to perform more than half of the drillings of freezing tubes without any impact on the tunnel trucks. After the completion of the soil freezing work, the tunnel lining, consisting of special reinforced concrete segments, had to be opened and the 8 m long excavation started. Temporary shotcrete was applied before a sealing membrane system was installed to ensure water tightness. Finally the highly reinforced inner lining of the CP had to be constructed. Freezing tubes and the high amount of invert reinforcement in the portal areas required a very professional execution of the works to match the quality requirements and stay within the program.

Galleries to Evacuation Shafts

The tunnel tubes had to be connected to the evacuation shafts, which were constructed as rectangular diaphragm wall shafts between the tunnels, by short, but wide and high connection galleries. The eight shafts are up to 40 m deep and only 3.4 m wide inside. The connection galleries were mainly built within treated soil by blocks of cement-bentonite with a low strength. Only one ES connection was constructed using ground freezing.

To construct the gallery an opening of 3 m × 4 m had first to be cut into the 1.2-m to 1.5-m thick d-walls to remove the reinforced concrete. Then the gallery with a height of almost 6 m had to be excavated within the water-retaining sealing block. The distance to the external side of the tunnel lining is around 0.55 m in the tunnel axis level and up to 1.7 m in the invert. After protection with shotcrete, the tunnel lining was cut and partly removed. The final structure of the gallery is a composite construction of concrete and steel profiles.

All track-related works are completed as of January 21, 2013.

MONITORED DISK CUTTERS—MOBYDIC

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ABSTRACT

Mobydic is the unique disc cutter monitoring system running on pressurized TBM allowing optimizing the excavation process with controlling TBM cutting parameters. It is based on wireless instrumentation embedded in the cutting tools which transmit data, on real time basis, to the TBM supervision system. Data measured are forces, rotation speed and temperature of disk cutter.

It helps for

- cutter disk wear monitoring making then the maintenance prevision more reliable,
- identifying the presence of obstruction like piles in urbanized areas or boulders allowing then either to adapt the TBM excavation speed or to organize an intervention in the excavation chamber for removing the obstacle,
- being informed on the excavations face geotechnical conditions especially useful when mixed ground is encountered by computing real average density and allowing a better control of excavated volume.

Mobydic has been developed since few years by the R&D department of Bouygues Travaux Publics, has been tested first on open mode TBM in France (A41 Highway) and Hong Kong (CLP), and later on a pressurized face EPB TBM (Gautrain) in South Africa. Then, a slurry mixed shield TBM equipped with Mobydic has been used for the excavation of the WIL703 Project in Hong Kong under pressure higher than 3.5bars. Parts of the results from this last utilization are shared in this article.

Mobydic become an essential system for tunneling in hazardous geological conditions and also for tunnels built under high water pressure conditions.

INTRODUCTION

The continuous growth of cities all around the world requires more and more tunnel projects performed with Tunnel Boring Machines (TBM) in more and more stringent conditions. Tunnel project are build in deeper layer, as close to surface layers are already occupied by several underground structures, other Project have to overcome obstacles like existing structures (old piles…) or heterogeneous and complex geotechnical conditions.

As a consequence, TBM cutting tools maintenance operations are more and more difficult and risky in term of safety. It becomes then more and more important to improve the feedback from the tools status as up to now, TBM operators are almost completely blind having only few overall and general parameters like TBM total thrust or TBM cutter head torque acting as an alarm only when huge damaged of many cutting tools or even cutter head structure is already done. As per example, Figure 1 and Figure 2 are showing a damaged cutter head leading to possible long TBM stoppage when not detected early enough.

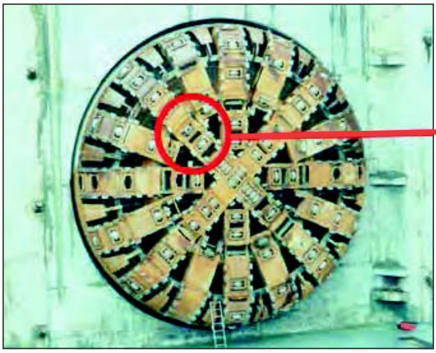

Figure 1. Damaged cutterhead general view

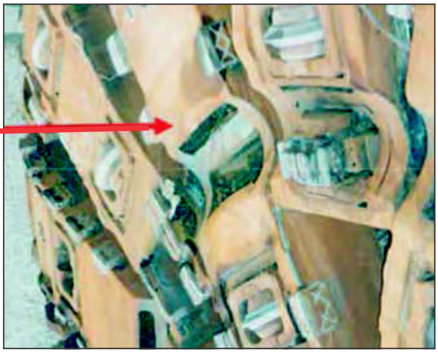

Figure 2. Detail of the damaged structure

Figure 3. Disc-cutter spalling

Figure 4. Flat and worn disc cutter

Cutterhead structure wearing process is always starting by a succession of non detected damaged cutting tools as it is shown on Figure 3 and Figure 4 leading to a direct contact of cutterhead structure with the ground that have to be excavated.

Up to now, this type of wear is mainly detected by human intervention for cutterhead inspections, which are difficult and time consuming due to the complex conditions and for more and more tunnel project.

Based on this unsolved risky issues, Bouygues Travaux Publics decided to develop a real time and continuous monitoring tool for disk cutters called MOBYDIC (MOnitored BouYgues DIsk Cutters).

MOBYDIC OBJECTIVES AND BASIC SPECIFICATION

The purpose of a monitoring system is to provide to the operators reliable indicators. As early stage it has been decided to develop a system able to measure, on real time basis, the cutter disk rotation speed, the temperature and the load applied during the excavation process.

From these measured parameters several indicators can be established:

- Optimization of TBM progress and excavation parameter
- Cutter disk wear status
- Excavation face geological mapping
- Control of excavated volume

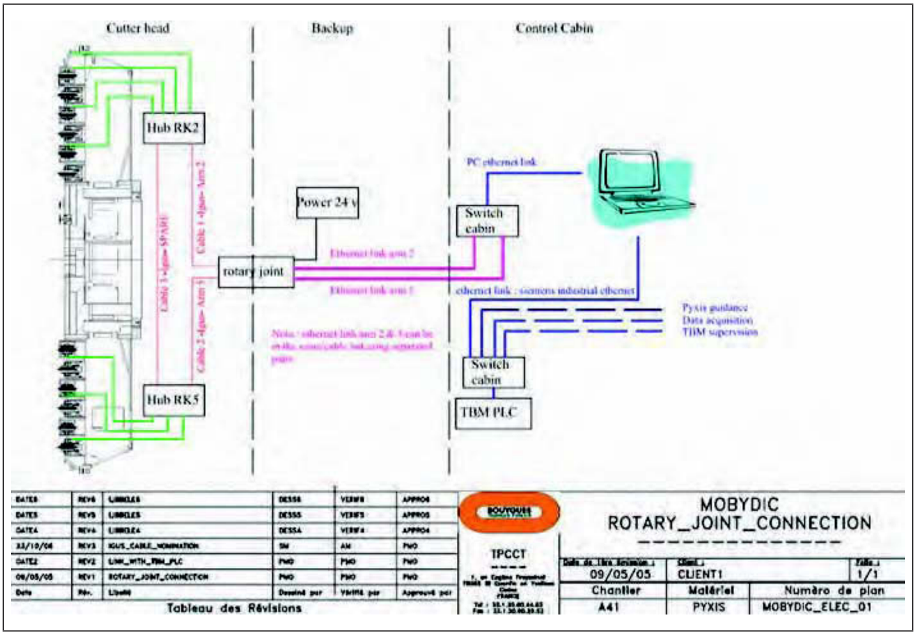

Figure 5. General layout of MOBYDIC instrumentation

MOBYDIC SYSTEM DESCRIPTION AND COMMISSIONING

A wire network is installed into the TBM cutter head structure for connecting via the TBM rotary coupling the electronic devices installed in the cutter disk housing to the computers and PLC installed in the TBM control cabin (Figure 5) for processing. Data acquisition and treatment are carried out at a high frequency.

Highest challenge has been to develop the instrumentation embedded in the cutter disk and its housing. Indeed, the system has to overcome the very stringent conditions encountered in the TBM excavation chamber and on the TBM cutter head (vibrations, pressure, water, dust…). Also, as cutter disk have to be changed regularly, a wireless connection between the removable cutter disk and the housing is mandatory for the data transmission, except for power supply to the cutter disk embedded electronic where induction process system has been developed. Figure 6 and Figure 7 show details of the electronic embedded devices and the now configuration of cutter disk.

In addition, a finite element analysis has been applied on the cutter disk (Figure 8) in order to validate the mechanical modification and determine the optimal positioning of strain gauges measuring the normal load. A calibration of the load measuring gauge has been carried out on a dedicated bench especially fabricated for this purpose (Figure 9).

Last generation of robust components and specific protections against aggressive conditions are used for hardware fabrication. Also an extensive program of testing and commissioning has been carried out in order to validate the hardware and software development. This program includes various cycles of pressure, vibration and heating tests in order to simulate the usual constraints.

MAIN RESULTS

As mentioned before, MOBYDIC has been tested on several sites with various conditions before and recently used as a continuous real time monitoring tool on a Project in

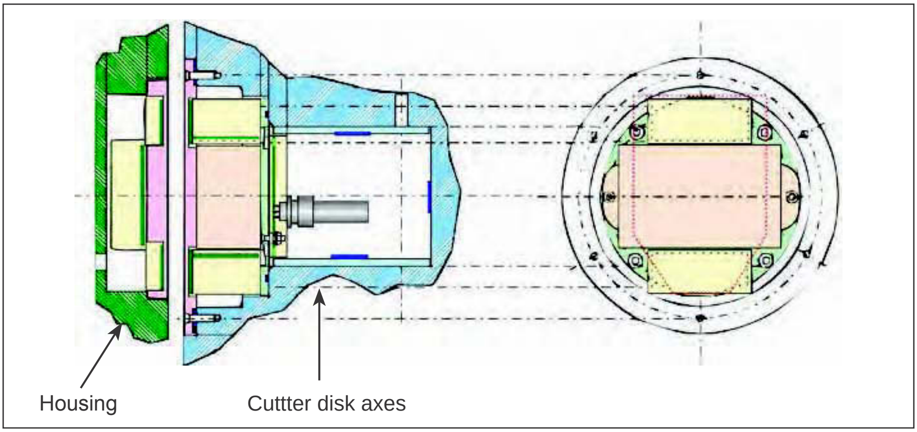

Figure 6. Detailed configuration of MOBYDIC Cutter disk

Figure 7. Cutter disk electronic

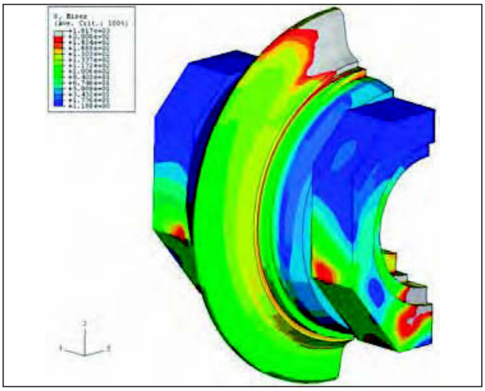

Figure 8. Disk cutter FEM analysis

Hong Kong. Part on the results coming from this site in relation with the above mentioned objectives are presented in the following.

Optimization of TBM Progress and Excavation Parameters

Thanks to Mobydic, the pilot has a real time feedback from the front. He can adjust the advance speed of the TBM to apply the maximum allowed force on the monitored disc cutters. This is very useful in heterogeneous conditions.

Heterogeneous Face

In the case of this kind of geological profile (boulder Figure 10), it is very important to follow the force repartition on the tools.

Only few disc cutters are in contact with the hard materials (Figure 11). In this case we count 3 discs in

Figure 9. Load test bench

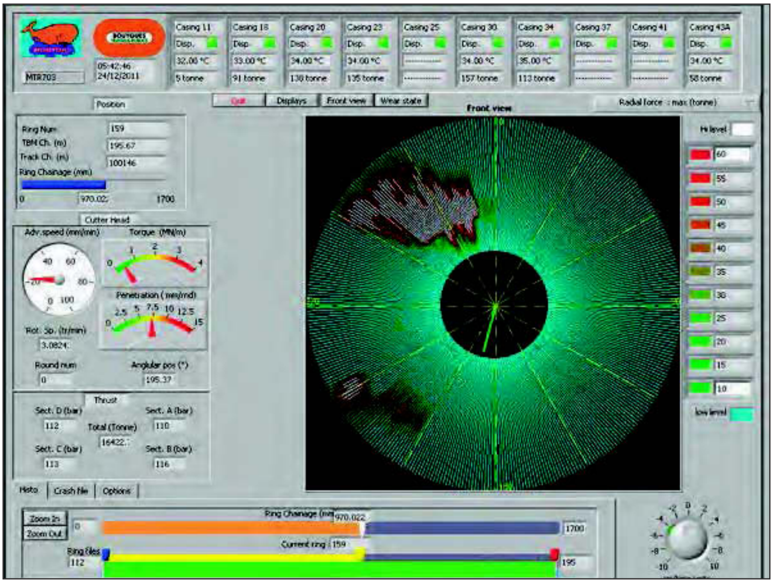

Figure 10. Heterogeneous face

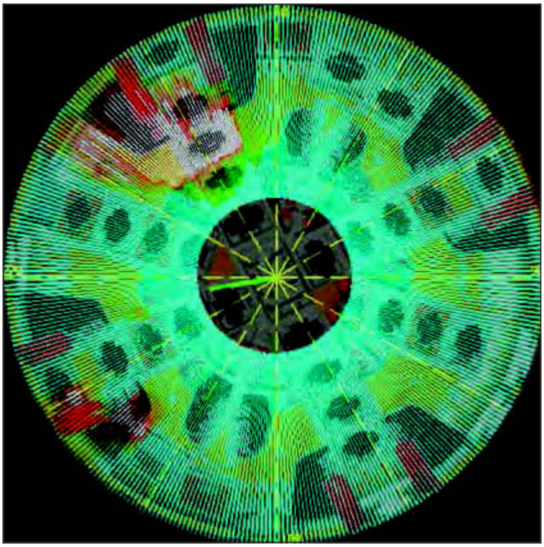

Figure 11. Only few disc cutters in contact with the boulders

contact which concentrates a high percentage of the thrust force. In Figure 10, the disc 20,23,30,34 are receiving forces higher than 100 T in contact of the boulder.

Adjust the advance and penetration to keep these maximum forces under the critic level allows preserving the tools and avoiding long maintenance stop. In an homogeneous face the pilots can increase the advance safely.

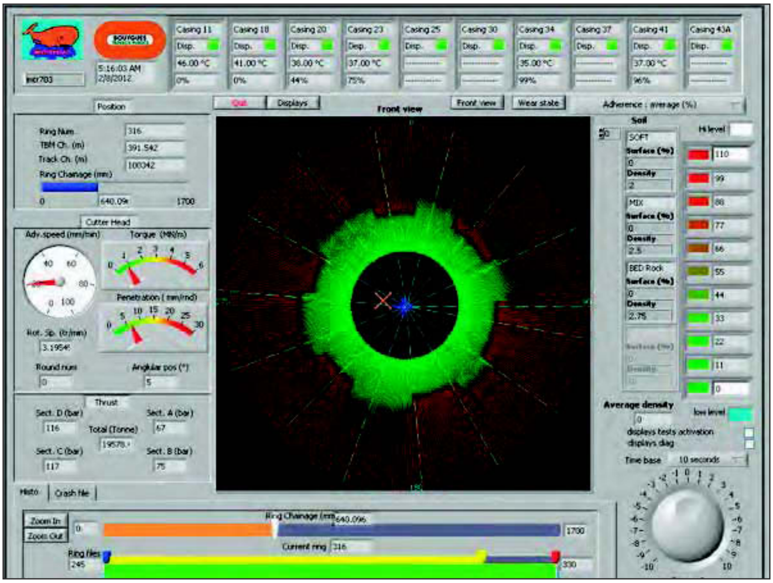

Figure 12. Only few disc cutters in contact with the boulders

Plugging Detection on Cutterhead Center

The system allows the detection of specifics situations like plugged cutter head centre (Figure 12).

Two criteria are significant:

- The discs rotation speed. The two disc cutters (11 and 18) near the center don't turn. The 20 and 25 begin to be plugged (turning intermittently).
- The discs temperature. The current temperature gradient normally shows that the temperature of the tools increases with the implantation radius on the cutter head. The peripheral tools are more solicited. Here the temperature gradient is opposite. The higher temperature (46°C) is on the disc 11.

Thanks to the system, the operator can detect in real-time such a situation on the cutter head and take as soon as possible the good decisions.

Cutter Disk Wear Status

Wear Computation

The system can compute the disc wear with a good accuracy. This is displayed clearly in the control cabin. As we can see (Figure 13) at the ring 74 the wear of the disc 41 is out of limit. It has been changed during the next maintenance operation.

The wear speed is not the same for all the discs. It depends of the physical location of the disc on the cutter head and also of the geological repartition at the front. Even if the system is computed with accuracy only the wear of the instrumented disc cutter, it gives a good idea of the wear speed repartition on the cutter head.

Disc Damage Detection

We can notice on the front view build on adherence criteria that we have a periodic adherence lost every 26 degrees of the disc 43A. 26 degree on the cutter head

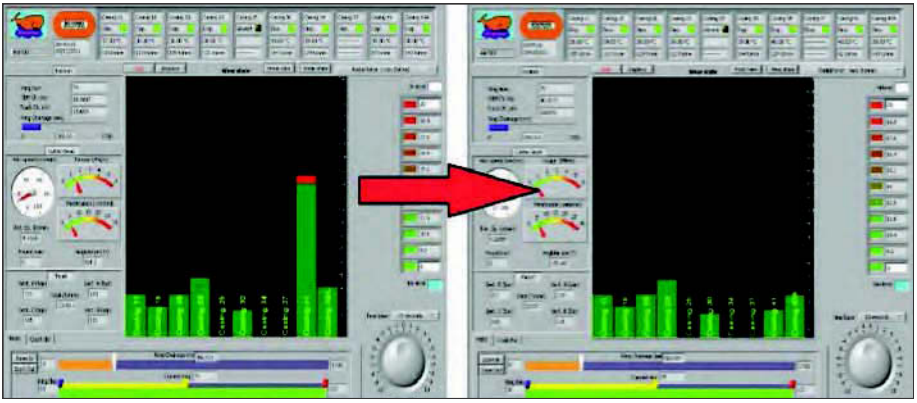

Figure 13. Tools wear follow-up and cutter change optimization

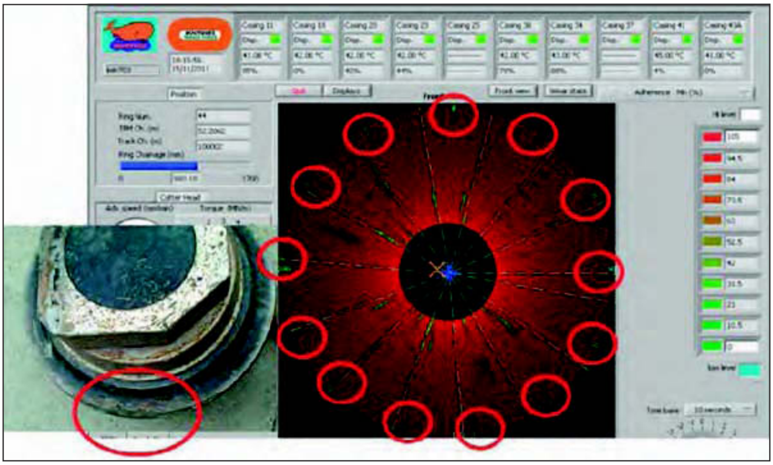

Figure 14. Only few disc cutters in contact with the boulders

corresponds for the 43A to a full revolution of the disc. The system allows detecting here a flat or chips on a disc (Figure 14).

Excavation Face Geotechnical Mapping

Mobydic produces in real time the geological mapping of the front. The accuracy of the mapping has been validated with several comparisons (Figure 15) done by the geologist of the site during front inspection. The geologist trusts the system which avoid many inspections in hyperbaric.

Thanks to the good reliability of the front mapping, it is possible to detect underground constructions. For example (Figure 16) when TBM was crossing below KSS shaft (on MTR 703 project) a change in front view was observed. Concrete has less strength as compare to granite so low radial force was observed in upper left side starting from ring no 46.

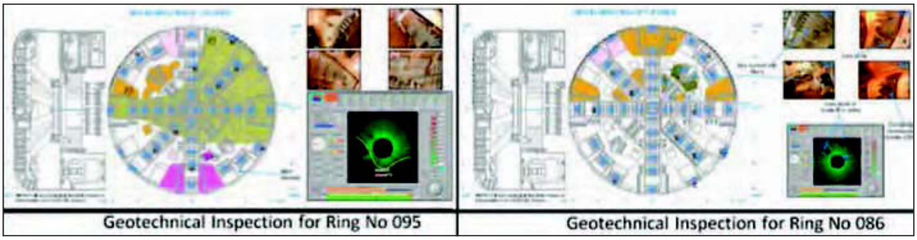

Figure 15. Comparison Mobydic front face/geologist inspection

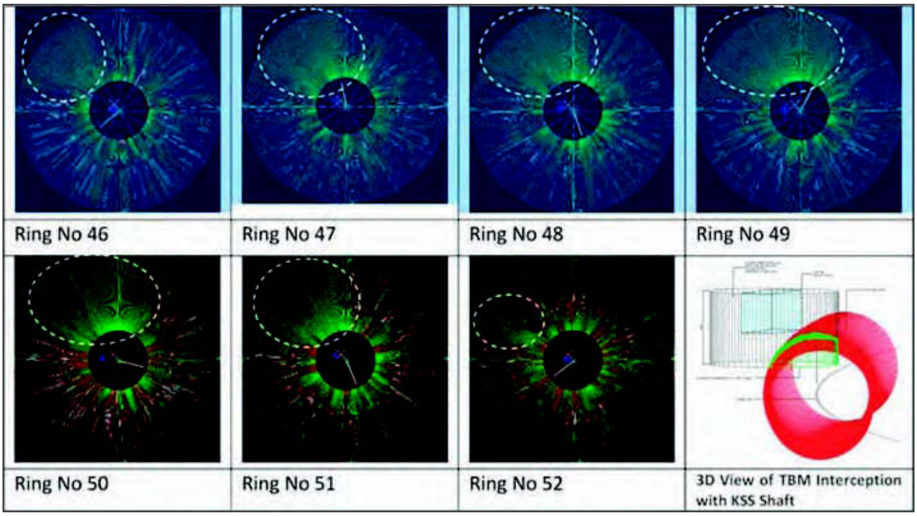

Figure 16. Visualization of KSS shaft in Mobydic

Control of Excavated Volume

From the front mapping, Mobydic is able to compute the percentage of the different materials present to front. Then the system is able to compute the average density of the excavated materials (Figure 17).

Theoretical excavated volume was calculated based on simple formula, i.e., $\pi r^2 \times L$ where "L" is the stoke length. To compare it with actual excavated volume and to calculate any over break to compare it with injected grout, two parameters were required Tonnage (Mass) & average density of face. Excavated tonnage was obtained from STP calculations whereas average density was calculated through Mobydic. Mostly the face was not homogeneous so a number of times density for different type of excavated material was calculated at on site STP lab. This calculated density along with calculated average applied radial force was used to calculate average density of face through Mobydic internal calculation. Mobydic then calculates percentage of material type and average density (Figure 18).

CONCLUSION

Tunnel Project performed with TBM have to overcome more and more stringent conditions. Therefore, having real time information about the geotechnical environment at

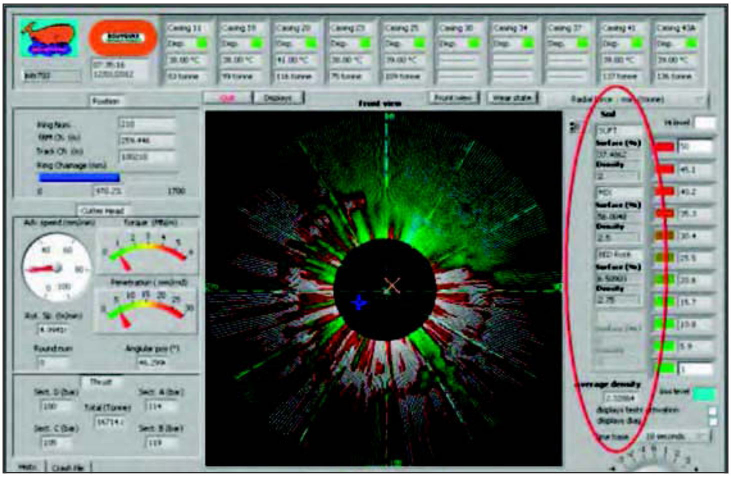

Figure 17. Computation of the average density and of the percentage of excavated material

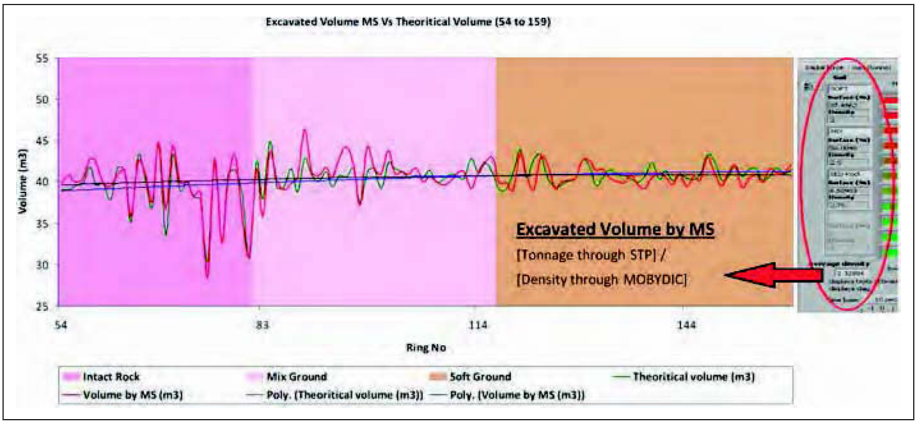

Figure 18. Only few disc cutters in contact with the boulders

excavation face and about the status of cutting tools becomes essential. It helps a lot for making the TBM progress and TBM cutting tools maintenance forecast more reliable. As it gives real time indication, Mobydic is not only a tool for doing post analysis but it is also used directly by the TBM operator for adapting the TBM progress parameters to the actual environment and then to mitigate risk of damaging cutting tools.

40° INCLINED TUNNELLING FOR PUMP STORAGE POWER PLANT WITH A TBM

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ABSTRACT

Next to fossil energy sources, renewable hydropower has the greatest importance and potential for global energy supply. The stability of the power grid can be significantly increased by extending underground pump storage power plants.

The paper highlights one of the most important hydropower projects in the Swiss Alps, the extension of the Limmern storage power plant, the storage basin of which has the highest altitude in Europe. The heart of the new pumped storage plant will be a huge central cavern. Here, four pump turbines will be installed and connected to the upper basin via two inclined pressure pipes constructed by using mechanized tunnelling technology. The tunnels are designed with a length of 1,050 meters each and a gradient of 40 degrees (85%) and will accommodate these pressure pipes. An inclined shaft Gripper machine (TBM) with a diameter of 5.2 meters was built with a number of new features not previously used on such machines. Compared with typical traffic tunnelling projects this project is located in a remote and environmentally sensitive area thus creating additional challenges with regards to access and logistics. The paper focuses on the applied technology and technical features for the 40 degrees inclined tunnelling which represents the current state of the art in TBM technology for specific project conditions such as displayed with this project.

GENERAL

In a world of growing demand for clean, reliable and affordable energy, the role of hydropower and multipurpose water infrastructure is expanding. Particularly with regard to the expansion of renewable energies, backup power plants are gaining a growing impact in terms of securing the power supply. Because electricity from solar or wind energy is not available at a constant and predictable level, both loss of production (e.g., calm) or periods of high electricity production combined with low demand (such as strong winds at night) must be balanced. For this grid control, pumped storage power plants, like compressed air storage power plants, are not only a good addition to the power plant park but constitute an important component in the integration of renewable energies into the existing supply system.

A pump storage power plant is a special type of storage power plant. It uses the potential energy of the dammed water in the upper storage basin and serves primarily to provide additional power during peak load periods. There are currently some pump storage power plant projects underway in the Swiss Alps that optimally fulfill the requirements because of their topographical conditions. The project that is highlighted is the Linthal project 2015 where a newly developed Gripper TBM is in operation for the excavation of two 1,050-meter-long pressure tunnels with an incline of 85 percent.

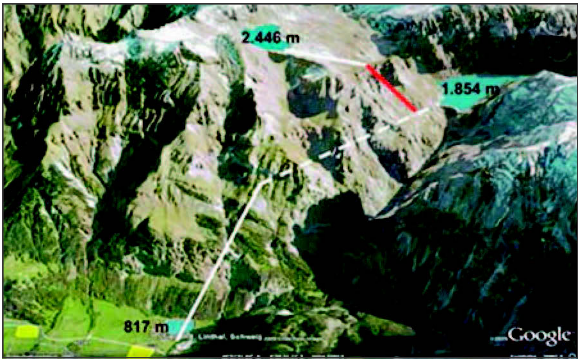

Figure 1. Aerial view of project area and 40° inclined pressure shafts marked in red

OVERVIEW OF THE LINTHAL PROJECT 2015

Between 1957 and 1968 the hydroelectric power plants Muttsee, Tierfehd and Linthal of Kraftwerke Linth-Limmern AG (KKL) had been built. They are located in the Canton of Glarus in Switzerland about 80km southeast of Zurich. The different power station stages use the water inflow from a catchment of around 140km^2 in Linth. The average electricity production from natural intakes is 460GWh per year.

The importance of the KLL for the Swiss electricity supply is far greater. As a storage power station KKL produces especially valuable peak energy when the demand is very large. They contribute decisively to the fact that the electricity consumption and electricity production can be kept in balance. As the demand for peak power increases continuously, the systems can be expanded with an additional, powerful pumped storage plant, the project Linthal 2015. A new underground pump storage power station is designed that will pump water from the lower lake Limmern (1,700m ASL) up 630 meters to the higher lake Mutt from where it will be fed through two headrace tunnels when needed for electricity production. Depending on the water levels of the two lakes, the head will range from 560–709m. The new project Linthal 2015 comprises a new dam construction, new caverns, about 5km of drill-and blasted access and water tunnels and TBM excavated headrace and tailrace tunnels which are situated mainly between two mountain lakes used as reservoirs. The new station is set to have both a 1,000MW pump capacity and a turbine output. This would bring a rise in capacity from today's 450MW to 1,450MW and ensures the future reliability of power supplies in north east and central Switzerland.

Construction of the new power plant is being carried out by the consortium ARGE Kraftwerk Limmern under the leadership of contractor Marti Tunnelbau AG. One, apart from the dam and caverns, is the excavation of the two 1,050m long pressure shafts, which are part of the core infrastructures of this major project will be the focus of this paper.

TWO PRESSURE SHAFTS TO BE EXCAVATED WITH AN INCLINE OF 40° BY GRIPPER TBM

Two pressure shafts are excavated from the powerhouse cavern at 1,700 meters above sea level with a gradient of 40° (85%) up to a service chamber in the shores of the lake Muttsee 630 meter higher. They will be lined with steel pipes to withstand the water pressure and ensure water-tightness. The headrace tunnels are essential elements of the water system of a hydropower plant. The two pressure shafts with an inner diameter of 4.2 meters are excavated in succession with the same tunnel boring machine (TBM).

The tunnels are to be built in limestone of the Quintner formation of medium to high strength with maximum predicted Unconfined Compressive Strength (UCS) values of up to 120MPa. The overburden comprises a maximum of 565 meters. The prevailing limestone is characterized to be stable and dry. However in the shear and fault zones loose rock is expected constituted by calcareous marl of lower rock strength. The geological investigations that had been performed, point the possibility of encountering karst columns and solution features. In the higher mountain areas also partially sediment filled cavities in the limestone may occur.

The gradient of 40° (85%) is a particular challenge for both the machine technology and the project team, and apart from the forecasted fault zone and the karst probability, is one of the decisive aspects for the design of the Gripper TBM.

SPECIFICS AND CHALLENGES OF THE PROJECT

The project is one of the challenging projects these days in respect of machine design but also in respect of the required logistical support throughout the project. The principal challenges are the remote Alpine location combined with the technically very demanding buildings and facilities. The sites Ochenstäfeli and Muttenalp are at 1,800 and 2,500m (ASL) and are thus highly dependent on weather conditions and through the seasons. Influences of nature and natural hazards are an issue and appropriate precautions must be taken. This results in limitations such as the interruption of work at the lake Mutt during the winter period. The length of interruption of construction is thus not determined by the client but by the winter. The provision of infrastructure for the construction sites and permanent access to the facilities, the logistics and overall organization are complex and extremely demanding. The key in this project is the interaction between all parties involved and the logistics in particular. The remote Alpine location and high altitude had been finally decisive for using cable cars to transport every single component required for construction, including TBM parts with maximum weights of up to 36 tons, because there are no access roads connecting the valley base camp at 800m above sea level to the working areas at an elevation 1,850m and 2,450m. Access to the main work sites such as power houses, caverns and pressure shafts is provided only by the in total three large ariel cable cars and a 3,000m long access tunnel. The cable cars with capacities of 40 tons are to date the largest in Europe and this for a temporary time period only.

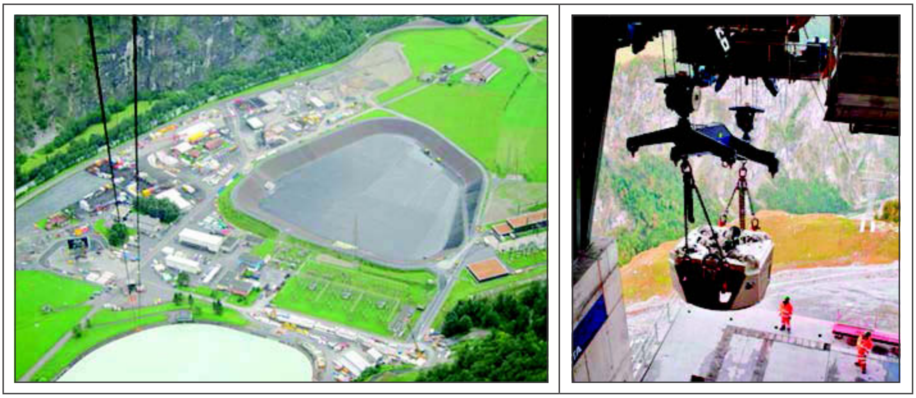

Figure 2. Access to the main work sites and transport of TBM components via cable cars

MACHINE SPECIFICS FOR TBM BORED HEADRACE TUNNELS

The excavation of the two pressure tunnels are challenging due to the steep incline of 40°. It is the first time that a large diameter machine was applied to such a steep incline. The machine in use is a 5.2m-diameter Gripper TBM which demanded some design adaptations due to 40° uphill excavation. The machine design is thus based on an innovative safety design with special attention on maintaining the safety of the personnel operating the machine.

The TBM was designed and manufactured by Herrenknecht in Germany. The contractor ARGE KWL, a joint venture of Marti AG and Toneatti AG, worked close together with the machine manufacturer Herrenknecht AG on machine design to address the challenges of machine transportation and site assembly. Before delivery to Switzerland, the Gripper TBM was fully assembled and tested at the Herrenknecht plant. Workshop testing with engineers of Marti Tunnelbau AG was completed in Schwanau on June 22, 2010, nine months after the contract was awarded. As the consequence of the close collaboration between the TBM manufacturer and contractor, the TBM was ready to go on site after only seven weeks of site assembly.

The first challenge which had to be dealt with after the machine components arrived in Tierfehd in Switzerland, which is at an elevation of 800m ASL, was the transport of the machine parts by cable car to the assembly cavern Hosenrohr in an altitude of 1,700m ASL. Therefore the components had been designed to maximum weights of 37 tons taking into consideration the maximum capacity of the cable car of 40 tons.

The Gripper TBM was assembled in a horizontal position and moved to the launch position onto a ramp of approx. 25° inclination to start then excavation for 1,050m through limestone formation up 600m to the service chamber.

The next demand was to bore the vertical curve radius of 150m directly after start of excavation with an overall machine length of 125m including trailers. The TBM and trailers had been assembled together and was positioned at an inclination of 25° for the launch of the gripper TBM. The TBM started directly from the ramp to excavate the 150m vertical curve radius for about 50m until

Figure 3. Gripper TBM (Ø5.2m) for 40° inclined drives

Figure 4. Ramp of 25° inclination–launch position for Gripper TBM

the machine reached the constant 40° incline. This launch procedure allowed the contractor to reduce the amount of drill and blast operations for the launch chamber and starter tunnel to a minimum and keep it within gradients that could be handled by standard equipment.

Figure 5. Additional gripping systems behind the TBM

For positioning of the machine at the tunnel face to start the excavation process, the TBM was moved via a steel ramp and then into the profiled start tube. Two displacement cylinders acted on the vertical support of the TBM (TBM displacement system) and two further cylinders acted on the carriage of trailer 1 (trailer displacement system). The shifting was done manually using a hand control valve installed on an external hydraulic power unit.

The steep tunnelling demanded a reliable solution to prevent the Gripper TBM slipping back. Compared to previous Gripper TBM designs with a single anti-reverse lock, this specific machine was designed with a double anti-reverse lock with full back-up redundancy of the available bracing levels for the 125 meter long TBM that weighed 800 tons. The double anti-reverse lock is located on the first back-up. The operating conditions of the TBM comprise advancement, standstill and re-gripping processes. In all operating conditions at least one anti-reverse lock is always securely clamped to the rock. There are always at least two of three locking systems independently and thus absolutely safely braced against the rock. This significantly increases safety for personnel, machine and structure in all operating stages. Any slipping back of the machine can reliably be prevented. The anti-reverse locks work mechanically on the principle of a self-locking toggle lever (automatic mechanical wedging) ensuring reliable bracing of the machine against the rock even in the event of a power failure or failure of hydraulic systems.

To start the TBM drive, the TBM was braced in the profiled start tube and the anti-reverse locks were still outside the tube. TBM advancement was realized in regular advancement mode and decoupled from the anti-reverse lock system. The trailing cylinders are thereby connected to a floating position. The advance of the trailer was done manually by means of cylinders of the trailer displacement system and the external power pack and the TBM determines the advancement speed.

The 5.2m diameter cutterhead is designed with 17-inch single disc cutters. In total the cutterhead is equipped with 24 disc cutters in the face area, 8 gauge cutters with two double-disc cutters in the center and four buckets. An overcut of 20mm is possible.

For exploratory drillings or for injection drillings there is one drill rig installed on the TBM. For rock support measures there are three anchor drilling devices installed in the L1 and L2 area and on the back-up N° 3.

The cutterhead has an electrical main drive with a power of 2,205kW. The rock chips that are excavated are taken by the buckets and transported through an opening in the bottom to a muck chute. From there the muck is channeled into invert muck chutes with the assistance of running water to an intermediate storage in the cavern at Hosenrohr. From there the muck is transported by dumper to a crushing plant. The water mixed with fine material will be collected in a sediment tank in the cavern.

The consumables are transported to the TBM working site by cable cars. On the back-up of the TBM there are locations for the consumables and intermediate storage locations for auxiliary and operating materials. During the excavation process the rock support measures are installed in the L2 area. This comprises anchor drilling,

Figure 6. Competent rock mass (left); Situation in the fault zone with loose blocks embedded in a loamy matrix (right)

Figure 7. Additional heavy rock support measures in the Mörtalbruch zone with installation of steel cassettes as segment behind the gripper shoe

placement of wire mesh and shotcreting. Mesh and anchors are also placed in the L1 area. The time necessary for installing the tunnel support is dependent on the support classes.

EXPERIENCES DURING THE FIRST TBM DRIVE

In November 2010 excavation started for the first pressure shaft. Performances of 15m to 20m per day were achieved until tunnel meter 570 where a forecasted fault zone and natural caverns filled with mud and water were encountered. This zone extended on a length of about 30 meters and in addition to the karst zone there was weak and heavily fractured rock with loose blocks embedded in a loamy matrix. The anti-reverse locks could not brace and the drilling of a pipe roof umbrella for the reinforcement ahead of the tunnel face was not possible due to non-injectable geology.

The crossing of this so called "Mörtalbruch" demanded little technical adaptation of the TBM but therefore additional heavy rock support measures which slowed down the TBM progress and led to a delay of about 6 months. The taken measures did foresee the installation of steel-cassettes as a segment behind the gripper shoe to take the thrust forces and the installation of liner plates on 360° behind the gripper shield. Cylinders were installed to push the trailer forward and to lock it in its position. In such cases of demanding and difficult conditions, the big effort done during the design of the machines, from both TBM manufacturer and contractor, for a well thought-out concept of all workplaces and material handling systems pays off to successfully overcome these conditions.

The first 1,050m long pressure tunnel was successfully excavated by October 14, 2011.

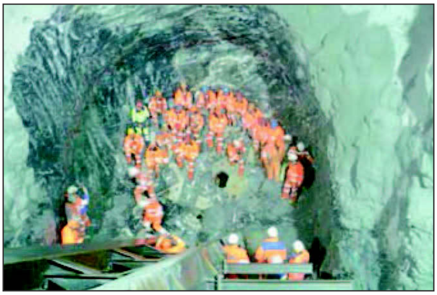

Figure 8. Breakthrough after first drive

The TBM was then prepared for being rejected. The cutterhead was partly dismantled. The four outer cutterhead segments and side supports were removed thus giving space between the TBM and the installed rock support. All services that had been required for the tunnel drive were removed and the TBM was retracted in strokes of 1,800mm. When it arrived at the bottom of the shaft, the TBM was retracted via pre-tensioned strands into the assembly cavern.

Excavation for the second pressure shaft started in March 2012. Parallel to the excavation of the second shaft, the installation of the steel lining started in the first shaft.

In May 2012, after excavating about 560 meters of the second pressure shaft, the Gripper TBM faced the predicted “Mörtalbruch” where the same measures were taken as applied on the execution of the first pressure shaft.

After the completion of the second pressure shaft, which is scheduled for the third quarter of 2012, the TBM will be lifted and successively disassembled in steps starting with the cutterhead. All components will be transported through the access tunnel Muttsee to the cable car number 2 followed by transportation through access gallery 0 to the cable car 1. From there all machine parts will be transported to Tierfehd.

CONCLUSION

The construction of the two steep inclined pressure shafts with a gradient of 85% for the Linthal 2015 project show the state of the art of TBM equipment that can safely handle challenging geological conditions such as encountered during the excavation of the two pressure shafts. Technical adaptations and additional rock support measures for the TBM drive enabled to handle the technical challenge to excavate through the Mörtalbruch where a bracing of the anti-reverse locks, necessary for steep-inclined tunnelling of 85%, was not possible. This specific project conditions were not only a challenge from the geology but also from the remote location of the construction sites in the Alpine area with the presence of natural hazards. Apart from this, the remote location demanded a safe and reliable logistics for the overall construction project.

With the successful TBM excavation of the two 85%-inclined pressure shafts, the basis is created for future hydropower projects in development in remote areas with complex specific project demands.

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DESIGN AND CONSTRUCTION OF A JACKED BOX CULVERT IN QUINCY, MASSACHUSETTS

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Steve Leius ▪ SECAC Tunnel Co. LLC

Kurt Braun ▪ L-7 Services LLC

ABSTRACT

The City of Quincy, Massachusetts as part of the Town Brook Enhancement Project (Phase 2A) is relocating the existing Town Brook Culvert to a new alignment running northeast to southwest from the intersection of Revere Road and Mechanic Street under Hancock Street to meet the existing culvert just northeast of the Concourse Street Bridge. A section of the new culvert required installation of a 140 foot long (Sta. 25+25 to Sta. 26+65), 10 foot high, by 12-foot 8-inch wide section of precast concrete box culvert located approximately 25 feet below ground surface beneath Hancock Street and multiple utility lines using trenchless jacking methods.

The design comprised estimation of expected jacking loads, box culvert section design (for construction loads & in-service loads), estimated jacking loads, thrust block design, jacking frame design, and tunnel shield design. Other design related issues included box culvert joint restraint connections to stiffen the first four box culvert sections and tunnel shield (for maintenance of line and grade), the incorporation of an anti-friction system (AFS) using mine conveyor belts, and bench scale testing performed to estimate friction factors between the box culvert and the mine belt during box culvert installation.

The paper will provide an overview of the project, a summary of the design, and actual conditions encountered and lessons learned during construction.

INTRODUCTION

The contract documents for the Town Brook Enhancement project required that a precast concrete box culvert be installed for a distance of 140 LF between STA 25+25 (jacking pit) and STA 26+65 (reciving pit) beneath Hancock Street located in downtown Quincy, Massachusetts. The support of excavation for the jacking pit and receiving pit was designed to resist the anticipated lateral soil pressures. The jacking pit shoring system consists of vertical pipe piles (soldier piles) located along the pit perimeter, interior walers and struts, and timber lagging attached to the pipe piles (Figure 1).

The box culvert configuration consisted of precast concrete box culvert units 12-foot 8-inch wide by 10-foot high outside dimensions (11-foot span/width by 8-foot rise/height interior dimensions) and 7-foot 6-inch nominal longitudinal lengths. The compressive strength of the concrete was 5,000 pounds per square inch (psi) for 28-day strength. The weight of the culvert sections were 22.25 tons per 7-foot 6-inch section. Steel reinforcement was included in the interior and exterior walls of the invert slab, top slab, and sidewalls. The joints between the culvert sections consist of a typical bell and spigot joint with 4-inch longitudinal overlap and a sealing gasket (Rubatex R-421-N) installed along the interior bearing surface. The exterior bearing surface provided a maximum flat bearing width of 7.5-inch for the top and bottom slabs and 5.5-inch for the side

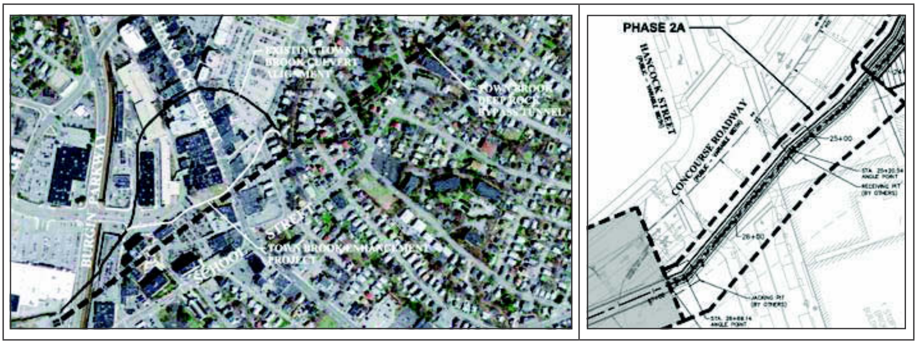

Figure 1. Site location plan

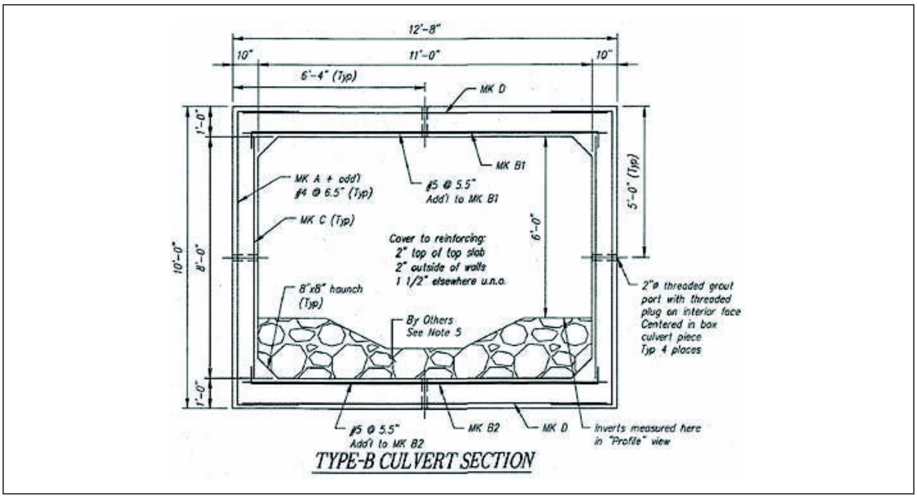

Figure 2. Geometry of the jacked box culvert [Type B]

walls. The precast concrete box was installed using trenchless jacking methods. The geometry of the jacked box culvert sections are shown on Figure 2. A plan view showing the tunnel alignment and individual box culvert sections is shown on Figure 3.

GROUND CONDITIONS

Based upon the Geotechnical Design Report the nearest borings to the proposed jacked box culvert alignment include Boring 2 located approximately 60 feet downstream of the Jacking Pit (Station 24+65), Boring 3 located approximately 20 feet upstream of the Receiving Pit (Station 27+10), and Boring 3A located approximately 15 feet downstream of the Receiving Pit (Station 26+50). Generally the ground types comprise the following (ground surface to depth):

- Fill which consists of Loam, Sand, Gravel, Brick, and Asphalt
- Loose fine Sand (encountered in Boring 3 only—1 foot thick)
- Very dense, coarse Sand, Gravel, and Cobbles (glacial till deposits)
- Very dense, fine Sand, Silt, Gravel, and Cobbles (glacial till deposits)

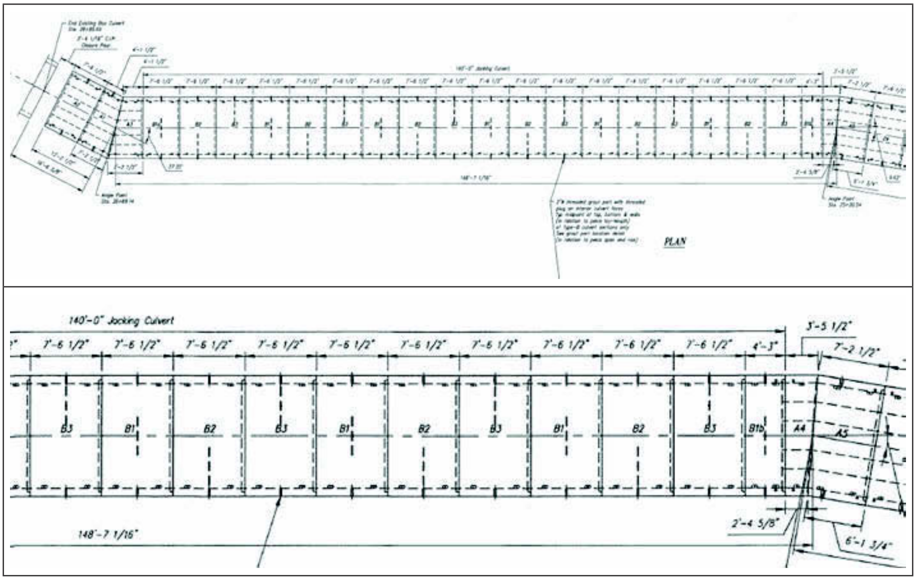

Figure 3. Plan of jacked box culvert alignment

Groundwater was not encountered in Boring 2. Groundwater was encountered at El. 11 in Boring 3A. The proposed tunnel invert ranges from El 11.3 to El. 11.8. However, it was noted that in Boring 3A possibly within the vicinity of the alignment (no location provided in Geotechnical Design Report) the groundwater was recorded at a depth of 16 feet corresponding to approximately El. 19 which is on the order of 8 feet above the box culvert invert.

Further evaluation and confirmation of the ground type, behavior, and groundwater elevation was performed during excavation of the pits and during drilling for installation of the piles associated with the jacking and receiving pit support of excavation. These excavations were generally dry with some limited seepage of groundwater into the excavation during significant rainfall events. A cross section of the tunnel alignment is shown on Figure 4.

DETAILED DESIGN

The detailed design included a jacking force estimate, evaluation of box culvert jacking capacity, evaluation of concentrated jacking force on box culvert joints, design of joint restraint connections, and design of thrust block including checking stability of jacking pit support.

Jacking Force Estimate

ASCE Standard 28-00 “Standard Practice for Direct Design of Precast Concrete Box Sections for Jacking in Trenchless Construction” dated 2001 provides some general guidance on developing jacking force estimates but states that “The resistance that has to be overcome during the jacking operation varies considerably, therefore, only ranges can be estimates.” The standard also goes on to state a variety of factors that influence the jacking force which include lubrication. Frictional jacking resistance values (resistance, psi of surface area) for various ground conditions are also provided. Assuming a “*dry loose sand*” ground type which is the nearest category that represents

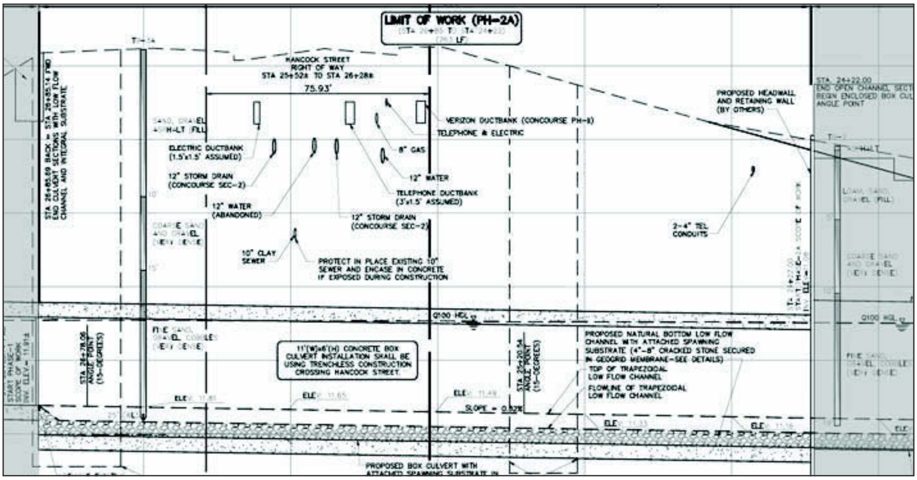

Figure 4. Cross section of the tunnel alignment

the conditions at the site the resistance is estimated to be between 3.6 and 6.5 psi of surface area (Figure 5).

Figure 5. Typical ground type during mining

The ground types based within and immediately above the tunnel envelope at the site consist of very dense coarse sand, gravel, and cobbles; and very dense, fine sand, silt, gravel, and cobbles. Using a resistance factor of 3.6 psi of surface area the estimated jacking resistance is on the order of 1,700 tons. This estimate does not take into account and effects of lubrication or use of anti-friction system (AFS) which are both intended for use on the project.

It should be noted that Najafi (2004) provides frictional resistance values (R) of 0.70 psi for clayey gravel and 1.10 psi for sandy gravel. He goes on to state that "a rule of thumb used in the industry for estimating maximum jacking force is to assume circumferential resistance of the pipe to be 1.5 psi per unit pipe or soil contact surface area" and "…lubrication can result in 30 to 50 percent reduction for clayey soils and about 20% reduction for sandy soil." Using a resistance factor of 1.5 psi the estimated jacking resistance would be on the order of 700 tons.

The jacking force estimate performed takes into account the factors influencing the value of the jacking force as presented in ASCE 28-00 (Appendix C, Article C1.2) which include length, alignment, and outside dimension of the line to be jacked; weight of box section, height of overburden, nature of soil and water table, and effect of dewatering; loads on shield, whether operation is continuous or interupted, size of overbore, and lubrication.

The jacking force estimate [F_j = F1 + F2 + F3 + F4] is based upon the sum of the following calculations:

- Drag force on top of box culvert [F1]
- Drag force on base of box culvert [F2]

- Drag force on sides of box culvert [F3]
- Face penetration force on shield [F4]

The calculations also take into account the effects of arching above the box culvert, and reduced friction coefficients based upon the planned use of an anti drag system on the top and bottom of the box culvert that will be lubricated using grease, and lubrication on the sides of the box culvert via grout ports.

Published friction factors for clean sand, silty sand-gravel mixture or hard rock fill is between 0.3 and 0.4. The design intent was to use a fully greased/lubricated steel cable reinforced mine belts during construction as the anti drag system on the top and bottom of the box culvert. A series of mine belt friction tests were performed during design to provide some estimate of the friction coefficient. Based upon the results of the field testing the calculated friction coefficients with the application of grease/lubrication had the effect of reducing the friction coefficient by approximately 70%. Based upon the mine belt coefficient testing a 70% reduction in 0.3 and 0.4 would provide friction coefficients of 0.09 and 0.12 respectively.

Considering planned means and methods assuming a friction factor of 0.15 for the top and bottom of the box culvert, a friction factor of 0.2 for the sides of the box culvert, a horizontal loading factor of 1, and a vertical load factor of 0.75 the estimated jacking force would be approximately 800 tons. For the purposes of box culvert design we recommended that a maximum jacking force of 1,200 tons was used to take into account potential variations/issues during construction.

Jacking System

The jacking force for the 140 foot long trenchless box culvert installation was estimated to be approximately 800 tons with the jacking system sized to generate a maximum of 1,200 tons. The jacking frame consisted of four 200 ton jacks located along the invert of the box culvert section with the ability/space to add an additional two 200 ton jacks if required. Although the frame has some minor wing walls to transfer thrust partly up the sidewalls, for the purposes of design, the applied jacking force was conservatively limited to only the invert slab of the culvert sections. An anti-friction system (AFS) comprised a series of 37 6-inch wide mine belts (18 on the box top and 19 on the box bottom) with injected lubrication utilized to minimize jacking forces. A simple steel cutting shield was utilized in front of the culvert sections. The shield and the first four culvert sections were to be rigidly connected together during jacking by temporary steel connections in the box culvert haunches. The shield was fabricated by the Contractor. The basic jacking frame configuration is shown on Figure 6.

Figure 6. Jacking frame

Table 1. Summary of longitudinal thrust capacity evaluation calculations

Jacking Force Application	Maximum Permissible Jacking Stress (f_p)	Allowable Jacking Force (P_{jm})	ASCE 28-00 Code Reference
Uniformly applied (entire section—4 sides)	2.55 ksi	4,053 tons	ASCE 28-00 16.1.1.1 & 16.2.1
Eccentrically applied (non-uniform, entire section—4 sides)	3.19 ksi	2,535 tons	ASCE 28-00 16.1.1.2 & 16.2.2
Eccentrically applied (non-uniform, invert slab only)	3.19 ksi	1,697 tons	ASCE 28-00 16.1.1.2 & 16.2.4

Overall Box Culvert Capacity

The box culvert sections were designed for conventional handling loads and the long term soil, water, and surcharge loads by the precast manufacturer. Our design focused on the application of the jacking forces to box culvert sections. Several evaluations were performed to assess the longitudinal thrust capacity of the typical box culvert sections. As an initial check of the capacity of the culvert to resist jacking forces (1,200 tons), the capacity formulas presented in ASCE 28-00 "*Standard Practice for Direct Design of Precast Concrete Box Sections for Jacking in Trenchless Construction*" were used. The application of jacking forces assuming uniform application (full section—4 sides), eccentric application (full section—4 sides), and eccentric application (invert slab only) were evaluated. The results of the calculations are summarized in Table 1.

It can be seem from Table 1 that the allowable jacking force exceeds the maximum jacking capacity (1,200 tons) indicating the box culvert has adequate capacity to withstand the maximum jacking forces.

In addition, the reinforced concrete sections (side wall and top/invert slab) were checked as a column following ACI 318-11 "*Building Code Requirements for Structural Concrete*" methodology. It was determined that the width of the invert slab alone will generate sufficient capacity to resist the maximum thrust capacity of the jacking frame with a live load factor of 1.6.

A detailed analysis of the bending, axial, and shear forces associated with jacking loads was performed. The evaluation utilized RISA-3D, a three-dimensional structural model, to assess various jacking force distributions on an individual culvert section. The jacking force was applied only to the invert slab to represent the concentrated jacking forces at the jacking frame. Three different load distributions were analyzed on the other (leading) edge of the culvert corresponding to upward steering, downward steering, and uniform load distribution. The maximum forces in the lining were compared to the maximum permissible forces to ensure that the box culvert sections would not be overstressed during jacking. It was found that no overstressing would occur for the three scenarios evaluated. Representative graphics of the structural model are shown in Figure 7. See also Figure 8.

Concentrated Jacking Force on Culvert Joints

At the culvert joints, the jacking forces are transferred via joint packing material along the exterior bearing surface of the box culvert bell and spigot joint. The critical force for design occurs near the jacking frame where the jacking loads are primarily applied to the invert slab only. Both the 800 ton and 1,200 ton jacking forces were checked to ensure that the bearing capacity of the concrete will not be exceeded. Additionally, the splitting tensile forces (bursting forces) were evaluated to determine if any additional tie reinforcement along the joint was needed. The induced bearing stresses and splitting

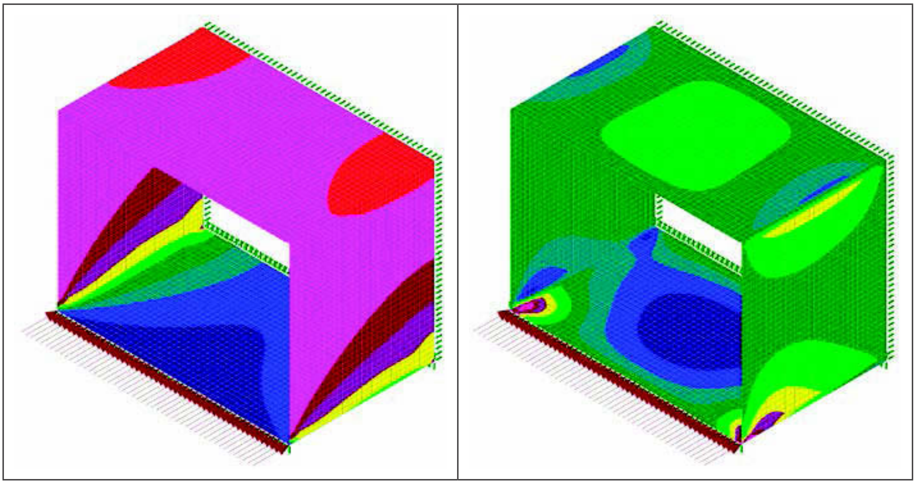

Figure 7. 3D Structural model of box culvert section (longitudinal axial and moment distributions shown)

Figure 8. Box culvert [Type B]

forces were within the capacity of the concrete and no supplemental reinforcement was required.

Joint Restraint Connections for Lead Box Culverts

The first four box culvert sections and the cutting shield were to be connected together to prevent opening of the joints and to essentially create a fixed forward shield with a length of approximately 30 feet (shield to first box joint, & three following box to box joints). It is assumed that the culvert joints will be sufficient to resist shearing forces and the only forces acting on the temporary joint restraints will be tensile forces associated with joint opening forces. The design concept was to utilize steel members anchored along each of the haunches to connect the culverts. For the ideal jacking scenario where the joints are in full compression all around the perimeter, the steel joint restraints will not be actively loaded. Tensile forces will develop when steering corrections are implemented, unusual bearing conditions are encountered, or other situations which cause the joints to open.

Since the jacking drive was a straight alignment and profile, the magnitude of these forces was expected to be relatively minor. As an estimate of the potential joint opening forces, an extreme load scenario was considered which treats the connected culvert sections as a simple beam with effective vertical supports provided by the forward shield embedded into the soil and the trailing culvert section and subjected to self-weight plus the anticipated maximum equipment load acting mid-span.

A C6 × 10.5 steel channel was selected which provided adequate tensile area for each haunch. The approach for anchoring the steel channel restraint member (C6 × 10.5) to the concrete comprises of threaded concrete anchor inserts precast into the haunches of the box culvert and secured in place using ¾ inch diameter, 6 inch long A325 bolts.

Thrust Block Design

The thrust block system was evaluated to verify adequate capacity to develop the necessary reaction forces at the jacking pit. The thrust block consists of the invert slab in the jacking pit and any additional end wall area and concrete shear key below the slab. The primary goal of the evaluation was to determine the minimum size of the reaction wall and any underlying shear keys to develop sufficient resistance to the anticipated jacking forces. Because the jacking slab and end wall were cast directly against the existing soldier pile and lagging excavation support system, the thrust block actively interacts with the support of excavation members. The evaluation assessed deformations and stresses induced into the support of excavation members to ensure that the thrust block would not overload or excessively distort the temporary support of excavation system for the jacking pit.

Calculations were performed designing the thrust block to resist the 800 ton jacking force with a minimum safety factor of 1.5. A second design criterion of a minimum safety factor of 1.3 for the 1,200 ton jacking force was selected to ensure that the thrust block would generate sufficient resistance for the maximum potential jacking force. The design approach was to assess the jacking resistance capacity of the invert slab by itself initially and then to design shear keys below the slab and/or design a vertical thrust wall along the rear of the jacking pit/jacking frame. The jacking frame design was intended to distribute the jacking force into the various steel members embedded into the concrete invert slab and fully engage the invert slab. Thus, the invert slab was evaluated as a single unit with the jacking force uniformly distributed within the slab.

Soil friction below the slab was calculated to generate a minimum of 191.9 kips of lateral resistance based on the 30° average friction angle for mass concrete cast against clean gravel listed in NAVFAC DM 7.02. Considerably more lateral resistance is generated by mobilizing the passive resistance of the soils. For the 3-foot basic slab thickness, the passive resistance was calculated to be approximately 3,000 kips for full mobilization of the passive earth pressure along the entire width of the jacking slab (24-foot). Thus, the jacking slab alone has the potential to resist the maximum jacking forces (1,600 kips anticipated/2,400 kips maximum). It was recognized that significant deformation is required to fully mobilize passive soil resistance. Per Thompson (1999) horizontal distortion on the order of H/100 is required to fully mobilize the passive soil resistance in dense sands; for the 30-foot excavation depth, this corresponds to lateral movement into the soil of approximately 3.6 inches. Recognizing that the movements of the jacking slab will directly engage the pipe piles and that large distortions of the pipe piles will not be acceptable, a more detailed calculation of the pile deflection and resistance to jacking was performed.

Since the jacking slab will be cast directly against the pipe piles, the interaction between the pipe piles and the jacking slab was specifically checked. Calculations confirmed that the pipe piles are capable of resisting the concentrated jacking forces without buckling or excessive localized deformation (pipe squatting). RISA-3D was utilized to simulate the deflections of the pile end wall due to the 800-ton jacking force and active soil pressure acting on pipe piles. Conservatively, only the end piles were assumed to directly resist the entire jacking force from the jacking slab. RISA-3D computed the resulting internal pile forces due to the deflections; Figure 9 shows representative graphics of the basic jacking slab model.

The thrust block configuration during fabrication and construction is shown on Figure 10.

The results indicated that the 800 ton jacking force applied via a 4 foot high thrust block across the entire width of the end wall would be acceptable. The maximum lateral deflection of the piles at the depth of the thrust block was calculated to be approximately 0.42 inches which was significantly less than the maximum pile deflection estimated for the support of excavation design. The maximum pile bending moment occurring at the waler location was well within the elastic allowable bending moment capacity.

Shear keys below the slab were evaluated as part of the design. Any extension of the slab depth increases the vertical height of soil engaged in passive resistance and further distributes the jacking forces acting on the piles. In addition to the thrust block, a reinforced shear key was installed to further increase the jacking resistance of the jacking slab and reduce lateral movement. It was important that the jacking frame be capable of transferring the maximum jacking loads into the invert slab and thereby, fully mobilizing the invert slab.

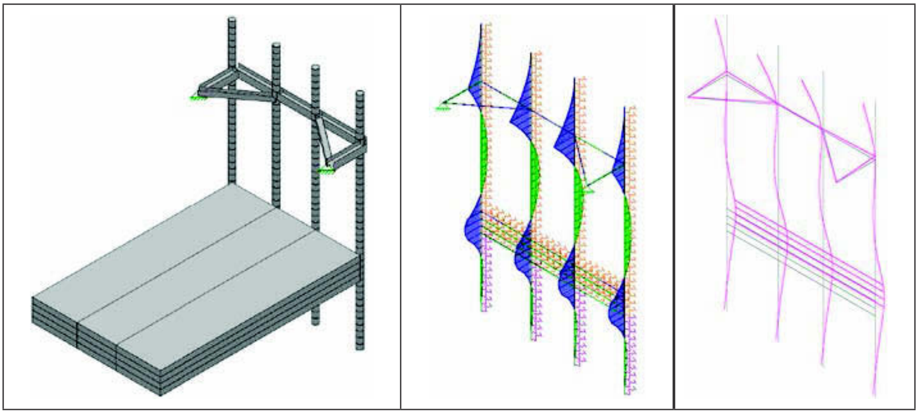

Figure 9. 3D jacking slab structural model (geometry, pile moments, and exaggerated deflections)

Figure 10. Thrust block fabrication and installation

Figure 11. Cutting shield and trailing/steering shield

CONSTRUCTION

The jacked culvert construction was performed between June and November 2012. The following are key dates in the construction process:

- Mobilization—June 12, 2012
- Shaft setup including thrust block construction—July 9, 2012
- Shield installation—August 1, 2012 Tunnel mining—August 21, 2012 to October 11, 2012
- Grouting and invert slab construction—October 19, 2012 to November 8, 2012
- Demobilization (final)—November 9, 2012.

Construction jacking and receiving pits were substantially completed prior to SECAC Tunnel site mobilization. Support of excavation was comprised of drilled pipe piles with timber lagging and a series of steel walers and struts. Care was taken during support of excavation design to ensure that sufficient clearance was provided to enable installation/placement of the jacking frame components, tunnel shield, and box culverts. Following completion of jacking pit construction the excavation of the shear key (in the pit invert) and preparation for placement of the thrust block was undertaken including placement of 6 inches of stone over the floor of the pit and installation of the prefabricated thrust frame/thrust block components. Upon completion of the thrust frame and inspection of all field welds, a final survey was performed to verify proper alignment. The frame was then encased in concrete (5ksi min) which was pumped and finished to a thickness of at least 42-inches. The cutting shield was then placed on the frame and fitted with hydraulically actuated poling plates which allow incremental advance of the shield top for increased safety of men and equipment working at the face (Figure 11). The hydraulic components including the 200T × 11' stroke jacks, power pack, and other were placed in final position and tested. Prior to installation of the trailing/steering shield and anti-friction belts, the eye was opened and the shield advanced several feet; this allowed installation of the trailing shield and the first culvert section in the shaft prior to the start of mining. As the shield was advanced, the belts were placed and anchored, the lubrication system installed, and load cells, pressure, and tilt sensors were fixed and calibrated. Tunnel mining then commenced on a two 10 hour shift at 5 days per week basis.

The ground types encountered during mining generally comprised of dense to very dense glacial till comprising coarse sand and gravel with cobbles and silt. The first

Figure 12. Completed culvert and break-through

approximately 80 LF of the tunnel could not be conventionally excavated. A transverse cutting roadheader mounted to a Brokk 180 became the primary means of excavation. The muck was transported the short distance to the shaft via a small skid steer loader and hoisted to the surface in muck cars. Several large boulders of Quincy granite up to 4 feet in size (long axis) were encountered within the glacial till during man required overmining for removal when encountered at the shield cutting edge. This approach limited impacts to design line and grade.

The anti-friction system mine belts were cut to length and then wound onto spools for unwinding during mining. Initially the belts were thread from the spool though slots in the shield and attached to a series of slots above the portal for the top of the culvert and to a series of slots cast into the invert of the floor slab for the base of the culvert. Once threaded the belts were clamped and an initial application of grease applied to the top of the first box culvert and injected via ports in the top and bottom of the shield. As mining progressed and additional box culverts installed behind the shield the mine belt strips unraveled from the spools to form a continuous anti-friction surface between the ground and the top and bottom of the box culvert. Bentonite was also injected on the sides of the culvert at regular intervals throughout the mining process. The lubricant used was a specially modified non-toxic, biodegradable, food grade, soy grease produced by Environmental Lubricants Manufacturing, Inc. (ELM) of Grundy Center, IA. Approximately 8,000 pounds of lubricant were injected during construction. Liberal use of this lubricant proved essential to the success of the project. Jacking forces were limited to a maximum of less than 500 tons. Moving loads were typically equal to startup loads except for the last 20 feet of installation (Figure 12).

Geotechnical instrumentation included conventional surveying for ground surface settlement and support of excavation movement, load cells to measure tensile stress on the mine belts, and hydraulic pressure gauges for monitoring jacking force.

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JACKED BOX TUNNEL UNDER A RAILWAY EMBANKMENT

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ABSTRACT

The Airport Link is a $4.8 billion project involving 15 km of tunnels and roads to relieve traffic congestion in Brisbane, Australia. The Jacked Boxes form a key component of the project where they allow the new motorway to pass beneath six busy railway lines. The Toombul junction site had a unique combination of ground conditions and technical challenges through which no similar scale jacked box operation had previously been attempted. The boxes measure a combined width of 38m, length of 65m and are one of the largest jacked box operations in the world. To enable the jacking of the tunnel, a wide range of ground improvement techniques were used during the works including jet grouting, canopy tubes, piling, geonails and tube á manchette grouting. The challenges that were overcome include soft clay ground conditions, 24/7 operating railway, low cover, man-made obstructions, strict settlement criteria and flooding from an adjacent watercourse. The boxes were successfully jacked into position during a continuous operation in 2011, with no disruption to the rail services.

INTRODUCTION

Airport Link Project

The Airport Link Project is the largest road infrastructure and one of the most complex engineering feats ever completed in Australia. It was built concurrently with the Airport Roundabout Upgrade and Northern Busway (Windsor to Kedron) projects, to deliver a A$4.8 billion investment in new transport infrastructure to relieve traffic congestion in the northern suburbs of Brisbane. The project was awarded to BrisConnections consortium in 2008 as a public private partnership (PPP) and Thiess John Holland Joint Venture (TJH) was engaged as the design and construct contractor.

The Airport Link toll road travels mainly underground and is the first major motorway to link the CBD to the airport precinct. The Airport Roundabout Upgrade project replaced an old signalized roundabout with a 750m elevated flyover and a high capacity fast diamond intersection. The project comprised approximately 15km of tunnels with connecting ramps, surface roadworks and elevated structures through densely populated inner-city suburbs. A combination of tunnel boring machines (TBM), mined roadheader and, cut and cover tunneling techniques were used on the project. Together they form the most sophisticated road tunnel network in Australia.

Toombul Jacked Box

The project was faced with the unique challenge of constructing a section of tunnel through a railway embankment directly beneath Queensland's busiest operating railway line. The jacked box construction method was an innovative solution put forward during tender stage, to enable the new seven lane motorway to be constructed without disrupting the railway operations. The A$75 million Toombul jacked boxes proved to be the largest of its type undertaken in Australia, and one of the largest ever worldwide.

For the jacked box operation, Benaim China was appointed as the temporary works designer and Arup London led the design peer review. The TJH team was augmented by VSL Australia as the specialist jacking sub-contractor and Keller Ground Engineering as the ground improvement sub-contractor.

The jacked boxes were two huge concrete structures that measured 65m long, 12.5m high and with a combined width of 38m. They were constructed in a launch pit beside the railway line, and then hydraulic jacks and steel strands were used to push and pull the boxes through the railway embankment. Before jacking could commence ground improvement works comprising canopy tubes, soil nailing and grouting were undertaken to stabilize the embankment. As the boxes were progressively jacked forward, mini excavators sitting inside the box were used to excavate 27,000 cubic meters of soil and rock until the boxes were shifted a distance of 55m into their final position. To monitor stability of the rail embankment, a sophisticated laser level system was used 24 hours a day, 7 days a week to detect any track movements.

The jacked boxes were successfully installed in June 2011 without any disruption to the rail operations.

SITE CHARACTERISTICS

Site Layout

The existing railway embankment was constructed in various stages over the past 120 years, and is built four meters above the surrounding Kedron Brook floodplain. The embankment is underlain by complex ground conditions that range from very soft estuarine clay to weak siltstone bedrock. The tunnel alignment intersects the railway line at a 30 degree skew resulting in a wider face and complicating the box, shield and jacking arrangement. The bases of the boxes are located approximately 13m below ground level with a 3.5% grade falling to the west (Figure 1).

Geotechnical Conditions

The project site is located within the Kedron Brook floodplain and is underlain by the Aspley Tingalpa formation which comprises a complex sequence of strata as follows:

- Fill, consisting of highly variable material to form the railway embankment. This material had been placed over a 120 year period and comprises generally firm to stiff silty clay, old ballast, ash, gravel and dumped rock that forms a bridging layer at the original ground level.
- Alluvial or estuarine soils, consisting of very soft to firm silty clays with an intermediate sandy layer in places. The clays and normally to very slightly

Figure 1. Site layout showing jackbox in relation to railway embankment

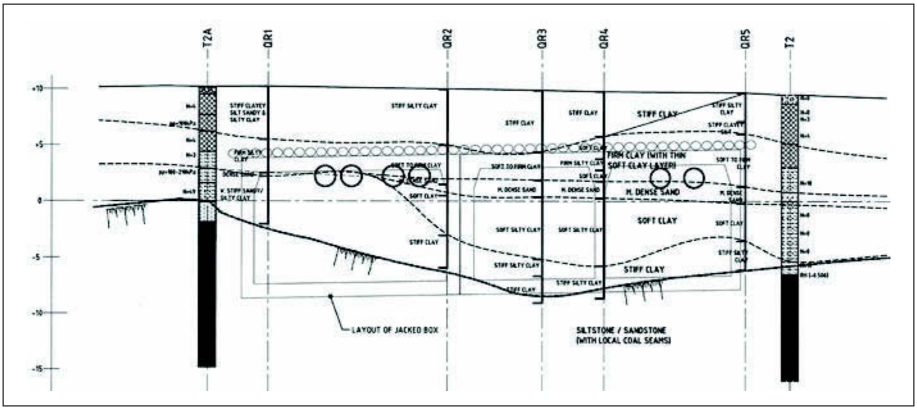

Figure 2. Tunnel section showing variable ground conditions in the face

over-consolidated and the sands were medium dense and fully saturated with a likely connectivity to the nearby creek.

- Stiff residual clay overlying weathered rock. The rock was encountered at up to 17m depth and was a weak interbedded siltstone and mudstone with coal seams.

All of these strata varied in thickness in both the longitudinal and transverse directions along the tunnel, resulting in mixed face conditions being experience throughout the jackbox alignment, refer Figure 2.

Technical Challenges

The Toombul site posed a number of unique technical challenges that included:

- Working adjacent to a live railway with over 380 train movements per day that could not be disrupted,
- Excavation within a flood plain that experienced some of Brisbane's worst flooding in the last 30 years during the construction period,
- Working adjacent to a local community that was highly sensitive to noise, dust and vibration impacts,
- Manmade obstructions within the jackbox envelope including four × 1650mm dia reinforced concrete pipe (RCP) and two × 1800mm dia RCP drainage culverts,
- Dealing with unknown obstructions that were gradually encountered during the jacked box installation such as:
  – Timber pile foundations and headstocks from an old timber trestle bridge that was left in place during an earlier embankment upgrade,
  – Large reinforced concrete abutment walls,
  – Disused sections of railway track buried in the embankment fill,
  – Large boulders placed as a soft ground bridging layer.

DESCRIPTION OF DESIGN SOLUTION

At tender stage, a number of traditional methods of tunnelling were considered such as top-down or bridge slide construction. However these options required removal and replacement of the railway formation resulting in up to 30 weekend track closures and significant disruption to passenger and freight rail services.

The jacked box method of construction was proposed as a cost effective solution to minimize disruption to the rail network. Initially a four box solution was proposed with two boxes launched from each side of the railway to keep the jacking forces down to a comparable level of other jacked tunnels that had been completed elsewhere in the world. However this solution required an in situ stitch beneath the railway which was deemed too risky, and site constraints associated with the adjacent TBM operation led the team to select a two box solution, launched from the east. This arrangement resulted in box dimensions and jacking forces that were unprecedented and required a number of engineering innovations to ensure the jacking operation was a success.

Figure 3. Soft ground clay conditions

Design Principle

The key design principle to ensure integrity of the railway embankment during jacking was to maintain face stability. The conventional approach was to reinforce the face with soil nails, however an adequate factor of safety against slip circle failure could not be achieved unless the strength of the soft clay was also increased to c_u>40kPa. The design team developed a technical solution using facture grouting that induced excess pore pressures to consolidate the soft clay, with a subsequent increase in the undrained shear strength as the pore pressure dissipated. This increase in strength also improved the bond stress around the soil nails and improved the stiffness of the clay by inducing a network of grout fracture patterns. The increase in stiffness assisted with reducing the surface movements (Figure 3).

This innovative approach had not been previously used before and extensive site trials were required to demonstrate the effectiveness of the design. The result of these trials is the development of the geonail ground improvement technique which was the key support element in the jackbox design.

General Arrangement

The general arrangement comprised a launch pit and receival pit east and west of the railway embankment. The pits were approx. 15m deep and supported by anchored diaphragm walls along each side, a piled headwall on the launch box, and a grout block on the reception side. The two boxes were cast side by side within the launch pit on top of a 1.5m deep jacking raft. Extensive ground improvement works were required to facilitate the jacked box operation as detailed in Figure 4.

Ground Improvement Works

Contiguous Bored Pile Headwall

A temporary piled headwall was constructed to support the 15m deep launch pit excavation immediately adjacent to the railway line. The wall consisted of 900mm dia contiguous bored piles at 1.4m centers, and socketed into the underlying bedrock.

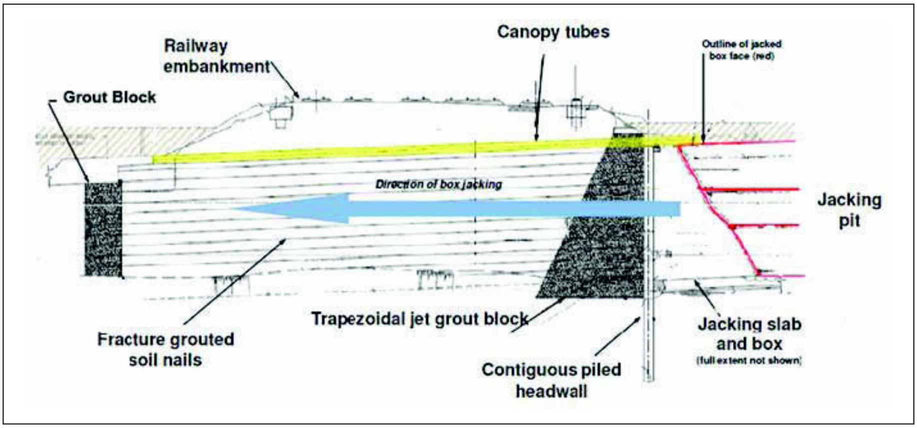

Figure 4. General arrangement of temporary work and ground improvement elements

Jet Grouting

A jet grout block was constructed immediately behind the headwall to act as a gravity type retaining wall, to support the face once the piles had been demolished. The block was 3m wide at the top, and comprised a number of interlocking grout and soil mixed columns in a trapezoidal shape. This block also reduced the earth pressures acting on the headwall, which enabled the piles to act in cantilever over their full length and eliminated two rows of ground anchors and walers that would have otherwise impeded the jackbox operation.

Canopy Tubes

The canopy tube structure comprised 48 interlocking steel tubes installed above the soffit of the jacked boxes. These tubes were 760mm diameter, 65m long and extended over the full length and width of the jacked boxes. This structure formed a complete canopy that provided separation of the railway embankment and jacked boxes. The primary function of the canopy tubes was to control localized face loss and settlement in advance of the face excavation by redistributing the embankment load onto the box roof. It also acted as an anti-friction surface to prevent the boxes 'dragging' the railway fill embankment during the jacked box installation. The tubes were tied into the headwall and reception pit roof structures to provide additional horizontal restraint to resist these drag forces.

Geonails

The geonails consisted of a tube á manchette (TAM) grouting tube combined with fiberglass rods to provide a combined soil nailing and fracture grouting support element. The soil nailing provided traditional pull-out resistance and reinforcement of the face, whilst the fracture grouting of the soft clay strengthened the weak soil properties through consolidation. Whilst soil nailing and fracture grouting are both proven methods, this is the first time they have been combined together form the geonail ground improvement technique. The geonails were also used for permeation grouting of the saturated sand layers, to minimize groundwater inflows and prevent a hydraulic connection with the nearby creek.

In total over 17,480m of geonails were installed and 346M. tonnes of cement grout were injected. To accelerate the pore water pressure dissipation, over 220m of

Table 1. Jacked box statistics

Details	Jacked Box 1	Jacked Box 2
Dimensions (L × W × H)	65.7m × 21.40m × 12.52m	61.3m × 16.70m 12.52m
Concrete	4,702 cubic meters	3309 cubic meters
Reinforcement steel	1,830 tonnes	1,470 tonnes
Jacking capacity	26,000 tonnes force	22,000 tonnes force
Weight of box	11,500 tonnes	8,100 tonnes
No. of days to install box	17 days	19 days

horizontal drains were installed. This allowed the soft soils to rapidly consolidate and gain strength after the fracture grouting,

Grout Wall

A low strength grout wall was installed immediately west of the railway line to form one side of the reception pit. This wall acted as both a groundwater cut-off wall and provided end anchorage for the geonails installed through from the east. The combined action of the low strength grout and nail tendons were able to maintain slope stability of the western face, enabling the reception pit to be partially excavated without the need for another expensive piled headwall.

Jacked Boxes

The jacked boxes were designed to form the permanent tunnel works beneath the railway embankment. In addition, they had to be specifically designed to resist the substantial jacking forces with sufficient redundancy to ensure the structural integrity would satisfy the 100 year design life. The boxes were cast side by side in the launch pit within very tight survey limits (±10mm), to ensure that the projected final box position was within tolerance. The box statistics are provided in Table 1.

To accelerate the box construction, a number of special construction techniques were used, including:

- steel reinforcement in the walls was pre-fabricated and lowered into place to minimize on-site fixing,
- the concrete forms were designed with single sided support to maximize the available working space,
- fixed form vibrators were used to enable the walls to be poured in 10m high panels.

Jacking Raft

The jacking raft was a 1200mm thick ground bearing slab cast in the base of the launch pit (refer Figure 5). It was designed to transfer the enormous jacking loads into both the underlying rock and the diaphragm walls around the perimeter of the launch pit. The jacking strand dead ends were cast into the front of the slab at the headwall, and a detailed arrangement of shear post recesses were cast into the slab to enable the very large push and pull forces to be distributed efficiently. 300mm high guide walls were cast into the top of the slab running along the outside edge of the boxes to ensure they remained on the correct alignment during installation.

Anti-Friction System

The anti-friction system was installed to reduce the jacking loads to within the capacity of the installed jacking force. This element was critical to ensure that the boxes did

Figure 5. Jacking raft during construction showing shear pin recesses

not get stuck halfway through the jacking process. The key components of this system included:

- a greased filled, double layer of polyethylene sheeting placed beneath the boxes to provide an initial bond break between the boxes and the jacking raft.
- a strip of anti-friction Matrox sheeting (similar to Teflon) was installed between the two boxes to provide a slippery surface for the boxes to slide alongside one another as they were jacked.
- a bentonite slurry injection system on all four sides of each box (top, base and both sides) to lubricate the interface between the external concrete face and the surrounding ground.

Mining Shield

Maintaining face stability during the jacking process was fundamental to the integrity of the railway embankment ahead of the box. To achieve this, a number of modifications were made to the front face of the boxes including:

- Angling the front face of the box back at 60 degrees to improve stability of the excavation face, refer Figure 6.
- Constructing temporary internal walls and platforms to compartmentalize the face into approximately 3m × 3m cells, refer Figure 7. This significantly reduced the effective span of the exposed face.
- Incorporate a steel mining shield into the leading edge of the internal and perimeter walls, to provide a cutting edge that could be embedded into the soil in advance of the box front face.
- Provision of moveable platforms and steel breasting boards within each cell, that could be extended up to one meter to locally stabilize any face instability or unraveling.

CONSTRUCTION AND JACKING OPERATION

Program

The jacked box operation spanned over two years of meticulous planning, design and construction, culminating in continuous jacking over a 36 day period in mid-2011,

Figure 6. Mining shield

Figure 7. Internal compartmentalized cells

working seven days a week, 24 hours a day. The boxes were completed two weeks ahead of program and in-line with the re-forecasted budget. Key project milestones are summarized in Table 2.

Table 2. Key program milestones

Date	Milestone
Apr 2009	Construction start
July 2009	Piled headwall
July–Nov 2009	Jet grout block
Jan–Jun 2010	Canopy tubes
Nov–Dec 2010	Geonail grouting trials
May–Dec 2010	Excavate launch pit
May–Dec 2010	Install geonails and grouting
Dec 2010	Jacking slab
Jan–Mar 2011	Jacked box construction
Apr 2011	Trial push
May–Jun 2011	Box jacking

Safety

Throughout construction, safety was paramount and the project team implemented a process of risk identification and mitigation to manage health and safety issues such as confined space entry, crane interaction and railway settlement. This process paid off with over 436,000 man-hours worked with no lost time injuries or disruptions to train operations.

Jacking Arrangement

Jacked box methods traditionally employ either a pushing or pulling technique to advance the boxes. The magnitude of the required jacking force for these boxes was such that geometrically there was simply not enough structure width to accommodate the jacking equipment for one method. In conjunction with the specialist jacking subcontractor, a hybrid solution was developed that employed a push and pull mechanism to advance the boxes.

This system used a series of 1000 tonne push jacks (SPE 1000/200 stroke solid ram jacks) and 750 tonne pull jacks (SLU 580/550mm heavy lifting jacks) mounted on the tail of the jackbox. The pull tendons each comprised 55 no. × 15.2 mm dia steel strands with the dead end cast into the jacking raft at the box front.

The larger box JB1 required 16 pull and 14 push jacks for a total jacking capacity of 26,000 tonnes. The smaller box JB2 required 12 pull and 14 push jacks for a total jacking capacity of 22,000 tonnes.

Jacking Progress

The boxes were jacked through the embankment in a continuous operation achieving an average progress of 1.5m per day with a maximum 24 hour advance rate of 4.0m achieved. The box progress is presented in Figure 8.

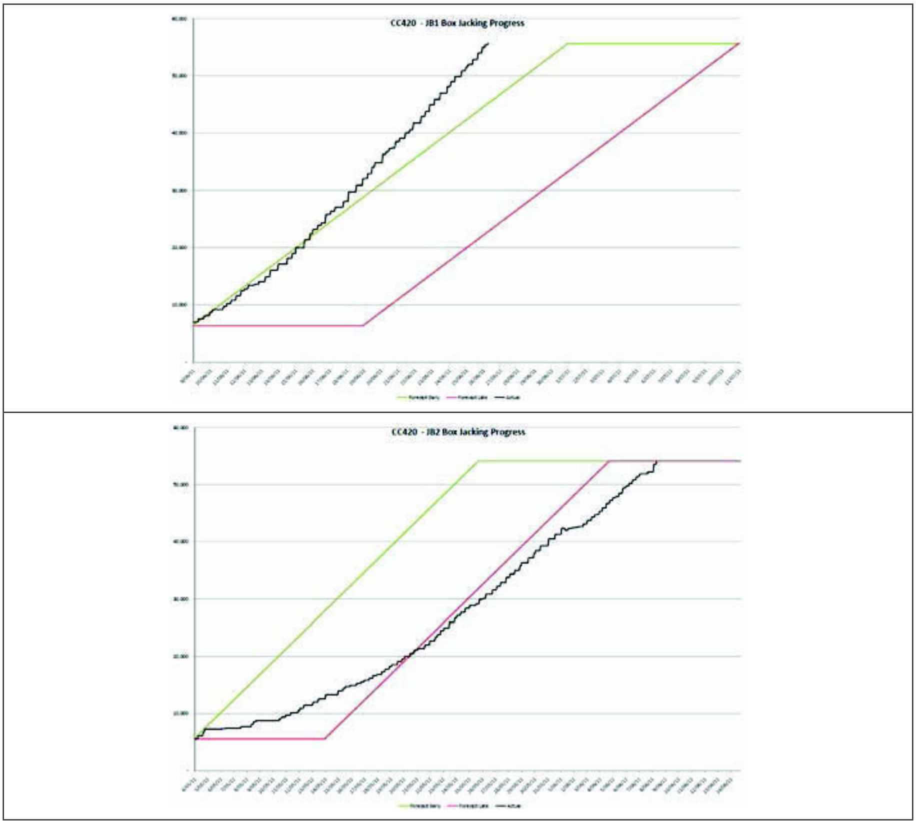

Figure 8. Graphs showing jacking progress (chainage vs. time)

Prior to jacking, the designers produced a plot of the predicted jacking force versus chainage and an estimate of when the push jacks would need to be engaged. During the push, the actual jacking force was then recorded and compared against the design prediction to verify the jacking operation was performing within acceptable limits. As can be seen in Figure 9, the actual jacking force closely followed the predicted jacking force.

Face Excavation

As the boxes were jacked a fleet of 10 mini excavators, each working in one of the face compartments, excavated over 27,000 m^3 of material. The small excavators worked off moveable platforms that could be retracted from the face to allow the excavated clay and rock to drop to the bottom level where it could be mucked out and removed to spoil.

Obstruction Removal

One of the main challenges experienced during the jacking was removal of man-made obstructions that were encountered within the railway embankment. A detailed spatial map of the obstruction locations was recorded using the geonail and canopy tube drilling installation records. This allowed the appropriate attachments for the excavators to be fitted in advance, including tools such as shears, hydraulic breakers and milling

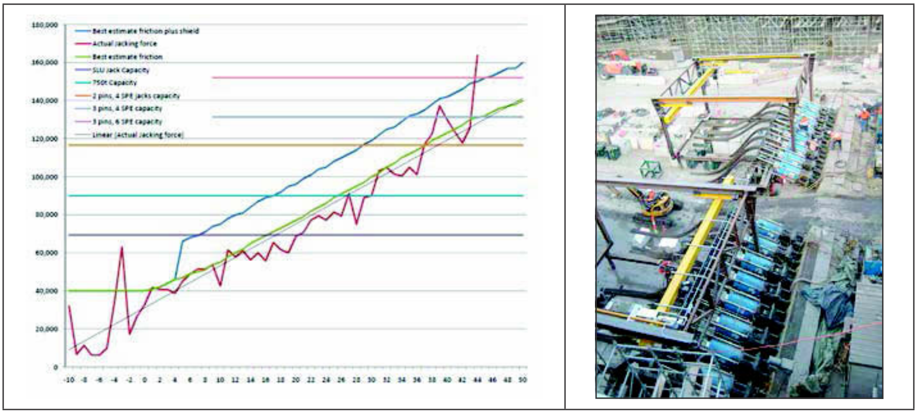

Figure 9. Plot of actual jacking force compared with predicted jacking force

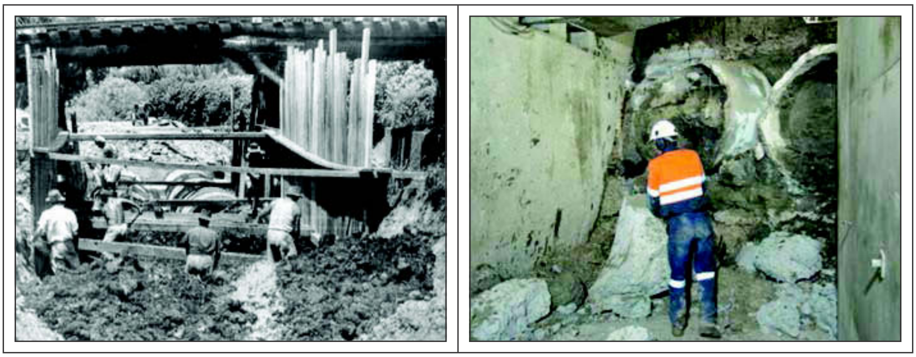

Figure 10. Concrete culverts installed in 1953 and demolition of pipes during jacking operation

heads that could efficiently remove the obstructions within minimal delay to the jacking operation.

During demolition of the culverts, the team discovered the concrete pipe surround ad been reinforced with disused railway track. The hydraulic breakers were found to be ineffective therefore an alternative solution using high pressure gas expansion known as the Cardox CO_2 system was used. This process was similar to blasting without the risks of handling explosives within a densely populated urban environment. The resulting pressure shock wave split the reinforced concrete to enable easier excavation (Figure 10).

Instrumentation and Monitoring

A sophisticated instrumentation and monitoring system was installed, to provide real-time monitoring of the railway tracks. An array of over 200 prisms was installed on the railway, sleepers and track formation and monitored using an automated survey total station. Any track movements were reported in realtime via text message or email alert notifications to TJH and Queensland Rail personnel.

Other monitoring instrumentation and testing undertaken include:

- vibrating wire piezometers to measure pore pressures in the ground during the fracture grouting operation and to record groundwater table levels.
- vertical inclinometers in the piled headwall and diaphragm walls to measure lateral movements.
- horizontal inclinometers installed inside the canopy tube structure to record vertical ground movements above the box.
- Insitu shear vane and horizontal static cone penetration testing (CPT) to verify the ground improvement of the geonails and fracture grouting.

Throughout the jacking operation, an average of 115mm total settlement occurred across the railway embankment. This movement occurred gradually and was managed in conjunction with Queensland Rail via a regime of ballast tamping and re-leveling at key stages of construction. The end result was no delays or disruption to train operations.

Detailed positional survey was undertaken during each jacking cycle to confirm the boxes remained on the correct horizontal and vertical alignment. The final position of the boxes was well within the ±150mm tolerance, achieving a remarkable 50mm horizontal and –60mm vertical positioning after being jacked over a distance of 55m.

CONCLUSION

The Toombul jacked boxes are a demonstration of engineering innovation combining with construction capability, to overcome a number of unique technical challenges. The project demonstrated that the jacked box method of tunnel construction can be adapted to complex ground conditions and undertaken on a scale not previously contemplated. The success of the method has been confirmed through the installation of the jacked boxes to within tight construction tolerances and with no disruption to the railway operations.

ACKNOWLEDGMENTS

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PART 13

New and Innovative Technologies—II

Chairs

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Traylor

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URS Corporation

ACCURATE SOFT GROUND TBM DOCKING—SURVEYING, TBM ABANDONMENT, AND DEFORMATION

Joe Clare ▪ MWH Americas, Inc.

Pete DeKrom ▪ Atkins North America, Inc.

ABSTRACT

The planning and execution of a soft ground tunnel connection 91 m (300 ft) below a residential neighborhood required intricate planning, accurate and confident tunnel survey as well as TBM guidance control. TBM abandonment also necessitated deformation analysis and prediction to establish a sequence for stripping of the TBM interior. TBM abandonment required resistance to high pressure groundwater, drilling and ground freezing loads. Paper presents a review of technical issues and lessons learned from a challenging and risky endeavor.

INTRODUCTION

King County's Wastewater Treatment Division serves about 1.5 million people in the central Puget Sound region of Washington State which comprises the urban areas of King, southern Snohomish, and northern Pierce Counties. The regional system includes two large treatment plants with one located in Seattle and the other located south of Seattle in Renton. Without expansion, the existing wastewater conveyance and treatment systems would reach capacity and likely lead to discharge violations. Such an event would have the potential to stifle regional growth particularly in the northern King and southern Snohomish County areas. King County planned the new Brightwater Wastewater Treatment Plant to be located north of Woodinville, WA to provide the increased capacity and infrastructure needed to support anticipated growth in the region. (Clare, 2011, Johnson 2007).

Design and construction of the Brightwater project is led by King County Department of Natural Resources and Parks, Wastewater Treatment Division. The Brightwater treatment plant was completed in 2012 and provides initial secondary treatment capacity using membrane bioreactor technology for 136 ML/d (36 million gallons per day (mgd)) with expansion planned for 204 ML/d (54 mgd) by 2040. In addition to the treatment plant, a new 645 ML/d (170 mgd) influent pump station was completed in 2012.

The Brightwater conveyance system connects the new Brightwater wastewater treatment plant to a new marine outfall into Puget Sound. The conveyance system included 21 km (13 mi) of concrete segmental lined tunnel with inside diameters (ID) between 4 and 5.5 m (13 to 18 ft) constructed from 2006 to 2012. The variable topography of the project area required dividing the alignment into four tunnel drives, BT-1 through BT-4, terminating at shafts up to 60 m (205 ft) deep that served as launch and reception points for the four TBMs (Figure 1) (Clare 2011). Geological conditions were characterized by glacial and non-glacial deposits from several glaciation events. The largest overburden thickness of about 134-m (440-ft) was within the 6,100-m-(20,000-ft)-long Brightwater Tunnel 3 (BT-3), where the potential maximum hydrostatic head was up to 7.3 bar. Excavation of this tunnel started with a Mixshield™ slurry TBM advancing westerly from the North Kenmore Portal towards the Ballinger Way portal.

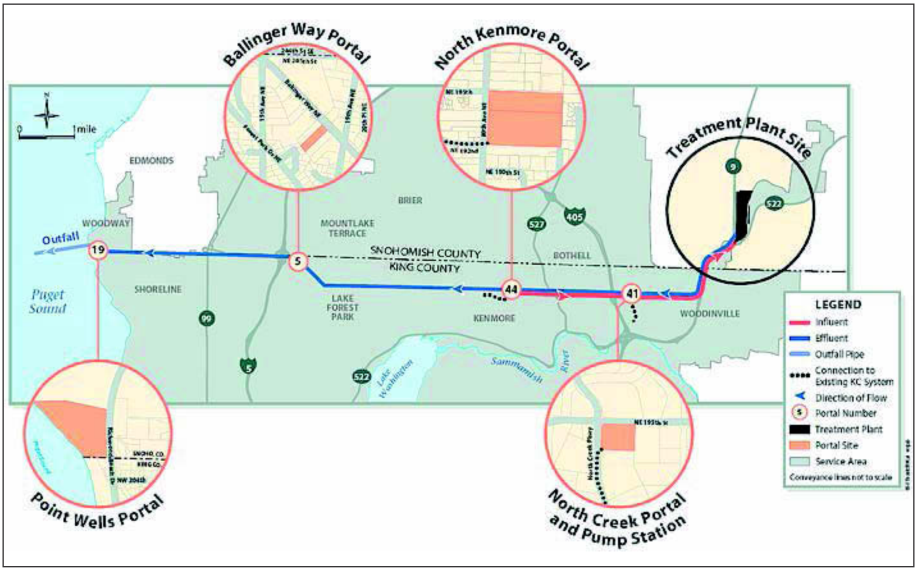

Figure 1. Brightwater conveyance alignment

This deep shaft also served as the reception point for the eastbound earth pressure balance (EPB) TBM of Brightwater Tunnel 4 (BT-4) (Frank et al. 2010, Shinouda et al. 2011). After about half the BT-3 tunnel had been mined, inspection of the TBM cutterhead identified significant damage to the rimbar, and as a result, tunneling was stopped. (Gwildis 2012).

Due to the potential for major delays in completing the project, the county decided to hire a replacement tunnel contractor. In early 2010, King County contracted with Jay Dee Coluccio Joint Venture to complete the remaining western half of the BT-3 tunnel, now identified as BT-3C (Hauser 2012). The BT-3C tunnel extended from the Ballinger Way portal eastbound to meet the abandoned shield of the BT-3 TBM located under street right-of-way in a residential area of Lake Forest Park, Washington. The street was less than 20 m (60 ft) wide, with the tunnel alignment crossing at nearly a right angle. At this location, the tunnel was at a 95 m (310-ft) depth with ground conditions dominated by glacially overconsolidated, layers and lenses of clay, silt and sand. The hydrostatic head was more than 5 bar at the tunnel invert elevation. Under these conditions, direct access to the TBM from the ground surface was not feasible. As a result, it was necessary to develop a plan for connecting the two tunnels underground and about 3 km (10,000 ft) away from the nearest portal.

TUNNEL CONNECTION

Following evaluations of several alternatives, the project team recommended a head-to-head connection of the tunnels in which the BT-3 TBM would be abandoned and dismantled and the smaller diameter BT-3C TBM would mine into the larger diameter shield (Figure 2). Ground freezing was necessary to allow safe conditions for removal of the cutterhead of the abandoned TBM and to allow the BT-3C TBM to mine into the abandoned shield. Following dismantling of both TBMs a shotcrete final lining was placed against the connected shields. Challenges included:

- Dismantling of the TBMs
- Ground freeze installation

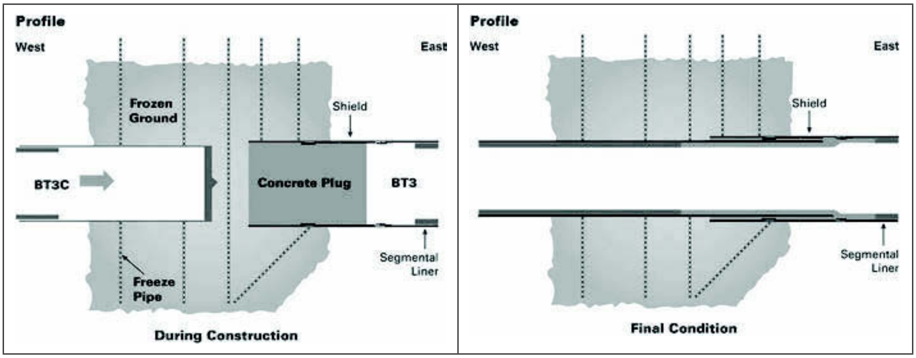

Figure 2. Schematic of head-to-head tunnel connection alternative

- Deformation of the TBM shield during dismantling
- High-order tunnel survey control to achieve on-target hole through

The project team recognized the risk that pressurized sand layers would flow in an unsupported condition with a potential that the tunnel would be inundated. After evaluating several alternatives for ground improvement, the team determined that ground freezing from the surface was the preferred approach (Gwildis 2012).

GROUND FREEZING

Ground freezing was utilized to provide stable face conditions to allow dismantling of the BT-3 TBM and cutterhead at atmospheric conditions. Vertical freeze pipes were installed to a depth of about 100 m (330 ft) in front of and around the sides and top of the abandoned TBM. Installation of the freeze pipes was initially executed by mud rotary; however, excessive loss of slurry into coarse grained soils was experienced with this method. In addition, a full bore hole of drilling mud increased the pressure at the TBM from 5.2 bar to over 12 bar and pushed the TBM shield back approximately 76 mm (3-in). Drillers switched to cased air rotary drilling methods to complete the remaining freeze pipe installation to avoid further impacts to the BT-3 TBM below. A total of 36 zone freeze pipes and 5 temperature monitoring pipes were installed from the ground surface. Pipe sections within the path of the incoming BT-3C TBM were made of aluminum to allow the TBM to mine though the softer metal. To freeze the soils beneath the BT-3 TBM, four 6 m (20 ft) long freeze pipes were installed from inside the TBM. These were drilled through the bottom of the shield at an angle of about 1:1 towards the tunnel face. The as-built layout of the freeze pipes as well as the system of trenches housing the brine conveyance infrastructure directly beneath the street level is shown in Figure 3. A schematic profile view of the freeze pipes in relation to the BT-3 TBM is depicted in Figure 4 (Gwildis, 2012).

CONSTRUCTION OF THE TUNNEL CONNECTION

The BT-3 TBM was manufactured to resist 8 bar of pressure and has bulkheads and internal reinforcements to resist face pressure and deformation of the shield. Prior to the disassembly of the TBM, an analysis of the shield was performed so that a sequence could be planned for the removal of internal superstructure to ensure that deformation of the shield would be minimized. Using the PLAXIS program, the design team developed a 2D/3D finite element model of the TBM shield to predict deformations with the shield fully stripped out. Analysis indicated that deformation of the shield would be more than several inches (becoming oval) if the TBM bulkheads were fully removed when

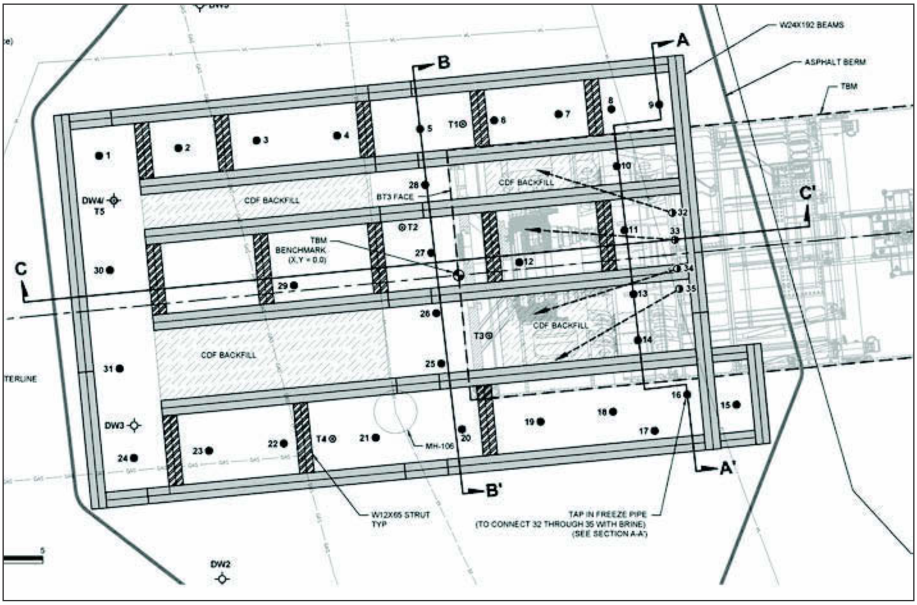

Figure 3. Freeze pipe layout plan

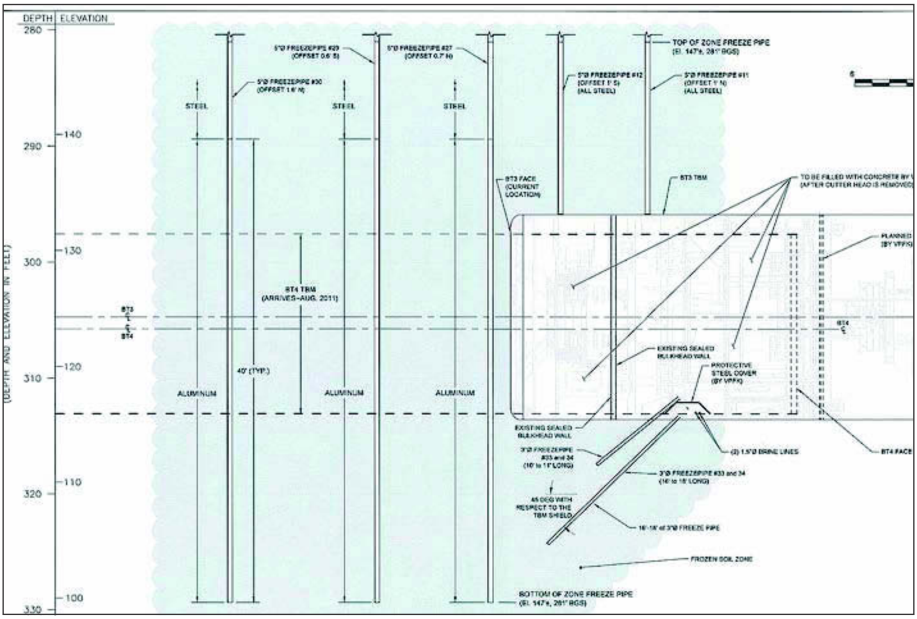

Figure 4. Freeze pipe layout profile

Figure 5. Temporary bracing installed

the surrounding ground was not yet frozen. The disassembly sequence required steel reinforcements welded across the articulation joints and at the tail shield prior to the removal of the hydraulic cylinders controlling thrust and steering. Monitoring devices were installed within the shield to measure deformations as the TBM superstructure was removed. While the ground freezing process was underway, the bulkheads were only partially removed with a circular ring beam left in place to keep the shield round. Based on predictions of the ground freezing process, the team had assumed that the ground would be frozen by the time this work had been completed. To keep the dismantling work going while the freezing was underway, temporary bracing was added in the shield to permit full removal of the bulkheads (Figure 5). This temporary bracing was located to limit shield deformation as well as provide for concrete formwork installation to enable the shield to be later filled with concrete. Even with the bracing installed, the analysis model predicted the shield to deform about 50 mm as the superstructure was removed and ground and freeze loads developed.

Once the surrounding ground was frozen, the BT-3 TBM cutterhead was removed in stages from the top down, with light breasting to protect workers from from localized thawing and slaking of the exposed tunnel face. Removal of the cutterhead continued over a period of more than four weeks without any leaks or loss of ground. As the last piece of the cutterhead was removed from the bottom, a small, localized leak in the freeze wall occurred. An inflow of about 8 m^3 (10 yd^3) of water and soil accumulated at the bottom of the shield. Short steel sheets and grouting were used to slow the inflow rate of water to less than 20 l/min (5 gl/min) so the remaining parts of the TBM could be removed (Gwildis 2012).

With the cutterhead and superstructure completely removed, the BT3C contractor placed a concrete plug within the shield. This plug extended about 4.5 m (15 ft) from the tunnel face, just past the forward articulation joint of the abandoned shield. The concrete plug was poured in three lifts—each pour two days apart—using a 27.6 MPa (4000 psi) low-heat-of-hydration mix that included Novomesh 950 synthetic fibers at about 6 kg per cubic meter (10 lbs per cubic yard). At this time, the shield deformations were measured at 46 mm—confirming the model prediction. To restrict further deformation of the shield, two thick-walled 254-mm- (10-in.)-diameter aluminum pipe sections were installed as vertical posts and filled with grout (Gwildis 2012).

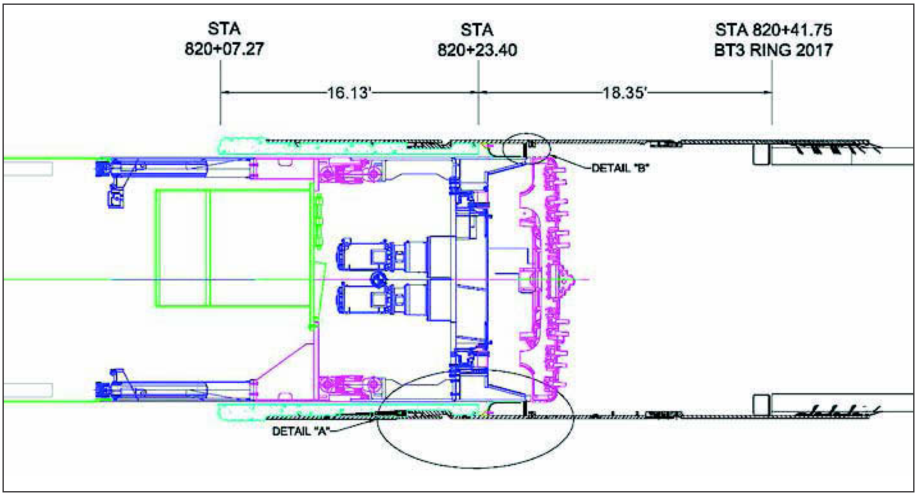

Figure 6. Depiction of the required accuracy for BT3C connection to BT-3

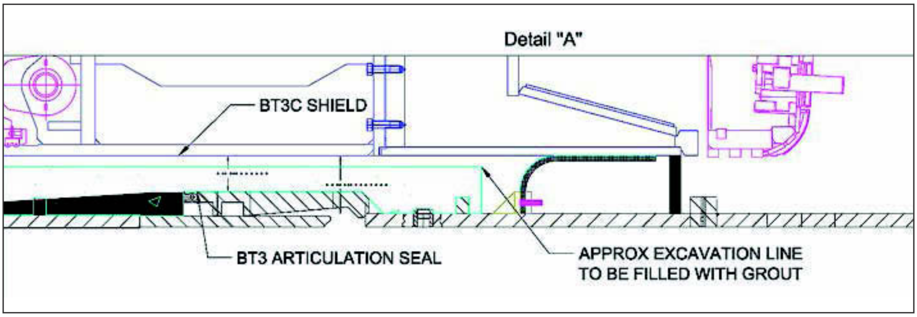

Figure 7. Detail of the required accuracy for BT3C connection to BT-3

SURVEY

The requirements for the completion of the mining operations for the docking of BT3C with the BT-3 tunnel required accurate and reliable tunnel survey control to ensure the BT3C TBM mines into the abandoned shield of the BT-3 TBM along the same tunnel axis. The clearance between the 4368 mm (172-in) outside diameter of the BT3C TBM and the inside diameter 5181 mm (204-in) of the abandoned BT-3 shield was 406 mm (16-in). This clearance was further reduced down to about 280 mm (11 in) due to protruding portions of the BT-3 TBM shield at the articulation joints. Figures 6 and 7 depict the breakthrough requirements for the tunnel connection.

The control in the BT-4, BT-3C, and the BT-3 tunnels, which was established during the mining operations, consisted of high-order survey observations including differential Global Positioning System (GPS) surveys on the surface, conventional observations (i.e., direction sets, zenith angles, slope distances, precision leveling, and gyro-azimuth observations). The surface control was densified around the project sites, through the shafts, and into the tunnels at regular intervals. The survey control was critical not only to ensure the correct alignment and grade of the TBM guidance system during the mining operations in the BT3C tunnel but it was also required to perform as-builts on the individual shield sections of the BT-3 TBM to determine its position and attitude. The success of the tunnel connection was dependent on ensuring the highest possible accuracy for the survey control. The survey requirements included:

- Surface control (GPS)
- Transfer of horizontal and vertical control through the portals (i.e., Port Wells, Ballinger Way, North Kenmore, etc.)
- Propagation of control through the tunnels to the headings (including gyro-azimuth observations)
- As-built of the abandoned shield

The surface survey control consisted of high-order differential Global Positioning System (GPS) Surveys and conventional observations for densifying control around the shafts. During the Brightwater Project, the surface control was verified for its internal accuracy and validation of a proper geodetic datum. The relative accuracy of the surface control, at each portal or shaft location, was determined by observing high-order differential GPS. Static differential GPS does provide the required relative accuracy for mining operations and was used throughout the Brightwater Project. The relative accuracy of the surface control is especially critical when multiple TBMs are used on the project and are connecting critical portions of the project such as the case with the Brightwater Project.

Differential GPS consists of collecting GPS data over long durations on the surface control simultaneously and post-processing to achieve a high 'relative' accuracy survey. In a majority of tunnel construction projects across the U.S., lower order design survey control points are usually established at the construction areas and along the alignment corridor for the purpose of establishing mapping products for design of the tunnel. In many instances, the accuracy required for the mapping control is of a lower order than what is required for tunnel control. Also, in certain instances, the project control is not even on a recognized survey system. This creates issues and errors that will degrade the accuracy of the tunnel survey control which may cause issues with the design tolerances.

In the case of the Brightwater Project, differential GPS was collected using the National Geodetic Surveys, standard for Order B accuracy (8 mm + 1 ppm) surveys. The results of the differential GPS survey performed on the design control indicated that surface control has a relative high accuracy suitable for the tunnel control requirements. In addition, by constraining or 'tying' to local Continuous Operating Reference Stations (CORS), allowed additional geometry and redundancy in the GPS network and validated the survey datum as North American Datum of 1983 (NAD83). The importance of validating the geodetic datum is critical to ensure proper reductions of gyro-azimuth observations.

Once the control had been established or verified in the vicinity of the portal project sites (i.e, Point Wells, Ballinger Way, North Kenmore, and the Influent Pump Station) control was then transferred to the starter tunnels. At the Point Wells Portal, the depth and length of the portal allowed conventional survey techniques to be used to transfer control to the tunnel. However the North Kenmore and Ballinger Way Portals were too deep and narrow to allow the conventional method to be used. Instead, the survey utilized a unique method that has been implemented on numerous tunnel projects throughout the United States and Canada. The method, allows horizontal and vertical survey control to be transferred from the surface to the underground, by using precision industrial metrological instrumentation (i.e., Taylor-Hobson Spheres and Bæchlor prisms) with conventional observations. The Bæchler prism spheres (Figure 8) were suspended from two temporary brackets set over the North Kenmore Portal at locations with clear vertical (plumb) visibility to the shaft invert. The Bæchler prisms and sphere holder are all designed such that the centers of each sphere maintains the same 3-D coordinate space to within 0.2 mm in any orientation, including orienting

Figure 8. Bæchler prisms used for transferring survey control through the North Kenmore Portal (shaft)

Figure 9. Taylor Hobson spheres used for transferring survey control through the North Kenmore Portal (shaft)

Figure 10. Gyro-theodolite observations in the BT-3 Tunnel using the DMT Gyromat 2000

the sphere vertically (i.e., pointing to the nadir). The center of the Bæchler prisms, on the surface, were precisely observed on the surface, conventionally from the nearby densified control. The Bæchler prisms were then replaced by Taylor-Hobson spheres (Figure 9) and oriented (i.e., facing) downwards. At the bottom of the shaft, a zenith (ZL) plummet was used to center precisely below the Taylor-Hobson spheres. Two temporary tripods were setup directly underneath the surface brackets and precisely centered using the ZL plummet. Once centering was achieved, the vertical distances were measured using a TCA2003 total station to the Bæchler Prisms mounted over the shaft to transfer elevations to the shaft invert. The control, now precisely transferred to the trunnion axis of the total station set at temporary tripods set underneath the Bæchler prisms on the invert, was then transferred to the underground control.

Conventional observations (i.e., directions, slope distances, and zenith angles) were observed in the tunnel using Automatic Target Recognition (ATR). ATR consists of a infrared beam transmitted from the TCRP1201+ and reflected back from the prism. The return beam is analyzed and the center of the prism is located. This methodology greatly enhances the accuracy as well as optimizes the time of observations. Underground observations were forced centered on brackets thereby avoiding the adverse influence of centering errors. The methodology of the horizontal tunnel control was to ensure there was redundancy in the observations, hence multiple back-sights and foresights were observed.

The tunnel control networks in the BT-3, BT3C, and BT-4 tunnels were complemented by numerous gyro-azimuth observations with a DMT Gyromat 2000 (Figure 10). Gyro-azimuth observations were critical as they minimize the propagation of random errors and minimize influence of other others such as lateral refraction which may exist in tunnel environments during mining operations. The gyro-azimuth observations became critical when access from the surface to the underground was not possible at the Ballinger Way Portal. As a result, there was no positional check on the tunnel control after 6,100 m (20,000 ft) when the BT-4 mining operations were completed.

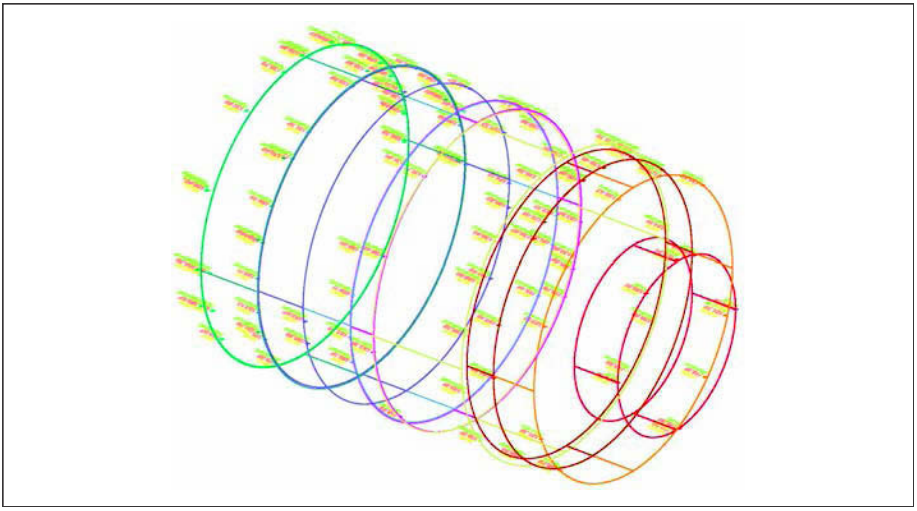

Figure 11. Three-dimensional as-built of the BT-3 TBM shields; tail shield on left and main drive on right

Further gyro-azimuth observations were performed in the BT-3 tunnel to increase the accuracy of the tunnel control near the heading of the BT-3 TBM. The purpose was to determine the TBM position at a higher level of accuracy. This required the highest possible accuracy of tunnel control.

To successfully ensure that the BT3C TBM would breakthrough into the BT-3 shield, as-builts of the three articulation shields of the BT-3 TBM were required. A three-dimensional location of the articulation shields were determined by conventional observations to key locations on each shield. At certain intervals during the dismantling of the BT-3 TBM, survey observations were performed from the survey control to key features of each abandoned shield. Such features included the springline, crown and invert of each abandoned shield section. The location and attitude (i.e., line and grade) of the abandoned shields were determined using AutoCAD. This allowed for revisions to the Design Tunnel Alignment (DTA) of the BT3C tunnel to be recomputed and verified prior to breakthrough. As further work was performed to dismantle the BT-3 TBM shields, the as-built surveys were used to re-observe the features of the shields and determine whether further deformation of the shields had occurred. On several occasions, shield deformation was measured when key bulkheads were removed. Figure 11 depicts the AutoCAD 3D model of the abandoned shields after the as-built survey.

CONCLUSION

The requirements for ensuring that the BT3C TBM could properly dock with the abandoned BT-3 TBM shields required proper analysis of the ground and geotechnical conditions, complemented by accurate and reliable survey. The geotechnical and structural requirements consisted of ensuring proper freeze of the soils, followed by deformation analysis of the TBM to verify the clearance between the TBMs. In addition, a precise as-built of the abandoned BT-3 TBM shield was prepared using the survey control that had been used for verifying the contractor's control during the mining operations. A high level of precision and accuracy in this as-built was needed so that the proper Design Tunnel Alignment could be determined for the BT3C tunnel to break in to the abandoned shield aligned center-to-center. The TBM entered the shield, mined through the concrete plug and into the abandoned shield nearly perfectly centered (Figure 12).

Figure 12. Breakthrough of the BT3C TBM inside the larger-diameter BT3 shield

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APPROACH FOR A COST-EFFECTIVE VENTILATION DURING CONSTRUCTION OF A HIGH-SPEED RAILWAY TUNNEL

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ABSTRACT

As part of a new high-speed railway line in Germany, a twin-tube tunnel with a length of 8,200 m (26,900 ft) is designed and will be constructed adopting the drill and blast method. The two parallel tunnel tubes feature cross passages every 500 m (1,640 ft) and will be built concurrently with the excavations, starting from five different locations. Due to the fact that the excavation works will all take place at the same time, a circulating air system for the supply of fresh air will be used during the construction phase. This innovative method was found to lower the equipment installation costs and to minimize the electric power consumption during construction.

INTRODUCTION

Harmful substances in the tunnel air have to be collected at the emission source and have to be transported out of the tunnel, or they have to be diluted inside the tunnel. Depending on the construction and operational mode, a) blowing ventilation systems or b) circulating ventilation systems are used in general [1]. Other variants such as extraction ventilation systems are used rarely, for economical and operational reasons.

a. Blowing ventilation system (Figure 1). Fresh air is transported to the working face of the tunnel. The contaminated air flows in the air flow cross section towards the portal. The area at the working face is provided with fresh air, whereas the working areas downstream of the working face receive contaminated air.
b. Circulating ventilation system (Figure 2). Especially for long tunnel systems circulating ventilation systems are the most preferable systems, because of a loss-free air movement. Therefore, huge air flow rates can be achieved very economically. Combinations of circulating ventilation systems with blowing ventilation systems are possible and useful.

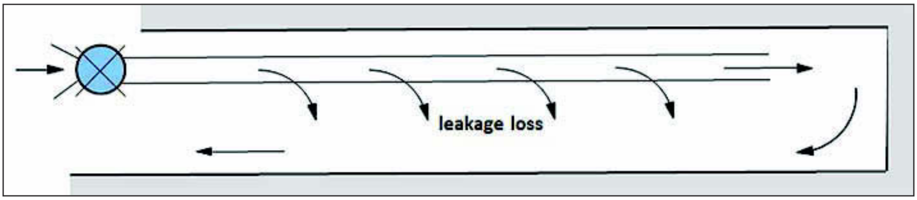

Figure 1. Blowing ventilation system

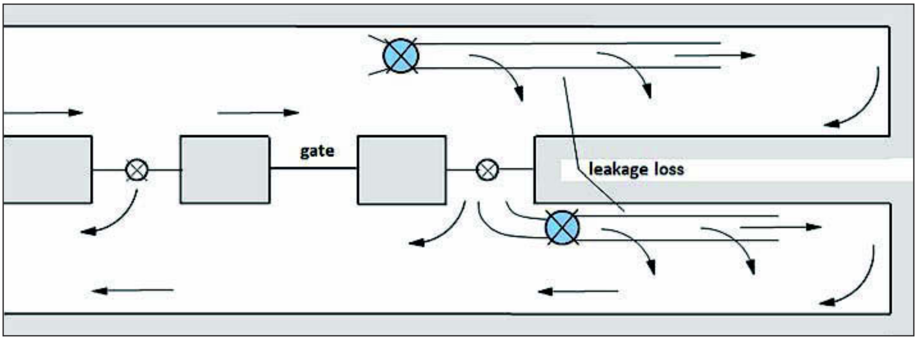

Figure 2. Circulating ventilation system

Notation

d = diameter of tubing (m)
p = pressure (Pa)
u = flow velocity (m/s)
x = longitudinal position in the tunnel (m)
ρ = air density (kg/m^3)
λ = friction coefficient
ζ = pressure loss coefficient
f' = leakage area per tubing surface (mm^2/m^2)

BASIC EQUATIONS

The calculation of the ventilation system has to ensure that the ventilation system is able to achieve the required fresh air flow rates during all construction phases. The pressure losses and the leakages have to be taken into account in the design of the fan. They primary depend on the diameter, the length of the continuously extending tubing, the flow rate and the friction coefficient of the tubing. This is taken into account with the following two basic differential equations [1]:

Change of static pressure:

$$\frac{dp}{dx} = \lambda \cdot \frac{1}{d} \cdot \frac{\rho}{2} u^2 \tag{1}$$

Change of velocity:

$$\frac{du}{dx} = \frac{4 \cdot f'}{d} \cdot \sqrt{\frac{p}{\frac{\rho}{2} \cdot \left(1 + \zeta\right)}} \tag{2}$$

The size of the leakage flow rate depends on the static pressure inside the tube. Therefore, also volumetric flow rate depends on it.

VENTILATION SYSTEM

Figure 3 gives an overview of the tunnel driving directions. Furthermore, intermediate headings 1 and 2 are depicted in Figure 3.

There are cross passages every 500 m (1640 ft) for the parallel driven tunnels to connect both tunnel tubes. The circulating ventilation system will operate after finishing the first cross passage. For operational reasons this can only be realized after a tunnel section length of about 700 m (2,296 ft). Until this point, the blowing ventilation

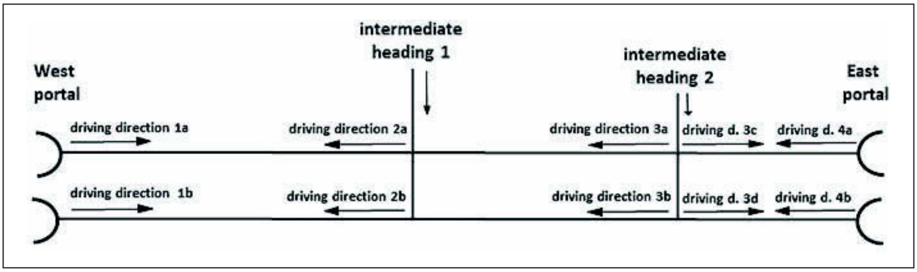

Figure 3. Sketch with tunnel driving directions

Table 1. List of tunnel section lengths and tunnel driving modes

Tunnel Driving	Length (m)	Driving Mode
1a, 1b	700	Parallel
2a, 2b	2,100	Parallel
3a, 3b	1,900	Parallel
3c, 3d	1,600	Parallel
4a, 4b	1,600	Parallel
Intermediate heading 1	1,000	Serial
Intermediate heading 2	300	Serial
Total	17,100	

Table 2. Standard values for fresh air requirement [1] and [2]

Requirement	Value
Fresh air per diesel-hp (nominal power)	4 m^3/s
Fresh air per person	2 m^3/s
Minimum air velocity	0.2 m/s
Maximum air velocity	6 m/s

system is operating to guarantee the required air quality conditions. These conditions are summarized in Table 2. Table 1 gives an overview of the tunnel section lengths and the driving modes.

As shown in Figure 4, the tunnel of the intermediate heading rarely has space for the installation equipment. 4 tubings with a diameter of 1.8 m (5,9 ft) for fresh air supply are located above the clearance profile. Additional tubings are necessary to transport the contaminated air from the blasting process out of the tunnel. Therefore, two tubings with a diameter of 0.8 m (2,6 ft) are installed. Intermediate headings 1 and 2 are both single-bored tunnels, therefore an escape route has to be provided in the tunnel (Figure 5). Fleeing people will be able to get out of the tunnel by using the escape routes. Additional free space is reserved for equipment, e.g., electrical power supply or wastewater pipes.

Fresh air is transported through tubing from the outside into the tunnel behind the sluice.

For the circulating ventilation system additional secondary fans are necessary to handle the air flows at the tunnel face. One fan is necessary to transport the fresh air to the adjacent tunnel face. In the fresh air tunnel an exhaust fan has to be installed to transport the contaminated air also to the adjacent tunnel tube. The parallel tunnel

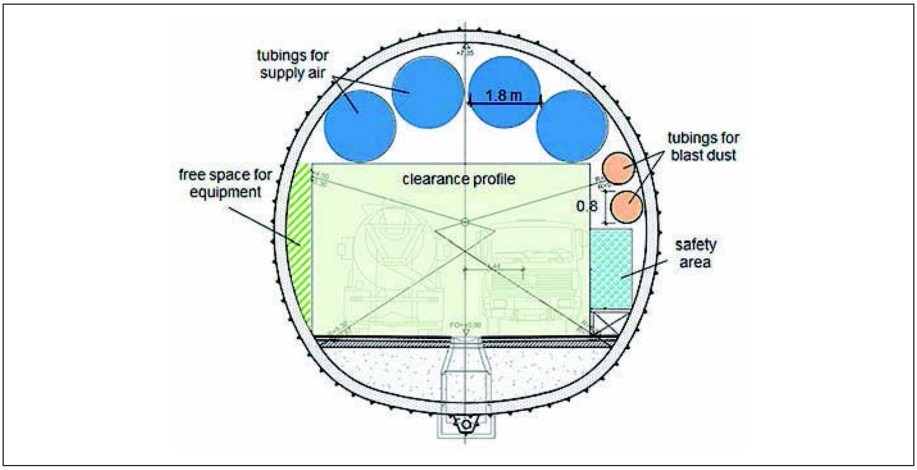

Figure 4. Typical cross section with tubings for air transport, intermediate heading

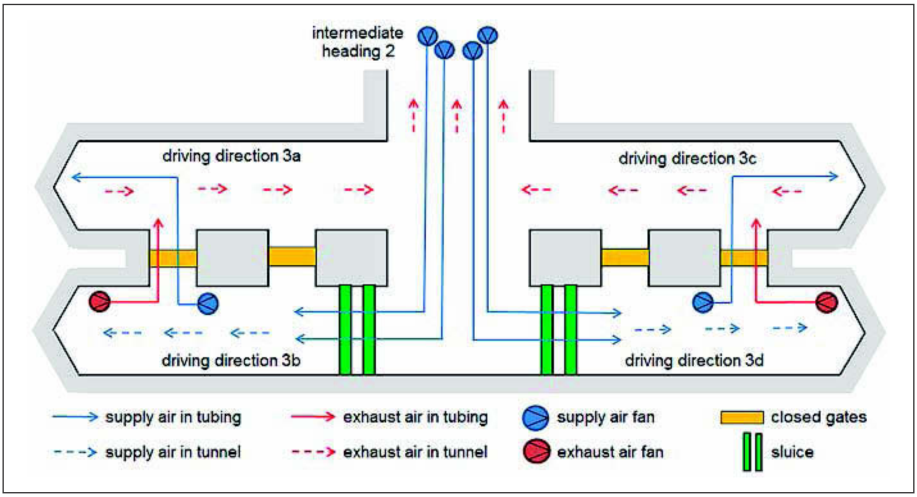

Figure 5. Schematic drawing of circulating ventilation system, intermediate heading 2

tubes have closed cross passages to prevent the contaminated air from flowing into the tunnel transporting fresh air. At the start of the tunnel transporting fresh air an automatic sluice is installed. This allows vehicles to access the aerodynamically isolated tunnel.

The start of the parallel driven tunnels operates with a blowing ventilation system, see Figure 6.

After finishing the first cross passage, the ventilation system can change into a circulating ventilation system. This will happen after a tunnel section with a length of about 700 m (2,296 ft) is excavated. Tables 3 and 4 list the fresh air requirements during construction phases.

The quality of the tubing for all performed calculation cases is specified as class A in SIA [1]. This quality class is indicated for new tubing which is serviced at regular intervals and installed with low risk of damage. The tubing has a friction coefficient λ of 0.018 and a specific leakage f' of 10 mm^2/m^2.

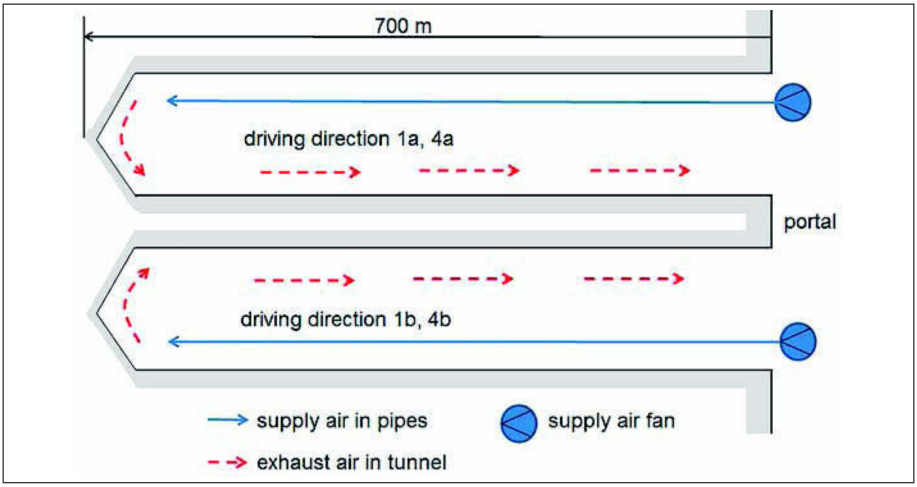

Figure 6. Starting situation with blowing ventilation system

Table 3. Heavy vehicles for tunnel driving

Heavy Vehicle	Nominal Power (kW)	Quantity
Wheel loader	280	1
Truck	250	Varying*
Excavator	160	2
Pickup truck	100	1

* depending on section length

Table 4. Volumetric flow rate of fresh air

Tunnel Section	Heavy Vehicles/ Persons	Flow Rate (m^3/s)	Section Length (m)	*d* (m)
1a, 1b	Table 3 + 2 trucks/10	80	700	2.0
2a, 2b	Table 3 + 4 trucks/10	114	2,100	2.2
3a, 3b	Table 3 + 4 trucks/10	114	1,900	2.2
3c, 3d	Table 3 + 3 trucks/10	97	1,600	2.0
4a, 4b	Table 3 + 3 trucks/10	97	1,600	2.0

COMPARISON OF BLOWING VENTILATION AND CIRCULATING VENTILATION

This chapter describes a comparison of the blowing ventilation system and the circulating ventilation system with regard to power consumption. The tunnel is driven using the drill and blast method. The average tunnel drive length per month is 100 m (328 ft).

The length of the tubing is about 50 m (164 ft) longer than the length of the tunnel section, because of the placement of the fan outside the tunnel. The energy price was set to 10 Cent/kWh to illustrate the energy costs for the different ventilation systems.

Tunnel Section 1a, 1b

Tunnel sections 1a and 1b are 700 m (2,296 ft) long. Therefore, the installation of the circulating ventilation system is not possible due to the short tunnel section (Figure 7).

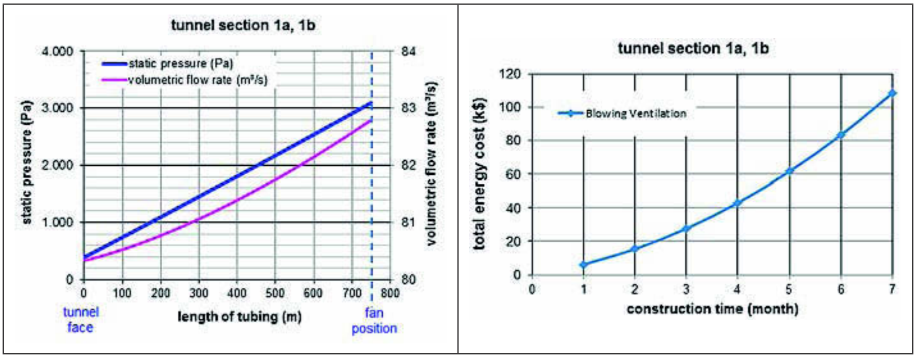

Figure 7. Tunnel section 1a, 1b with blowing ventilation system

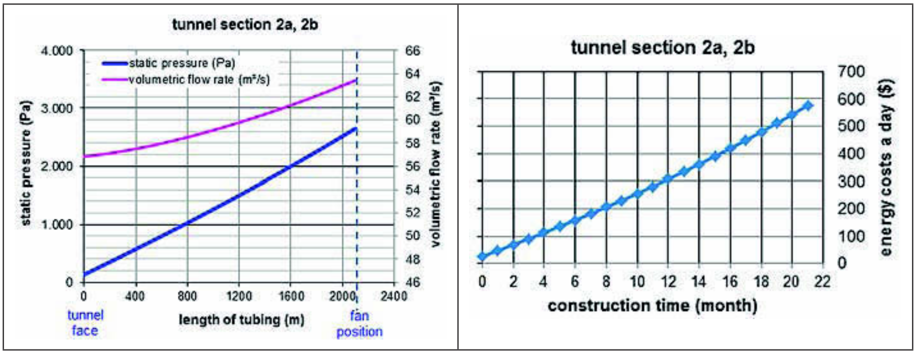

Figure 8. Tunnel section 2a, 2b with blowing ventilation system

Tunnel Section 2a, 2b

In tunnel sections 2a and 2b are two tubings installed for fresh air supply. The diagram depicted in Figure 8 shows the pressure profile and the volumetric flow rate of one of the tubing for a tunnel section length of 2,100 m (6,888 ft). Furthermore, the continuously rising energy costs per day are shown over the construction duration of 21 months.

In tunnel sections 2a and 2b are two tubings installed for fresh air supply. The diagram depicted in Figure 9 shows the pressure profile and the volumetric flow rate of one of the tubing for a maximum tubing length of 700 m (2,296 ft). Furthermore, the alternating energy costs per day are shown over the construction duration of 21 months.

Figure 10 shows a comparison of the total energy costs of blowing ventilation versus circulating ventilation. Both are operating with a constant tubing diameter of 2.2 m (7.2 ft). The total energy costs of the circulating ventilation are 41% of the blowing ventilation.

Tunnel Section 3a, 3b

In tunnel sections 3a and 3b are two tubings installed for fresh air supply. The diagram depicted in Figure 11 shows the pressure profile and the volumetric flow rate of one of the tubing for a tunnel section length of 1,900 m (6,232 ft). Furthermore, the continuously rising energy costs per day are shown over the construction duration of 19 months.

In tunnel sections 3a and 3b are two tubings installed for fresh air supply. The diagram depicted in Figure 12 shows the pressure profile and the volumetric flow rate

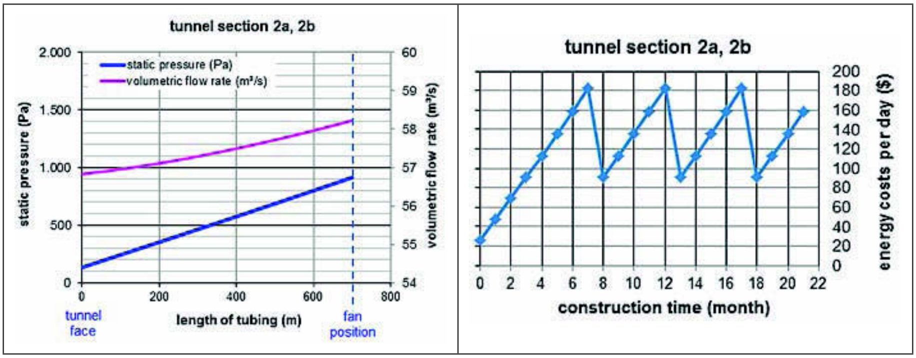

Figure 9. Tunnel section 2a, 2b with circulating ventilation system

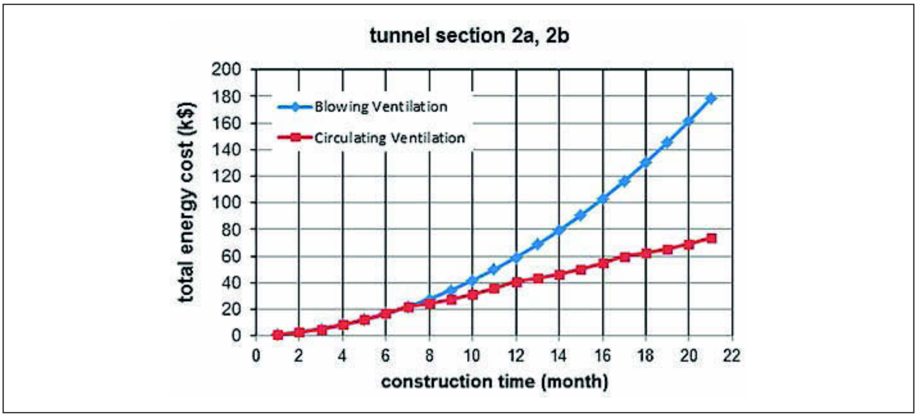

Figure 10. Comparison of energy costs of blowing ventilation vs. circulating ventilation

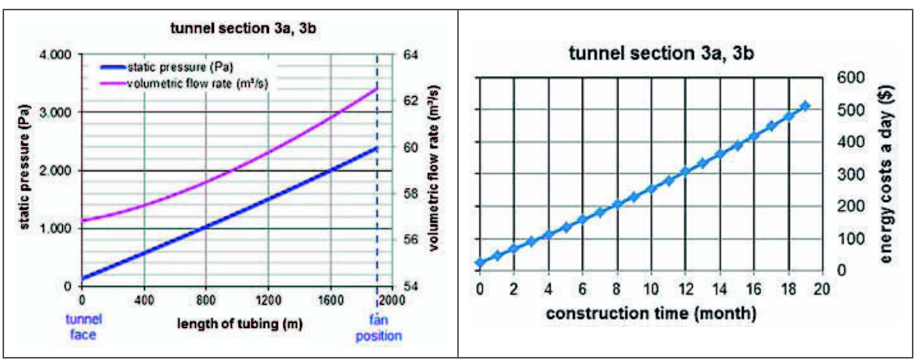

Figure 11. Tunnel section 2a, 2b with blowing ventilation system

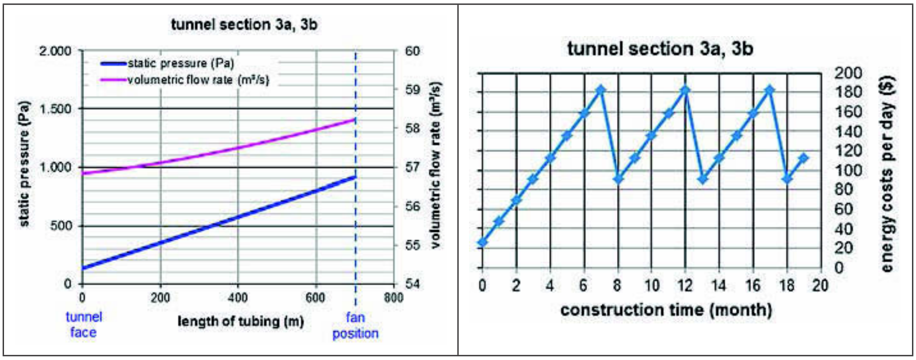

Figure 12. Tunnel section 3a, 3b with circulating ventilation system

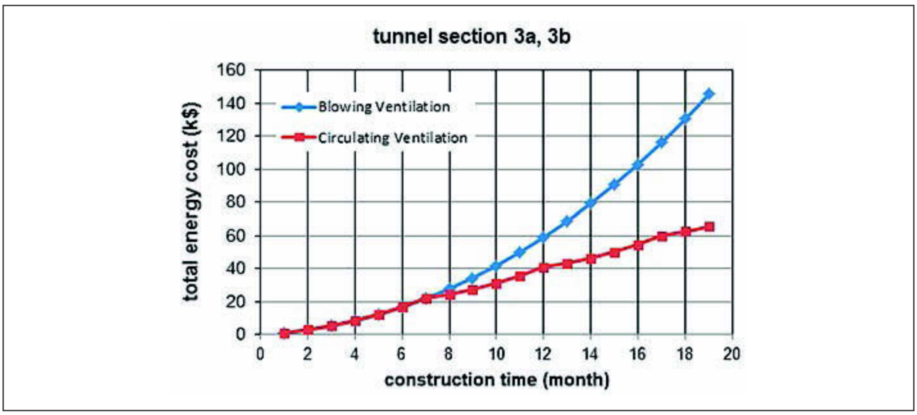

Figure 13. Comparison of energy costs of blowing ventilation vs. circulating ventilation

of one of the tubing for a maximum tubing length of 700 m (2,296 ft). Furthermore, the alternating energy costs per day are shown over the construction duration of 19 months.

Figure 13 shows a comparison of the total energy costs of blowing ventilation versus circulating ventilation. Both are operating with a constant tubing diameter of 2.2 m (7.2 ft). The total energy costs of the circulating ventilation are 45% of the blowing ventilation.

Tunnel Section 3c, 3d, 4a, 4b

In tunnel sections 3c, 3d, 4a and 4b are two tubings installed for fresh air supply. The diagram depicted in Figure 14 shows the pressure profile and the volumetric flow rate of one of the tubing for a tunnel section length of 1,600 m (5,248 ft). Furthermore, the continuously rising energy costs per day are shown over the construction duration of 16 months.

In tunnel sections 3c, 3d, 4a and 4b are two tubings installed for fresh air supply. The diagram depicted in Figure 15 shows the pressure profile and the volumetric flow rate of one of the tubing for a maximum tubing length of 700 m (2,296 ft). Furthermore, the alternating energy costs per day are shown over the construction duration of 16 months.

Figure 16 shows a comparison of the total energy costs of blowing ventilation versus circulating ventilation. Both are operating with a constant tubing diameter of 2.0 m

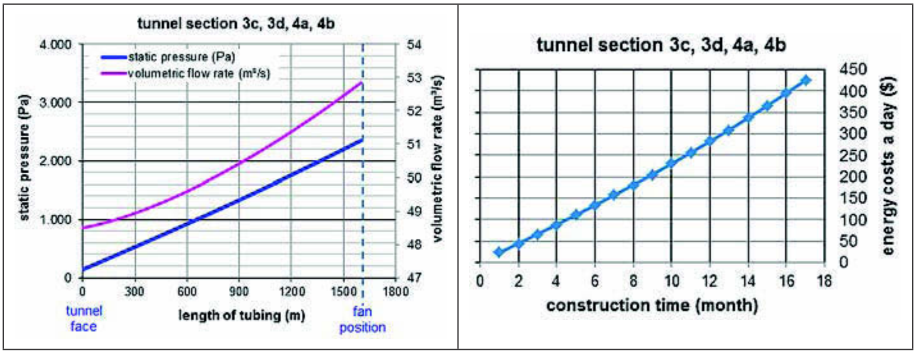

Figure 14. Tunnel section 3c, 3b, 4a, 4b with blowing ventilation system

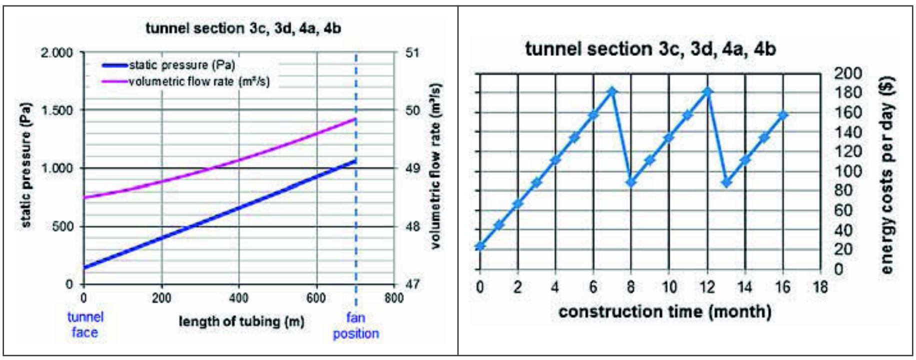

Figure 15. Tunnel section 3c, 3b, 4a, 4b with circulating ventilation system

(6.6 ft). The total energy costs of the circulating ventilation are 52% of the blowing ventilation.

CASE STUDY: VARYING TUBING DIAMETERS

In this case study the diameters of the tubing for the circulating ventilation system varies. The diameter of the tubing has significant impact for the required fan power and therewith for the total energy costs. The energy costs in the diagrams are indicated for one tubing. Two tubings are necessary to provide the tunnel sections with the required air flow rate.

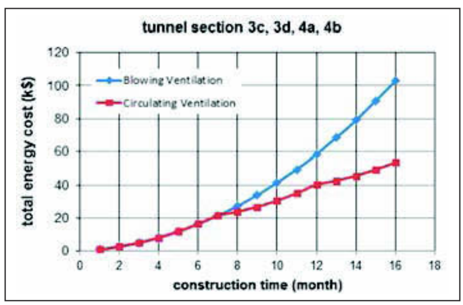

Figure 16. Comparison energy costs blowing ventilation vs. circulating ventilation

Figure 17 shows the total energy costs during construction time. The total energy costs for the blowing ventilation are \$178.5k and for the circulating ventilation are \$73.8k. Therefore, the energy costs for blowing ventilation are 2.4 times higher than for circulating ventilation. The tubings of both ventilation systems have a diameter of 2.2 m. A reduction of the tubing diameter has a high impact on the total energy costs. A reduction of the diameter of the circulating ventilation from 2.2 m to 2.0 m and to 1.8 m

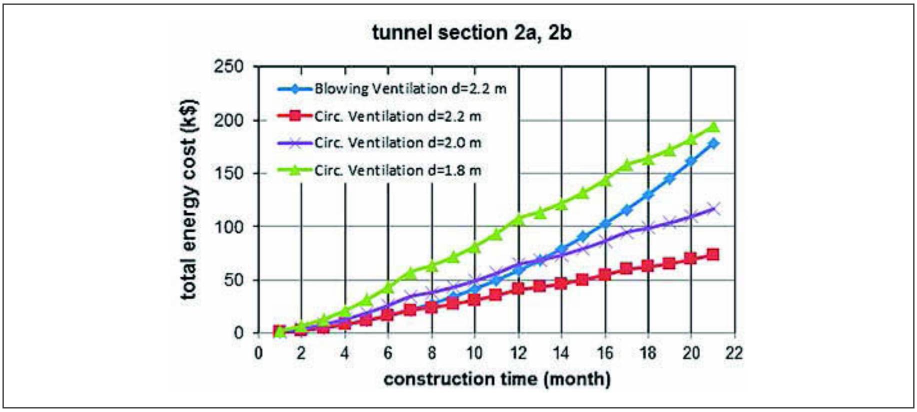

Figure 17. Comparison of energy costs with varying diameters, tunnel section 2a, 2b

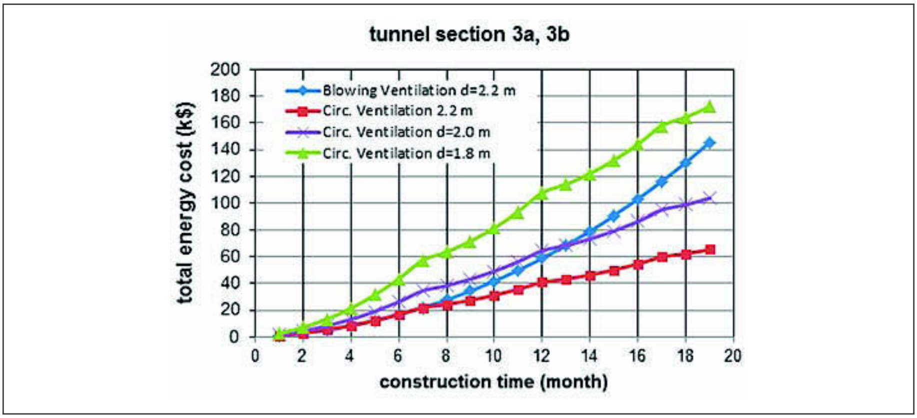

Figure 18. Comparison of energy costs with varying diameters, tunnel section 3a, 3b

increases the total energy costs for 58% and for 163%. The last variant is more expensive than the blowing ventilation system.

Figure 18 shows the total energy costs during construction time. The total energy costs for the blowing ventilation are $145.4k and for the circulating ventilation are $65.5k. Therefore, the energy costs for blowing ventilation are 2.2 times higher than for circulating ventilation. The tubings of both ventilation systems have a diameter of 2.2 m. A reduction of the tubing diameter has a high impact on the total energy costs. A reduction of the diameter of the circulating ventilation from 2.2 m to 2.0 m and to 1.8 m increases the total energy costs for 58% and for 163%. The last variant is more expensive than the blowing ventilation system.

Figure 19 shows the total energy costs during construction time. The total energy costs for the blowing ventilation are 103.1k$ and for the circulating ventilation are $53.8k. Therefore, the energy costs for blowing ventilation are 1.9 times higher than for circulating ventilation. The tubings of both ventilation systems have a diameter of 2.0 m. A reduction of the tubing diameter has a high impact on the total energy costs. A reduction of the diameter of the circulating ventilation from 2.0 m to 1.8 m and to 1.6 m increases the total energy costs for 66% and for 195%. The last variant is more expensive than the blowing ventilation system.

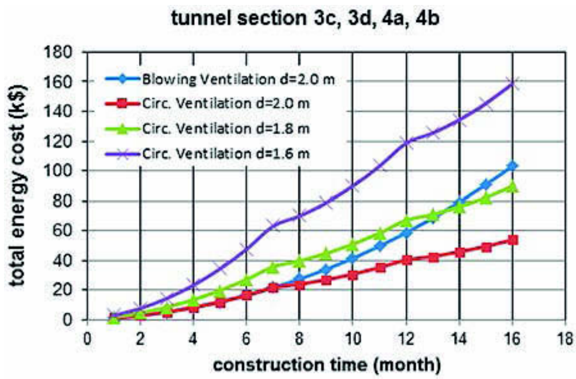

Figure 19. Comparison of energy costs with varying diameters, tunnel section 3c, 3d, 4a, 4b

SUMMARY

The results of the calculations show that circulating ventilation system can reduce the total energy costs up to about 60% for the longest tunnel section, compared to the blowing ventilation system. Therefore, especially for long tunnels with parallel tunnel driving modes, the circulating ventilation is an economic way to ventilate during construction works. The longer the tunnel section, the higher the energy reductions with the use of circulating ventilation system. The diameter of the tubing has significant impact for the required fan power and therewith for the total energy costs. Excavation time is direct proportional to the power consumption and therewith direct proportional to total energy costs. An additional advantage of the circulating ventilation is the ability to produce an overpressure in case of fire in the non-affected tunnel with fresh air. Therefore, the non-affected tube can be used as an escape route for fleeing people.

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KING COUNTY USES NEW SHAFT TECHNOLOGY ON THE BALLARD SIPHON PROJECT

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Marty Noble ▪ King County Wastewater Treatment Division

Jeremy Johnson ▪ Jacobs Associates

Doug Genzlinger ▪ Tetra Tech

John Fowler ▪ James W. Fowler Company

ABSTRACT

As part of a trenchless project in Seattle, Washington, the King County Department of Natural Resources and Parks–Wastewater Treatment Division has achieved a North American first by utilizing the Herrenknecht vertical shaft machine (VSM) to construct a 9 m diameter (29.5 ft), 47.5 m deep (156 ft) shaft, which will act as the tunnel boring machine launch shaft. The general contractor, James W. Fowler Company, elected to utilize the VSM to successfully complete the excavation and segmental lining of the shaft as an alternative to its originally planned slurry wall method. Excavation was completed in-the-wet through fill and overconsolidated glacial till, with groundwater at approximately 4.6 m (15 ft) below the ground surface. This paper provides a brief overview of the geotechnical conditions and design considerations for the shaft and structure, outlines the use and capabilities of the VSM, describes the construction technique and installation progress achieved. Also provided is a summary of lessons learned on the project with regard to the use of the Herrenknecht VSM.

INTRODUCTION

The Ballard Siphon Replacement Project (BSRP) is located in Seattle, Washington, USA; and is being undertaken by the King County Department of Natural Resources and Parks–Wastewater Treatment Division. The north project site is located in Seattle's Ballard neighborhood, while the south project site is located in the Interbay neighborhood. The tunnel connecting the two project sites crosses beneath the Lake Washington Ship Canal (Canal), which is a US Army Corp of Engineers maintained navigable channel connecting Lake Washington with the Puget Sound.

The BSRP is being constructed to rehabilitate two existing 70-year-old wooden sewer pipes, which cross beneath the Canal at Salmon Bay. In addition, the BSRP will provide additional capacity to the sewer system in order to protect the water quality of the Canal, and to allow for future growth in the North Seattle area. The major project construction activities include:

- A 27 m deep (89 ft) shaft, Forebay Structure, and Regulator Structure addition at the north project site
- A 41 m deep (134 ft) shaft, Afterbay Structure, and Junction Structure at the south project site
- Slip-line rehabilitation of the two existing 915 mm (36 in.) internal diameter (ID) wood-stave pipelines that currently carry wastewater across the Canal

Figure 1. Project overview

- A new 603 m long (1,977 ft) tunnel connecting the north and south sites, which will be finished with a 2,250 mm (88.5 in.) ID final lining

Figure 1 provides a project overview.

The shaft located on the south project site (South Shaft) is the focus of this paper. During construction, this shaft was used to launch and stage the tunneling activities. In its final configuration, the shaft will house piping to convey wastewater from the tunnel to the new Afterbay and Junction structures.

James W. Fowler Co. (JWF) is the general contractor; the project design was prepared by the team of Tetra Tech, Landau Associates, and Staheli Trenchless Consultants; contractor-design of the South Shaft caisson, shaft invert, and ring beam was completed by Brierley Associates; and construction management is being provided by Jacobs Associates.

SOUTH SHAFT GEOTECHNICAL CONDITIONS

The Geotechnical Baseline Report (GBR) for the project was completed by Landau Associates, in cooperation with design team members Tetra Tech and Staheli Trenchless Consultants. It was anticipated that four different geologic units would be encountered during construction of the South Shaft: Holocene Fill, Vashon Advance Outwash, Pre-Fraser Interglacial Deposits, and Pre-Fraser Slickensided Deposits.

The Holocene Fill unit was encountered to a depth of about 2.4 m (8 ft), and consisted of loose sand and silty sand, and medium stiff sandy clay. The Vashon Advance Outwash Deposits were present from about 2.4 to 5.5 m (8–18 ft) below the ground surface, and consisted of medium dense, silty, fine to medium sands with gravel. Pre-Fraser Interglacial Deposits were encountered for the next 21 m (70 ft) of excavation, from about 5.5 to 27 m (18–88 ft) below the ground surface. This glacially overconsolidated unit consisted of interbedded layers of very dense granular (sand and silty sand) and hard cohesive (silt and clay) materials. The final 21 m (69 ft) of excavation were completed through the Pre-Fraser Slickensided Deposit. This glacially overridden deposit was similar to the finer-grained portions of the Pre-Fraser Interglacial unit above, consisting of very stiff to hard clay containing very small sand-filled slickensides.

Groundwater was anticipated to be about 4.9 m (16 ft) below the ground surface. Vibrating wire piezometers around the shaft indicated the groundwater levels were stable throughout shaft construction at about 6.1 m (20 ft) below the ground surface.

SOUTH SHAFT DESIGN

The South Shaft provides temporary ground support to allow the tunnel and Afterbay Structure to be built; however, it is not part of the permanent facility. Because of this, King County specified that the shaft be contractor-designed. The Contract Drawings specified the shaft center point and the tunnel invert elevation, and the Contract Specifications listed several acceptable construction methods suitable to the ground

Figure 2. Herrenknecht VSM machine at the BSRP South Shaft

conditions, including caisson, slurry diaphragm wall, and ground freezing. The contractor was free to choose the other variables, including the shaft diameter.

Contractor Design

JWF originally planned to construct the 41 m deep (136 ft) (minimum finished depth) South Shaft using slurry wall techniques, assuming a shaft outside diameter (OD) of 17.7 m (58 ft). However, cost disputes with its slurry wall subcontractor, blamed on the delay between the contract bid and award dates, forced JWF to choose a different construction technique. Several options were investigated to construct the shaft, including ground freezing. Eventually, a 9 m ID (29.5 ft) caisson was chosen, to be installed using Herrenknecht's vertical shaft machine.

Vertical Shaft Machine

The vertical shaft machine (VSM) is a mechanized piece of equipment developed by Herrenknecht to excavate shafts in a variety of soil or rock conditions. The type used on the Ballard Siphon project is designed for soil applications, and is capable of excavating a shaft with a 9 m ID (Figure 2).

Current VSM capabilities include shafts with internal diameters ranging from 5.5 m to 12 m (18.0–39.4 ft), and depths of up to 85 m (280 ft). This was the first time a VSM had been used in North America, so to ensure the shaft would be installed by qualified and experienced personnel, JWF partnered with Herrenknecht to complete the shaft. The soil version of the VSM is essentially a modified caisson sinking operation, with the modifications consisting of how the shaft is excavated, and how the shaft lining, or caisson, sinks.

Ring Beam

The first step in building a shaft using the VSM is construction of the ring beam, which is essentially a foundation to support the VSM. Because of the VSM machinery supports the entire weight of the caisson during shaft installation, the ring beam is much more robust than the typical guide walls used for slurry panel or secant pile operations.

The ring beam for the South Shaft had a rectangular cross section, with a depth of 1.8 m (6 ft), and a width of 1.75 m (5.75 ft). Eight #8 steel bars ran circumferentially around the outside face of the ring beam; nine or eleven #8 bars were present around the inside face, depending on location; six or eight # 8 bars ran around the top of the

Figure 3. VSM excavation assembly

Figure 4. VSM excavation cutterhead

beam, depending on location; and ten or thirteen #11 bars were present around the bottom of the beam, depending on location. These were all tied together with #6 ties at 140 mm (5.5 in.) spacing around the entire circumference. The design required concrete with a 34.5 MPa (5,000 psi) 28-day strength. The internal diameter of the ring beam was 9.9 m (32.47 ft), allowing for a 49.5 mm (1.95 in.) gap between the caisson outside diameter and the ring beam internal diameter.

Shaft Excavation

Shaft excavation is accomplished with a mechanized excavation arm attached to the bottom of the shaft lining system (Figure 3). The excavation arm is centrally located in the shaft, and attached to the shaft walls at six locations via three gripper arms where embedded steel plates are fitted with shoes to allow the excavator to be raised and lowered as needed. The excavation arm is capable of rotating 180 degrees around the shaft, and pivots from the center of the shaft towards the edge. A horizontal rotating cutterhead (Figure 4), very similar to what is found on a typical roadheader, is positioned at the end of the excavation arm. As the soil is excavated, it is hydraulically lifted as slurry to a soil separation plant located at the ground surface. Because of the hydraulic lifting of the shaft muck, excavation is completed in the wet. Following a trip through a typical soil separation plant, relatively soil-free water is recirculated back to the shaft to maintain the desired water level.

A major advantage of the VSM excavation arm is its capability to excavate beyond the outside diameter of the shaft lining. This is useful during shaft sinking to significantly reduce the friction levels along the outside of the shaft lining, allowing caisson installations at greater depths, and in harder soils, compared to traditional caisson installation methods. This overexcavation capability also has significant advantages related to the design and installation of structural invert slabs, which is discussed below. The VSM used on the Ballard Siphon project was capable of excavating about 500 mm (19.5 in.) beyond the shaft lining extrados.

Shaft Sinking

Caisson sinking is another VSM modification. Traditional caissons have a cutting shoe at the bottom of the caisson, and the shaft sinks via controlled bearing failures as material is excavated from within the shaft adjacent to the cutting shoe. Bentonite is often injected along the outside of the shaft to keep friction forces low enough to allow the shaft to continue sinking. Shaft depth is a limitation for traditional caisson installations because of increasing friction along the outside of the caisson as it sinks deeper, and the difficulty of maintaining shaft verticality at large depths. Soil hardness is also a

Figure 5. VSM hydraulic caisson jack (1 of 4)

Figure 6. VSM excavation assembly winch (1 of 3)

limitation for traditional caissons because of the need for controlled bearing failures at the cutting shoe in order to keep the shaft sinking.

The caisson used with the VSM still has a cutting shoe at the bottom; however, the overexcavation capability of the VSM means the shaft sinking does not entirely rely upon it. Overexcavation below the cutting shoe negates the need for the localized bearing failures at the shoe to keep the shaft sinking, allowing caisson installation in harder soils. The overexcavation beyond the shaft outside diameter reduces the friction forces that develop along the outside of the shaft, allowing for deeper caisson installations.

The VSM excavation arm cuts beneath the caisson cutting shoe; therefore, the caisson itself must be independently supported during the entire installation process. This is achieved by four large hydraulic jack assemblies (Figure 5), which sit at the ground surface and contain cable bundles that attach to steel shoes embedded within the bottom of the caisson lining system. The cable bundles travel from the bottom of the caisson to the surface hydraulic jacks along the outside of the lining system, support the caisson during the excavation cycle, and then lower the caisson incrementally as needed. The four jacks operate independently, and are used in conjunction with inclinometers mounted inside the caisson lining to control the verticality of the shaft during installation. This allows for much greater control of shaft verticality than with traditional sinking methods.

Throughout excavation, three surface winches (Figure 6) hold cables that attach to the VSM excavation assembly. These winches are capable of lifting the excavation assembly if maintenance or repairs are required, and they retrieve the excavation assembly following completion of shaft excavation. Raising and lowering the excavation assembly is relatively quick, as remotely operated pins release the gripper arms from the steel support shoes, and the entire assembly travels along a simple guide system installed inside the caisson as the shaft is sunk.

Shaft Lining

The caissons walls installed with VSM can be constructed with precast concrete segments, or in lifts with cast-in-place concrete as is common for typical caisson construction. JWF chose to use the precast segmental lining, constructed using forms supplied by Herrenknecht. The segments were cast by Bethlehem Pre-Cast at its Cashmere, WA facility.

Segment dimensions were predetermined, based on the geometry of the existing Herrenknecht forms. Each of the 45 segment rings had an ID of 9 m (29.5 ft), a height of 1 m (3.3 ft), and contained four individual segments. The segments were 400 mm

(15.8 in.) thick, and contained a continuous EPDM rubber gasket around the perimeter to prevent groundwater intrusion at the segment boundaries. Segment rings were attached across the horizontal joints by eight (two per segment) equally spaced and continuous 18 mm (0.7 in.) steel rods, joined by couplers at the joints. To aid with segment installation, the circumferential face of each segment also contained three centering dowels, similar to the type used with tunnel segmental lining systems. Individual segments were connected to each other across the vertical joints by two angled 24 mm diameter (0.9 in.) bolts. Finally, each segment contained a 25 mm (1 in.) grout port near its center point to facilitate bentonite injection during shaft sinking, and grout injection during shaft completion.

Almost all of the segment rings were constructed with steel fibers, which comprised the majority of the concrete reinforcement. Steel bar reinforcement was only used around the lifting lugs to allow the segments to be removed from the forms prior to the concrete reaching its full design strength. The concrete 28-day compressive strength was 41.4 MPa (6,000 psi), and a dosage rate of 38.6 kg/m^3 (65 lb/yd^3) of concrete was required for the steel fibers.

Segment rings #1, #3, and #5 (numbered from the bottom of the caisson up) contained traditional reinforcement necessitated by the additional loads imposed upon them. Ring #1 served as the caisson cutting shoe, while rings #3 and #5 contained the steel embeds that ultimately supported the VSM excavator assembly.

Shaft Invert

Hydrostatic uplift is very often a concern for deep shaft invert slabs. The capability of the VSM to excavate below the caisson and its cutting shoe, and beyond the outside diameter of the caisson walls, allows unique design options for an invert slab that can resist significant hydrostatic uplift forces. Long-term depressurization of the groundwater surrounding the South Shaft was not an option, so a structural invert slab capable of resisting about 42.7 m (140 ft) of hydrostatic pressure was necessary. In order to accomplish this, the excavation for the invert slab was extended below the caisson bottom and past the caisson outside diameter, aided by relatively stable soils at the shaft bottom and the wet excavation method. The radial motion of the VSM excavation arm meant the invert slab excavation was shaped like an inverted dome, with the excavation bottom at the center of the shaft extending approximately 3 m (10 ft) below the caisson cutting shoe, and the excavation bottom at the caisson wall extending about 1.5 m (5 ft) below the cutting shoe. The maximum thickness of the completed concrete invert slab was 3.9 m (13 ft) at the shaft center point because of the invert concrete extending several feet into the shaft lining.

The invert slab excavation also extended approximately 500 mm (19 in.) beyond the outside diameter of the caisson walls for the full shaft perimeter. This plug geometry allowed the invert concrete to encapsulate the bottom of the caisson lining, forming an inverted “champagne cork” shaped plug capable of resisting the hydrostatic uplift forces. The invert slab overpour also aided the completed shaft structure in resisting the hydrostatic uplift forces, and the concrete encapsulation of the caisson bottom proved very effective in limiting groundwater intrusion into the shaft at the base. The monolithic shaft invert was unreinforced, constructed using 34.5 MPa strength (5,000 psi) concrete, and was placed in the wet to maintain stability of the shaft bottom.

CONSTRUCTION

Construction of the South Shaft began on January 31, 2012, and the shaft was fully completed by around the end of June 2012. The following sections chronicle the major activities involved in the shaft construction. Figure 7 shows the duration of shaft construction in calendar days.

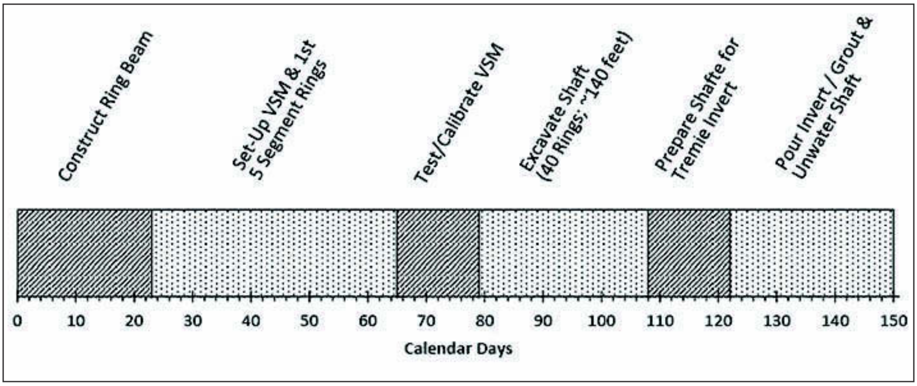

Figure 7. South Shaft construction duration

Ring Beam

As detailed in Section 3, the first step in building a shaft using the VSM is construction of the ring beam, which is a foundation to support the VSM. The ring beam for the South Shaft had a rectangular cross section, with a depth of 1.8 m (6 ft), and a width of 1.75 m (5.75 ft). Ring beam construction took 22 calendar days, beginning on January 31, 2012, and ending on February 22, 2012.

Figure 8. VSM excavation arm during calibration

Setup and Calibration

Setup for the VSM operation began by installing five segmental lining rings, necessary because the VSM excavation machinery is supported by rings #3 and #5. The bottom two rings sit within the ring beam, while the top three extend above it (see Figure 2). Following assembly, the excavation unit was lowered into the completed segmental rings, the necessary services were attached, and the excavation equipment was calibrated and tested. Figure 8 shows the VSM excavation arm during calibration and testing.

The complete setup and calibration process lasted 56 calendar days, beginning February 23, 2012, and ending April 18, 2012. Within that period, ring installation and VSM assembly took about 35 calendar days; attachment of the excavation machinery to the segments and connection of the required services (cables, hoses, power, etc.), required about 7 calendar days; and testing and calibration took 14 calendar days.

Shaft Excavation

Production shaft excavation began on April 19, 2012, and finished on May 17, 2012, requiring 29 calendar days (25 working days at 6 days per week) to excavate and install 40 rings and excavate for the invert slab, totaling about 42.7 m (140 ft) of excavation. This is an average of 1.7 m (5.6 ft) per working day; however, excavation for the invert slab took longer than expected because of the time necessary to separate the clay spoils from the shaft water as discussed below. Installation of three segmental rings (3 m [9.8 ft]) was achieved on most of the production excavation days.

The primary issue encountered during shaft excavation was the ability of the soil separation plant to effectively remove the clay from the water prior to it being returned back to the shaft, due to its fine-grained nature. JWF's soil separation system consisted of two 38,854 L (10,000 gal) slurry tanks, one vertical clarifier, one Derrick flo-line primer, three Brandt LCD2 shakers, one Cobra shaker with 380 mm (15 in.) cones, one Derrick DE 7200 centrifuge, and two Brandt CF2 centrifuges.

It was discovered early that excavation had to be stopped if the specific gravity of the slurry leaving the soil separation plant exceeded about 1.2, because at that value just as much material was being returned to the shaft as was being excavated. There were multiple days when the separation plant was left running after hours to remove material from the shaft water, and when excavation had to be stopped because of high slurry specific gravity readings. This issue became more pronounced as the shaft got deeper and the volume of water in the shaft increased, explaining why the excavation for the invert slab took longer than expected. Despite this issue, 42.7 m (140 ft) of shaft were excavated and lining installed in less than a month.

One other minor issue came up during shaft excavation. At a depth of approximately 21.3 m (70 ft), very dense granular soils, or possibly a large boulder or zone of nested cobbles, were encountered. It was difficult to determine exactly what was being encountered by inspecting the cuttings. This zone was present for several feet, and caused the slurry intake pipe at the bottom of the shaft to become clogged. The clog was quickly cleared in less than 4 hours by winching the excavation machinery to the surface. Neither the VSM machinery nor the shaft lining system suffered any damage as a result of encountering this zone, and production was not significantly impacted other than by the short delay to clean the clogged intake pipe.

Invert Placement and Shaft Completion

Preparation for placement of the structural invert slab included removal of the VSM equipment and cleaning of the water within the shaft, both of which happened simultaneously. Removal of the VSM equipment, including all equipment except for the hydraulic jacks and cables still supporting the caisson, was completed in approximately four days. Following VSM removal, an access platform was placed across the shaft and two tremie pipes were installed on either side of the platform, both of which were used during placement of the invert slab concrete. One day prior to concrete placement, sonar measurements were taken of the invert slab excavation to confirm that it had not collapsed. The entire preparation process required 14 calendar days, and the invert slab was placed on June 1, 2012. A total of 350 m^3 (459 yd^3) of concrete were placed over approximately 5 hours and 20 minutes, using two tremie pipes and two pump trucks.

The final step in construction of the South Shaft involved grouting around the caisson lining, unwatering the shaft, and cleaning the muck from the shaft bottom. Because of difficulties in effectively removing the fine-grained soil from the shaft water, approximately 2.4 to 3.0 m (8 to 10 ft) of very loose material had settled out on top of the invert slab and had to be removed. Grout was pumped through several grout ports embedded into the segmental lining near the bottom of the shaft prior to the shaft unwatering. This entire process was completed in approximately 28 calendar days; however, it could have been completed in a much shorter time if it would have been critical to the schedule. Figure 9 shows a view of the completed shaft.

Figure 9. Looking down the completed South Shaft

SUMMARY AND LESSONS LEARNED

The Herreknecht VSM proved to be an effective and efficient method for excavating the South Shaft on the Ballard Siphon Project. No major problems were encountered during shaft excavation, and the 9 m diameter (29.5 ft), 47.3 m deep (156 ft) (41 m deep [134 ft] in final configuration) was installed in 150 calendar days. The shaft could have been completed even faster if the schedule had required it, and if this had not been the first time the VSM was used locally. In addition, the completed shaft verticality was very accurate, and groundwater infiltration into the completed shaft was minor, at about 8 to 12 L/min (2–3 gpm), despite the shaft being subjected to about 35 m (115 ft) of groundwater head. The following is a summary of observations and lessons learned from the Ballard Siphon South Shaft installation:

- The Herrenknecht VSM is an efficient method for installing deep caissons in difficult soil conditions.
- The capability of the excavation assembly to excavate below and beyond the outside diameter of the caisson lining allows for unique solutions to designing a structural invert slab to resist hydrostatic uplift forces.
- Effective soil separation is critical to maintaining shaft excavation rates.
- The 150-calendar-day installation time could have been reduced by 30-plus days, had the schedule required.

REFERENCE

King County Department of Natural Resources and Parks—Wastewater Treatment Division. 2009. Ballard Siphon Replacement Project Rebid Contract Documents, Contract C00507C10, Seattle, WA, US.

MULTI-MODE TBMs—STATE OF THE ART AND RECENT DEVELOPMENTS

Werner Burger ▪ Herrenknecht AG

ABSTRACT

Tunnel alignments with variable ground conditions have become commonplace challenges for many underground projects. The conditions along the course of the tunnel often range from stable rock faces to soft, water bearing soils. Standard technologies for shielded TBMs have been optimized to handle a wider range of specific ground conditions. Technical and commercial limits are often reached when variable ground conditions become too extensive. Multi-Mode or hybrid machines incorporate the possibility to operate in different modes and therefore adapt the excavation technology in the tunnel to the actual ground conditions encountered. The paper will highlight case histories and latest developments, especially the new Herrenknecht Variable Density TBM design.

GENERAL CONSIDERATIONS

The general concept of multimode machines, i.e., shielded tunnel boring machines which are designed to operate in different modes, goes back to the early 1980s; and was to become the idea upon which the MixShield patents are based.

Basically there are three different shielded machine types:

1. open single shield for stable and usually non water-bearing ground conditions with excavation under atmospheric pressure and dry muck removal with belt conveyor
2. closed earth pressure balance machine (EPBM) for fine-grained and usually unstable and water-bearing soils with excavation under controlled positive face support pressure and thick-matter-type muck removal from the excavation chamber with screw conveyor
3. closed slurry machine (STBM) for coarse-grained and usually unstable and water-bearing soils with excavation under controlled positive face support pressure and muck removal from the excavation chamber with slurry circuit and above ground slurry treatment plant

Although a significant progress of the development in the meantime brought about large overlapping areas of the individual techniques with respect to the application range, each technique has its economically and technically optimized range of application. Therefore, there is a common objective to use the technique that is best for the individual segments of the tunnel length, especially for long tunnels with longer individual segments of different ground conditions.

Today, two basic solutions for changing the mode of operation are available on the market, which are used according to specific project requirements.

- Modular machine concept with an exchange or conversion of individual modules or subassemblies for a change of operation mode either in the shaft or inside the tunnel.

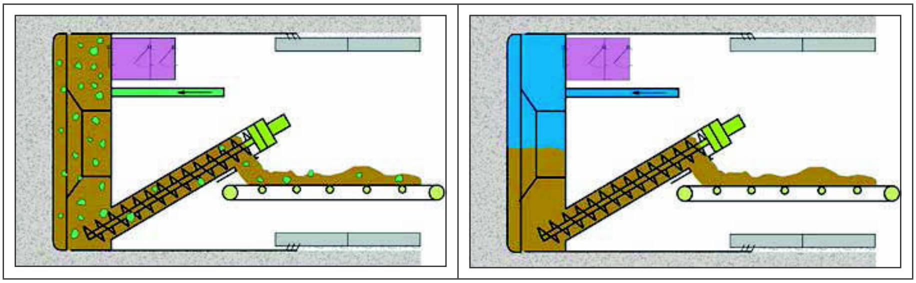

Figure 1. Earth pressure balance machine in closed mode (left) and in compressed air mode (right)

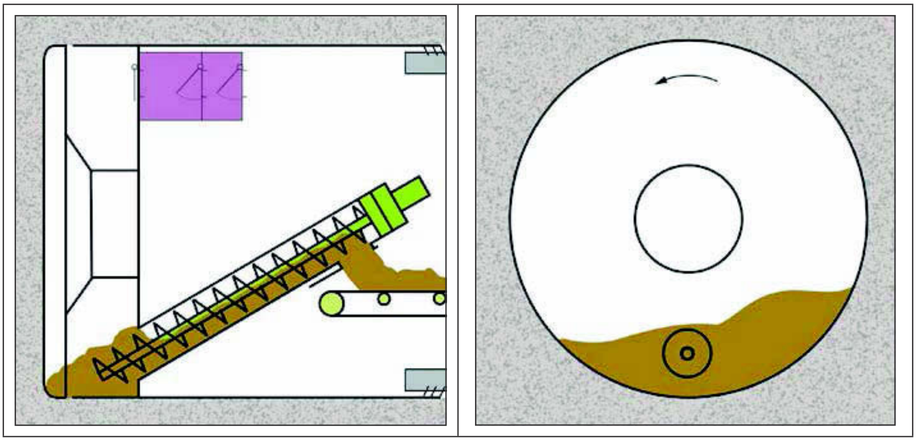

Figure 2. Earth pressure balance machine in open mode with partial filling remaining in the chamber

- Integrated machine concept with a complete parallel installation for a change of the operation mode inside the tunnel.

It is obvious that the integrated machine concept is the technically more complex solution. However, it has the advantage of faster and less labor extensive change of the operation mode.

CHANGING BETWEEN OPEN SINGLE SHIELD AND EARTH PRESSURE BALANCE MACHINE (1↔2)

An earth pressure balance machine of the classic design with screw conveyor located in the invert can without further actions and without problems change to open operation with a only partly filled excavation chamber. Theoretically, compressed air operation with partly filled excavation chamber is possible too. In all cases, the screw conveyor is used to remove the muck from the excavation chamber, but in atmospheric operation it is only used for muck transport, and not additionally for pressure control and reduction. A disadvantage, particularly for abrasive soils, is that the muck pile remains inevitably in the chamber, and the fact that the heavy screw conveyor is not the most elegant method for dry tunnel muck transportation. However, an advantage is the fact that the discharge gate of the screw conveyor can be closed immediately at any time, thus

Figure 3. Katzenberg Tunnel, machines after breakthrough

isolating the excavation chamber quickly and safely from the aft tunnel in the event of a sudden water inrush or instabilities at the face.

If the above-mentioned disadvantages shall be avoided, a belt conveyor with retractable muck hopper can be installed in the center. However, two significant challenges will have to be considered.

- The rotary coupling that is mandatory for EPB operation for the transfer of the conditioning additives into the cutting wheel, requests compromises and/or significant conversion effort for the belt conveyor–muck hopper combination.
- Since the muck buckets and channels or guide plates to be provided on or in the cutting wheel for dry operation bring considerable disadvantages, they cannot be permanently installed during earth pressure balance operation, and must therefore be removed and installed in the excavation chamber with a certain effort.

A typical project where two 11,1m EPBMs have been operated in open as well as in closed mode without any mechanical modifications was the Katzenberg Rail Tunnel in Germany (Abele, 2012). The ground conditions along the 8,9km twin tube tunnel were expected to be predominately soft rock of medium to low abrasiveness. However the low cover portal sections in partially soft soil conditions had to be excavated in closed mode as well as possible fault zones along the alignment. Due to the moderate expected abrasiveness and the uncertainty related to the number of possibly required mode changes due to faults along the alignment, the excavation and machine concept was decided for the use of the screw conveyor for primary muck transport from the chamber in both, open and closed mode operation.

The concept proved to be the right decision. Both, open and closed mode operation had to be used during the tunnel excavation with no significant stoppages required when changing between the two modes. The more unfavorable wear behavior especially related to secondary wear on the cutterhead structure could be addressed by an appropriate wear protection from the beginning and the possibility of a planned cutterhead front access for wear protection reconditioning approximately half way at a ventilation shaft location.

A totally different situation was encountered at the twin tube Saverne rail tunnel close to Strassbourg in France (Cuccaroni et al. 2011). The 3,86km twin tube tunnel alignment passes through abrasive weak sandstone. At least one portal area has to be excavated in closed mode with positive face support. Also a short section approximately half way of the tunnel was expected to present unstable face conditions and therefore may require closed mode operation as a consequence.

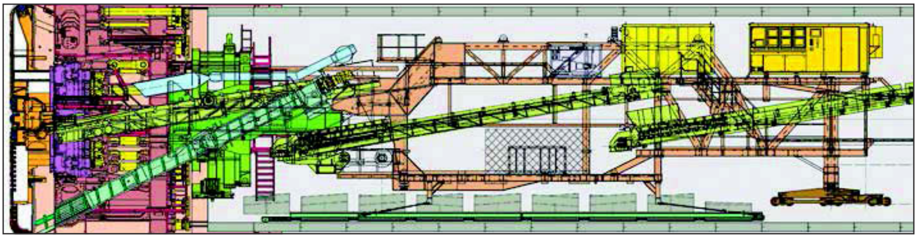

Figure 4. Saverne dual-mode machine

For these reasons the decision was taken for a convertible 10,0m EBPM with a center belt conveyor–muck hopper configuration as primary muck transport system for open mode and a bottom arranged telescopic screw conveyor for closed mode operation. Both systems are permanently installed in parallel discharging the muck onto a gantry belt conveyor behind the ring erection area.

To change from closed to open mode, some of the conditioner supply pipe installation in the cutterhead center has to be dismantled and the retractable muck hopper has to be pushed forward into the cutterhead chamber. On the rear side of the cutterhead guiding plates have to be installed for the muck transport from the periphery buckets to the muck hopper in the center. The special design of the muck guiding plates allows the bottom screw conveyor to remain operational in the partially retracted position.

During the excavation of the first drive, the mode of operation was changed from closed to open mode after passing a soft ground section at the beginning of the drive. As a result of the well designed and planned concept/process of mode change, the time required for the mechanical modifications was less than one week. After finalizing the first drive two months ahead of schedule in June 2012 the machine is currently excavating the second drive with a scheduled completion date of April 2013.

The concept for different mucking systems in open and closed mode proved to be the best solution for such abrasive rock conditions. The open mode configuration while still operational bottom arranged screw conveyor was a big advantage for invert cleaning purposes and to help overcome short sections of limited face stability.

CHANGING BETWEEN SLURRY SHIELD AND OPEN SINGLE SHIELD (1↔3)

Since a slurry shield is based on hydraulic muck removal from the excavation chamber with a slurry circuit, dry muck removal is only possible with an additional different transport system. Even in the case of a stable face and a possible compressed air—or atmospheric flushing operation with partial slurry filled excavation chamber—the transport system would still be a pressurized pumping circuit.

The installation of a belt conveyor in the center with a retractable muck hopper as a second, dry muck transport system is possible without major compromises. The same applies to a cutting wheel concept with additional muck buckets and channels or guide plates for muck transfer to the center muck hopper in dry, open mode that fulfills the functional requirements of slurry shield operation without mechanical modification.

The installation of a jaw crusher for the slurry shield operation in the invert is still possible and in no way is compromised by the convertible nature of the machine.

Depending on the technical effort for the concurrent installation of the two muck transport systems along the gantry and in the tunnel as well as above ground, such a mode change can be implemented as an integrated machine concept with the possibility of a fairly quick conversion between the operation modes.

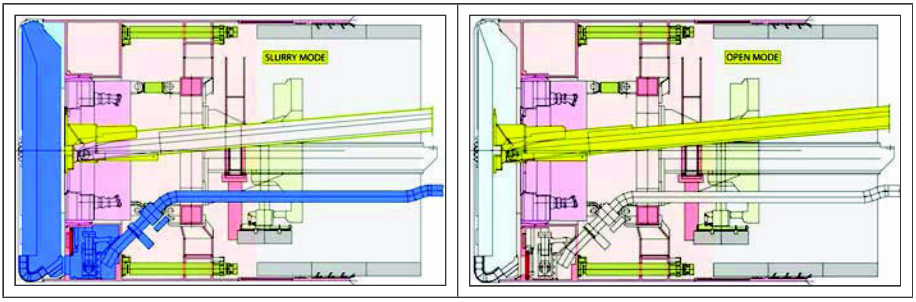

Figure 5. Integrated machine concept for change between slurry shield (left) and open single shield (right)

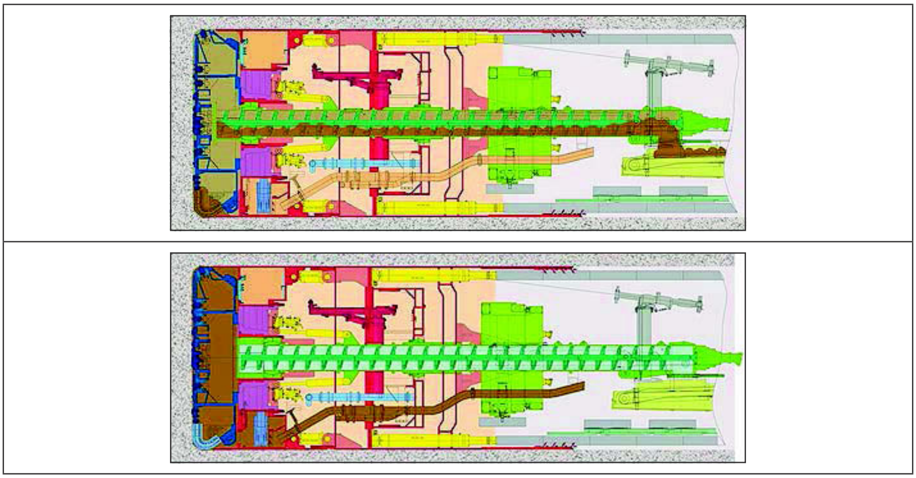

Figure 6. Machine for Lake Mead Intake No. 3, open mode with center screw conveyor (top) and closed mode with slurry system (bottom)

When operating in open mode the machine can be closed and the excavation chamber can be safely isolated from the tunnel within 2 to 4 hours by retracting the muck hopper in the center. Depending on the operational availability of the slurry circuit in the tunnel, the compressed air regulation system and the above ground slurry treatment plant a restart in closed slurry mode can take place within a few shifts.

According to project conditions, the economic aspect of such a "wet–dry" change is most interesting, making it until today the most common multimode option. Especially for tunnel alignments with major portions of soft ground below water table as well as dry rock face such a system could offer an interesting option, using the slurry mode for the soft ground and transition section and changing to open mode in the dry rock section.

In 1989, this concept saw its first application at the Grauholz tunnel in Switzerland (Steiner, Becker, 1991), and—after further development—the next generation of such convertible TBM was deployed for the Thalwil project and for the Önzberg tunnel, both in Switzerland as well. Recently two such convertible machines successfully finished the Finne tunnel (Rieker, 2010) in Germany and one the Weinberg tunnel in Zürich. Another TBM incorporating this convertible system is under operation at the challenging Hallandsas project (Burger, Dudouit, 2009) in Sweden, designed for a maximum pressure in closed mode of up to 13 bar.

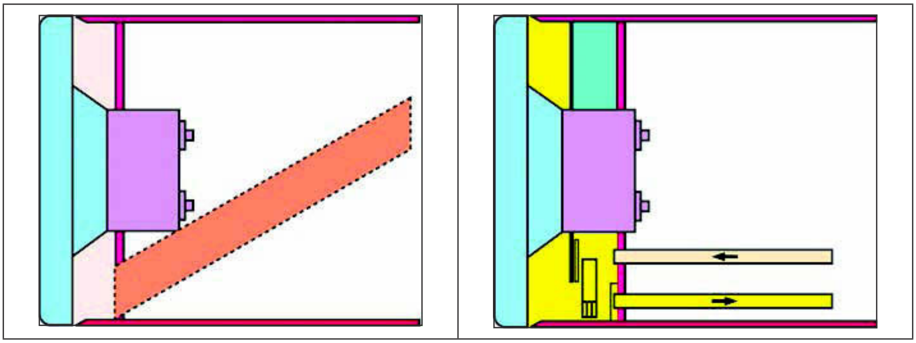

Figure 7. Basic layout of an EPB machine (left) and a slurry machine with submerged wall and air bubble (right)

Another machine that will raise the limits of application further by making potential face support pressures of up to 17 bar possible is currently excavating the water intake tunnel #3 underneath Lake Mead in Nevada, USA, featuring a screw conveyor in the center for dry (open mode) operation (McDonald, Burger, 2009). Different to the previous convertible slurry–open mode machine concepts the Lake Mead machine is fitted with a retractable screw conveyor in the center for muck transport from the excavation chamber in open mode. The installation of a screw conveyor instead of standard and more efficient belt conveyor is a consequence of the mandatory requirement for a system that can isolate the excavation chamber from the tunnel within seconds. By using a screw conveyor this can be achieved safely by hydraulically closing the screw conveyor rear discharge gate.

CHANGING BETWEEN SLURRY SHIELD AND EARTH PRESSURE BALANCE SHIELD (2↔3)

Slurry machines (STBM) and earth pressure balance machines (EPB) are operated with a filled excavation chamber under controlled pressure. The basic differences are the properties of the chamber filling such as viscosity, shear strength and density and the type of the support pressure control. The design of the cutting wheel and excavation chamber does not require any compromises between the operation modes.

The major mechanical differences are related to the muck transport and handling systems of the excavation chamber and in the tunnel. Whereas a slurry machine uses a closed, pressurized slurry circuit with above ground slurry treatment plant, the EPB machine uses a screw conveyor for controlled muck extraction from the excavation chamber and an open tunnel transport system with muck cars or continuous conveyor. Also the type of chamber/face pressure control causes mechanical differences. Face pressure control for an EPB machine is realized by controlling the operational parameters advance speed and muck extraction volume (screw conveyor speed). In a slurry machine a remote pressurized air bubble is employed to control the face pressure. Mechanically such a remote air bubble is realized in most cases by separating the excavation chamber in two compartments by a submerged wall.

For large machines both muck removal systems could be arranged in parallel in the invert area of the excavation chamber by accepting some minor functional compromises. However, if slurry operation requires the installation a jaw crusher in front of the suction grid, then this causes a significant additional mechanical effort which will require manual intervention to change the operation mode and becomes even more difficult for machines of a diameter less than 8,0m.

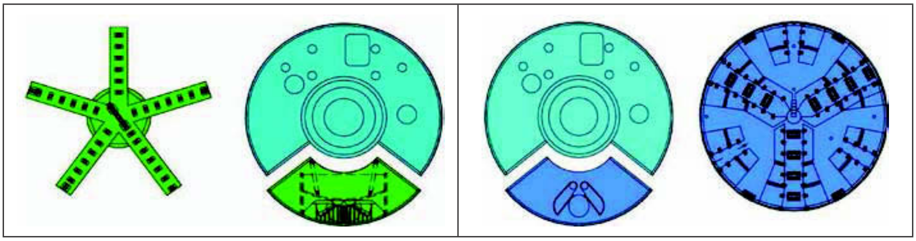

Figure 8. Modular machine concept for change between slurry machine (left) and earth pressure balance machine (right) by replacing individual TBM modules

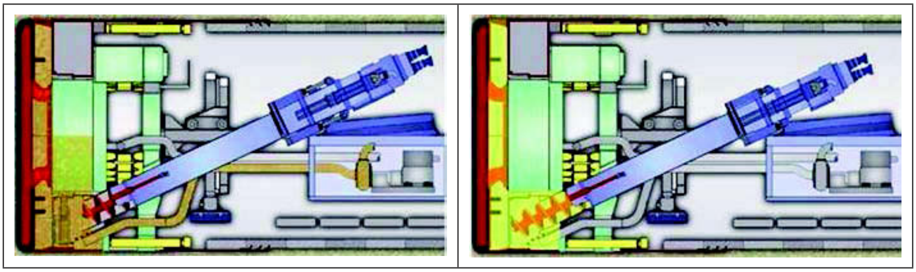

Figure 9. Socatop machine in slurry and earth pressure mode

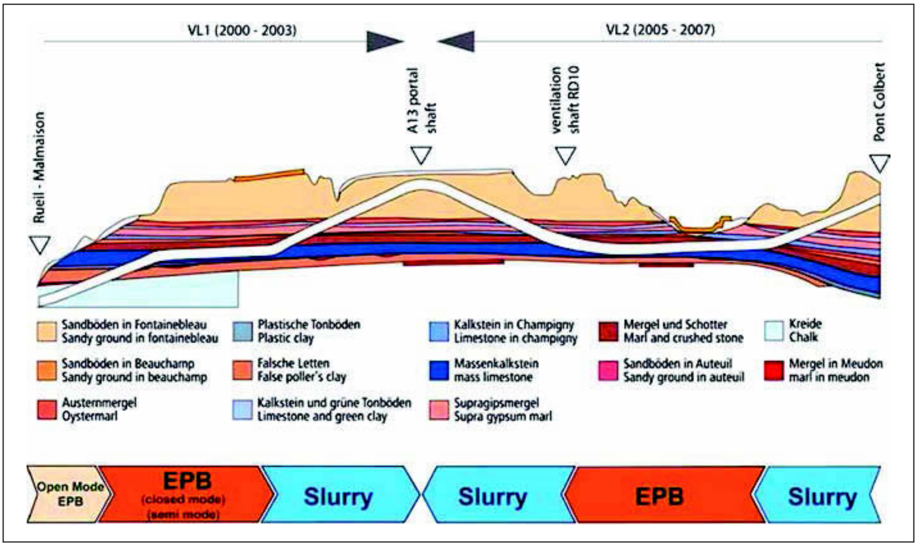

Figure 10. Socatop tunnel alignment and mode of operation for VL1 and VL2 section

Interchangeable mid size machines have therefore been realized based on the modular concept that requires the exchange of individual parts or modules of the machine in an intermediate shaft along the alignment. The modular modification concept for a system change in the tunnel has so far not been realized since it would most likely require a free air chamber access which for most projects is a very difficult and time consuming challenge.

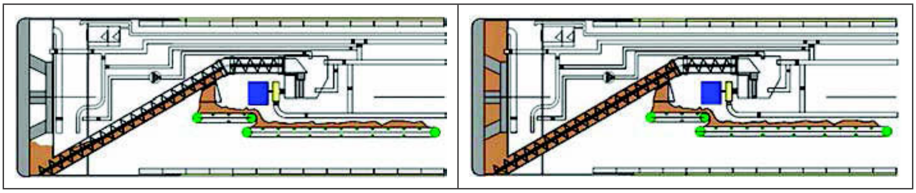

Figure 11. Variable density machine in EPB open mode (left) and EPB closed mode (right)

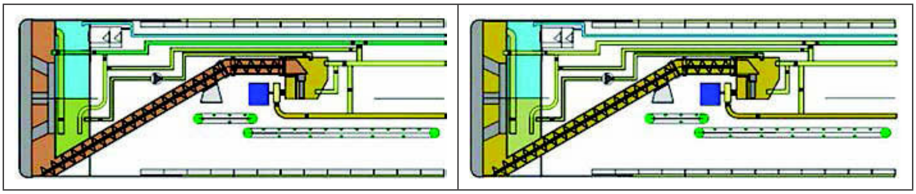

Figure 12. Variable density machine in high density slurry mode (left) and slurry mode (right)

To justify the significant effort for the machine design capable to completely change from EPB to slurry mode in the tunnel, there must be special project requirements and/or conditions. The Socatop Project in Paris (Toris, 2007), France had such conditions, with a tunnel length of 10km, approx. 60% of which in optimum soil for the utilization of an EPBM under earth pressure or compressed air support and 40% of which demanded slurry supported face as an optimum solution. The fact that the different solutions could be applied at a stretch in large individual tunnelling segments was to be the decisive factor. Although the Socatop project was to this day the one of a kind, it showed conclusively that a complex combination of different technologies can make sense if the project circumstances are right.

THE VARIABLE DENSITY MACHINE CONCEPT

In light of the fact the change from a slurry supported face to an earth pressure supported face being the most difficult for practical realization Herrenknecht AG has been working on the development of a new machine concept and principle that combines the individual advantages of each system into one machine. The development target has been to achieve a system that can be transformed from a slurry face support into an earth pressure face support in the tunnel without any need of mechanical modification in the excavation chamber or behind in the gantry/tunnel area.

The Variable Density Machine can be operated in the classical STBM mode incorporating an air bubble system for face pressure control, as well as in a full EPB mode. The transfer between the operational modes can be done gradually under permanent and full control of the face pressure and without any need of chamber interventions. An extended operation with a high density in the excavation chamber, that would be too dense for a pure STBM but too fluid for a pure EPB is an additional possibility.

When fully equipped, the Variable Density Machine would require two muck transport systems in the tunnel, a closed slurry circuit for STBM mode and a dry system with muck cars or continuous conveyor for EPB mode. However, depending on the specific project conditions one of the two systems may be chosen to be the high performance primary system and the other, the reduced performance secondary system.

Muck extraction from the excavation chamber of Variable Density Machine is via a screw conveyor in the EPB as well as in the slurry mode. The twin screw arrangement

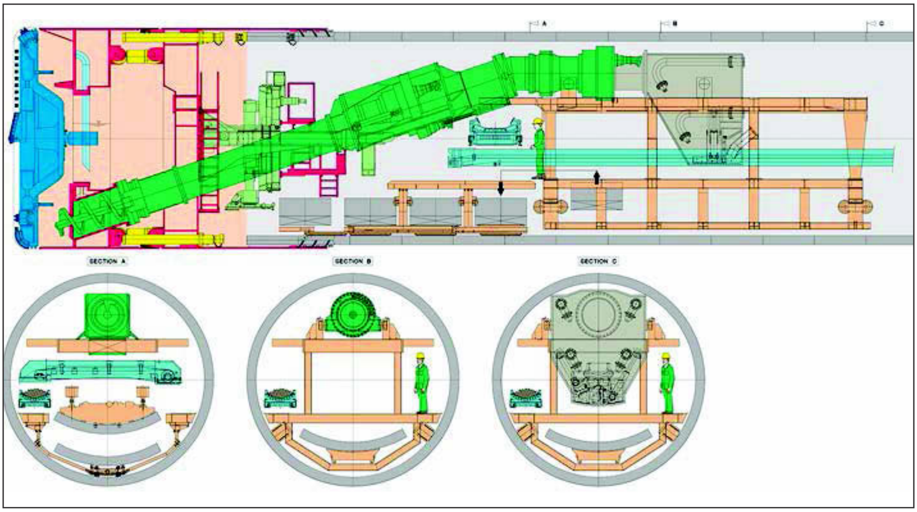

Figure 13. Typical layout of a fully equipped 6,5m variable density machine

with a flat gate between screw #1 and #2 has a muck discharge gate at the end of the first screw for muck discharge onto a belt conveyor in pressurized or open EPB mode. In slurry mode the discharge gate of screw #1 is closed, the flat gate between screw #1 and screw #2 is open and the muck is discharged into a flushing or slurrifier box at the rear end of screw #2. Inside the slurrifier box a standard jaw crusher can be installed to reduce larger particles that could pass the screw conveyor to a size suitable for the slurry circuit. After closing the screw conveyor flat gate the slurrifier box is accessible under atmospheric conditions for simplified crusher maintenance.

In slurry and high density mode the muck transfer along the screw conveyor is a combination of a mechanical and hydraulic transportation.

The air bubble chamber for the STBM operation is located in the front shield between a front and a rear bulkhead. The classical STBMs typical submerged wall opening in the invert is not existing, the connection between the pressurized air bubble volume and the excavation chamber is guaranteed by communicating pipes between the two chambers. Such a configuration has been successfully applied for earlier slurry machines designed with a so called “closed invert” concept. In EPB mode this pressurized bentonite reservoir is used as automatic chamber refill volume for additional safety in case of sudden pressure drop in the excavation chamber.

The system does not incorporate a crusher in the excavation chamber as it is possible for a standard slurry machine, therefore the cutterhead design and tool configuration had to limit the particles that can enter the excavation chamber to a size suitable for the installed screw conveyor. As far as a screw conveyor of 700 mm diameter or more can be installed this limitation does not present any difficulties as known from numerous EPB applications in bouldery conditions.

Depending on the project requirements simplified systems or partially equipped systems of the Variable Density Machine may be employed. The large slurrifier box at the rear end of screw #2 can be replaced by a smaller version with a rotary crusher. Such a configuration is actually under operation at the OARS Project in Columbus OH. Also a single screw version is possible with a rotary crusher–slurrifier box combination at the screw conveyor #1 outlet. The single screw version however requires some modification when changing between slurry mode and EPB mode. The rotary crusher–slurrifier box has to be moved into a parking position before the belt conveyor can be put in operation. Such a configuration is used at the Miami Port Tunnel machine for the so-called water control mode and in combination with a high density slurry supply system for the Klang Valley MRT project in Kuala Lumpur.

CONCLUSION

More demanding tunnel alignments either relating to tunnel length, required face pressure or variation of ground conditions require more and more flexible TBM concepts. Even the range of application for each individual classic shield machine technology has been significantly enlarged within the last decades increased safety requirements and economy will ask for further progress in providing machines that can offer the best possible system for each individual section along the alignment. Flexible solutions that require extensive modification work in the tunnel often including hyperbaric work can only be an intermediate step and not the final answer for such requirements. The introduction of the Variable Density Machine can therefore be seen as a significant step towards the next level of flexible tunnelling machinery, still following the original idea of the Herrenknecht MixShield concept.

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PART 14

New Plant and Equipment Applications

Chair

Paul Zick
Obayashi Corporation

RAPID DRILL-AND-BLAST TUNNELLING, THROUGH THE APPLICATION OF SYSTEMS ENGINEERING METHODS

Keivan Rafie ■ McNally International

ABSTRACT

The net present value of many tunnelling projects depends on the time from capital development expenditure to revenue generation from production. The more rapid the tunnelling stages the higher the project NPV. This paper investigates the application of system engineering tools on safe rapid tunnelling and illustrates the benefits and limitations of such tools in real world.

Lean manufacturing, Six Sigma, Benchmarking and Simulation implementation case studies from mine tunnels in Canada and Australia, as well as from the construction of the Channel tunnel in UK are examined. These case studies demonstrate how the repetitive cyclic nature of underground development is well suited to systems engineering methods. And, explains how systems engineering methods have been used to improve advance rates across a variety of projects.

The paper concludes by identifying the availability of reliable and appropriate data as the most challenging aspect of applying these methods and suggests a number of opportunities for developing systems engineering methods by utilizing faster and more reliable reporting systems. This approach was identified as key to sustained implementation of systems engineering methods which offers the potential to continuously improve tunneling rates by incorporating systems engineering methods into the system itself.

INTRODUCTION

According to Atlas Copco (2005), hardrock tunnelling rates have increased on average by only 24 per cent over the last 25 years (Figure 1). This paper reviews the experience of the mining and civil tunnelling contractors in applying systems engineering concepts to advance tunnelling rates. Systems engineering involves the systematic analysis and improvement of processes through the development of process maps, measurement and simulation of cycle times and application of Lean production and Six Sigma concepts to improve cycle time and work quality.

BACKGROUND

Quick tunnelling improves net present value (NPV). This is critical for big projects and mining industry where several kilometres of tunnelling is initially required at high capital cost (Suoreneni et al. 2008). This paper presents a review of system engineering applications for rapid tunnelling.

System engineering methods are the business improvement methods of choice for many manufacturing and processing industries around the world. Other systems engineering methods applied to rapid tunnelling and discussed in this paper include: lean manufacturing, six sigma, benchmarking, process mapping, simulation and standardised work.

SYSTEMS ENGINEERING METHODS

Lean Manufacturing

Lean manufacturing has its roots in the production systems developed by Toyota from the 1950s. The Production System has contributed to the rise of Toyota as one of the most successful automotive businesses in the world. "Problems" in the Toyota and Lean manufacturing view of the world, are sources of waste, where performance does not measure up to expectation. A formal definition of lean production techniques might be "the ceaseless elimination of waste" (Dunstan et al., 2006). Dunstan et al. (Dunstan et al. 2006) have done a comparison (Table 1) between resource/mineral businesses and Automotives and document a number of successful case studies of the application

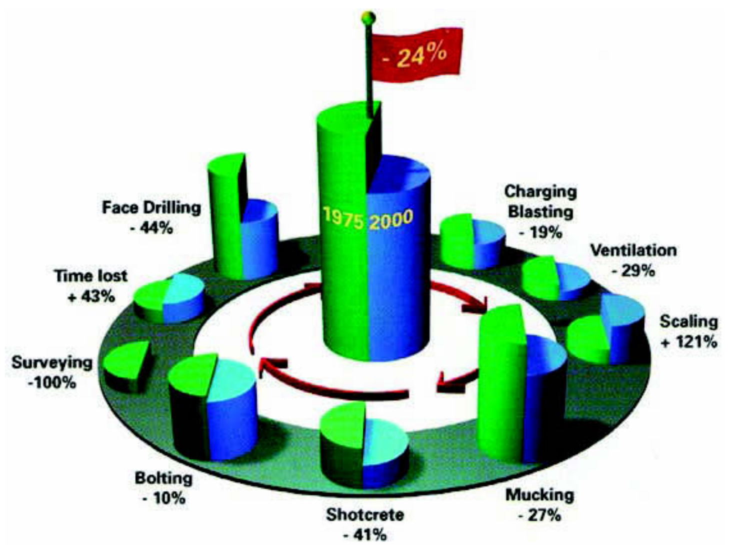

Figure 1. Atlas Copco drill and blast diagram (AtlasCopco 2005)

Table 1. Comparison between resource/minerals businesses (after Dunstan et al. 2006)

Resource and Minerals Business	Automotive Business
A smelter or refinery cannot be stopped so there is inherent production push in the process	An automotive assembly line can be stopped so there is the ability to create pull systems
Production is in continuous units and around the clock	Production is in discrete units and often on less than one day cycles
Generates considerable dust	Little dust
Physically challenging environment	Ambient conditions
Inherently variable environment	Stable work environment
Remote locations	Large centres
Impact of weather	Indoor environment
Inherently variable raw materials	Controlled raw materials
Geographically spread output teams	Compact plants
Molten metal has a short shelf life before it solidifies	Long-life components suitable for supermarket-style storage

of lean manufacturing techniques by Rio Tinto Aluminium, The Northparkes mine and the Hunter Valley Coal Operations. In practice, Lean relies on:

- Engaging workplace leaders
- Asking employees to set agreed standards for their work
- Empowering employees to write their own standards and improve them
- The visual representation of key production performance data, empowering employees at the lowest level to make operational decisions based on the data
- Forming operations and maintenance employees into manufacturing teams
- Application of a suite of business improvement tools

Lean manufacturing as a system engineering method for rapid tunnelling has limited applications because it does not consider the overall system nor does it consider interactions between processes. Because rapid tunnelling has complex interactions between processes it is unlikely that Lean manufacturing would be successful as a stand-alone method. That being said, Lean manufacturing's focus on waste would be applicable to certain processes in the tunnelling cycle where waste is a problem. For example, Lean manufacturing would be well suited to reducing wastage in particular ground control process. For example, reducing excessive bolting and shotcreting by ensuring ground support designs are responsive to conditions. However, if applied to isolated waste issues without considering the overall system then eliminating waste could adversely affect tunnelling rates. For example, an attempt to reduce shotcrete wastage could make the shotcreting process take longer thereby increasing ground control times.

Six Sigma

"Six Sigma" was pioneered by Motorola and later popularized by Jack Welsh, CEO of General Electric Corporation. Its name derives from quality control principles relating to statistical process control.

If product quality is regarded as being normally distributed, a manufacturer will typically impose an upper control limit (UCL) and lower control limit (LCL) to define an acceptable quality range. In a three sigma system, the distribution is such that plus/minus three standard deviations lie within the upper and lower control limits. Thus, using standard normal tables, it can be seen that 2,700 defective products per million (0.27%) can be expected to fail both the upper and lower limit tests (see the blue zone on the left hand of Figure 2). Furthermore, if the process is such that the mean shifts by 1.5 sigma, then the proportion failing the upper control limit will increase to 67,000 per million.

To avoid such losses, Motorola defined their desirable product quality such that plus/minus six sigma fall between the upper and lower control limits. This means that only 3.4 defects per million are acceptable at each of the distribution cut-offs. The methods chosen to achieve this aim became known as the six sigma approach to continuous improvement.

Central to the six sigma approach is the use of a structured, disciplined, rigorous approach to process improvement based on DMAIC (see Figure 3). DMAIC is an acronym meaning Define, Measure, Analyse, Improve and Control. The following explanation of the DMAIC cycle is drawn from Rath and Strong (2000):

The first phase is *Define*. The project's purpose and scope are defined. Background information on the process and customer is collected. The output of this phase is:

- A clear statement of the intended improvement (the business case and team charter)

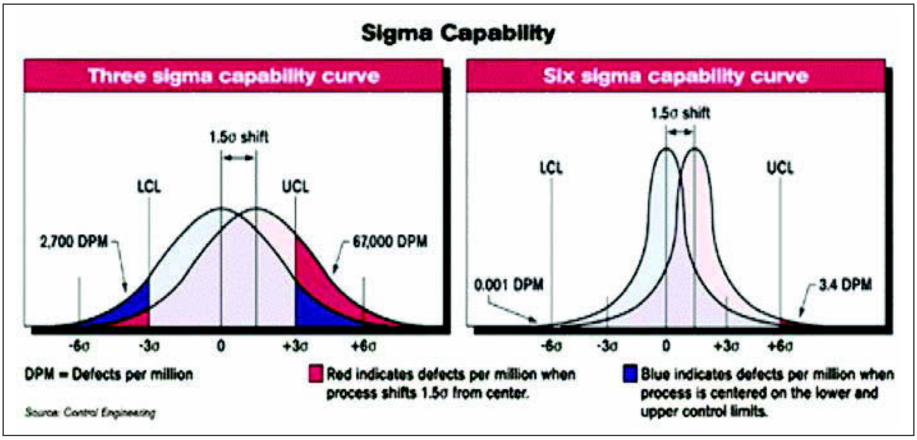

Figure 2. Six Sigma product defect capability (Source: Harrold 1999)

- A high level map of the process (this uses an input-output map called SIPOC, considering Suppliers, Inputs, Process, Outputs, and Customers)
- A list of what is important to the customer (Critical-to-Quality or CTQ factors)

The second phase is *Measure*. The goal of Measure is to focus the improvement effort by gathering information on the current situation. The output of Measure is:

- Baseline data on the current process performance
- Data that pinpoints problem location or occurrence
- A more focused problem statement

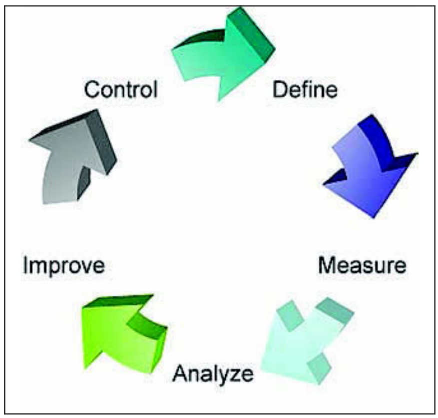

Figure 3. Six Sigma DMAIC improvement cycle (after Rath and Strong 2000)

These outputs provide the basis for the *Analyse* phase. The goal of this phase is to identify the root cause(s) and confirm them with data. The output is a theory that has been tested and confirmed. The verified cause(s) forms the basis for the next phase.

The goal of the *Improve* phase is to try out and implement solutions that address root causes. The outputs are planned, tested actions that should eliminate or reduce the impact of the identified root cause(s). Additionally, a plan is created for how the results will be evaluated in the next phase.

The goal of the *Control* phase is to evaluate the solutions and the plan, maintain the gains by standardising the process and outline steps for on-going improvements including opportunities for replication. The output is:

- Before and after analysis
- A monitoring system
- Completed documentation of results, learning and recommendations

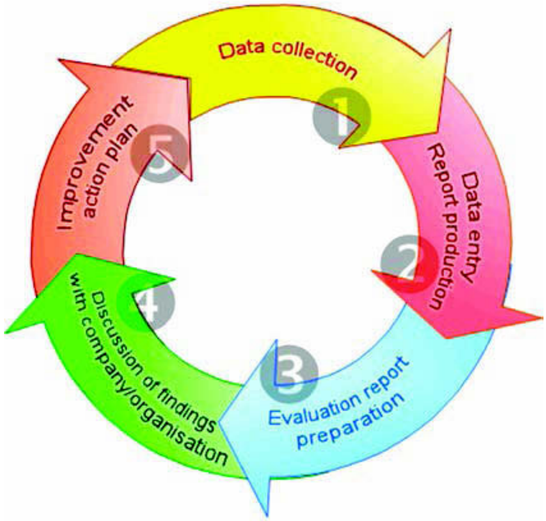

Figure 4. Benchmarking process

Like Lean Production techniques, Six Sigma draws upon a suite of business improvement tools for each of the DMAIC phases. Six Sigma relies on training a number of high level business improvement specialists within an organisation. Using martial arts terminology, these specialists are referred to as green, yellow and black belt Six Sigma practitioners.

Companies such as BHP Billiton and Caterpillar have successfully implemented six sigma business improvement programs throughout their operational units.

Benchmarking

According to Hall and Harper (2005) Benchmarking is a practical and effective method of measuring operational performance, identifying performance gaps and providing and prioritizing performance targets. Furthermore for benchmarking (or any performance improvement process) to add value, it must consider the complexities of underground mining and work within the framework of the strategic plan. Most benefit is derived from having the right plan; however the plan cannot provide value if it is not implemented in an effective and sustainable way to be successful benchmarking must adhere to a rigorous and structured process. The benchmarking process comprises the following main components. (Hall and Harper 2005): Data Collection, Data Entry and Report Production, Evaluation report preparation, Discussion of findings, Improvement action plan and on-going monitoring (Figure 4).

To add value, benchmarking must incorporate the strategic goals of the organisation into the process (Hall, AJ & Harper 2005). These goals should be linked to the underlying cost and physical drivers of operation performance. Hall (2005) argues that it will ensure that the implemented solutions will add value to the operation. Undertaking a benchmarking project is a significant commitment and it is essential that sufficient resources are allocated to the process to ensure the maximum benefit is derived (Hall and Harper 2005). The benefits derived from a properly conducted benchmarking project will often far outweigh the costs.

Hall (2005) states that benchmarking is often used by site mining personnel to assess how well mining systems and processes are operating relative to comparable sites. At this stage benchmarking emphasises processes that appear to be performing less than predictions and picks out processes where improvements could be achieved by other system engineering methods. On the other hand, benchmarking outcomes can be employed more directly as part of the solution to processes that perform less well than expected by providing samples of best practice and focusing on processes where improvements are most likely to be made.

Process Mapping

An underground mine can be considered as a process which transfers a mineral resource from the ground into a product, concentrate or metal (Hall and Harper 2005). Hall argues that the process is made up of a number of sequential process steps which transfer ore from one stock type to another. Each consecutive ore stock has a greater worth than the previous caused by less time and labour being necessary to transform the ore into a product. Hall (2005) states that to achieve the performance targets set during the planning process it is important that sufficient ore stocks are maintained to allow for the uncertainties encountered during the normal course of the underground mining process. Ore stocks need to be conserved at adequate levels for a mine to deliver the specified ore requirements in a sustainable and efficient manner to the processing plant.

Standardised Work

Variability in operating procedures within and between crews is often an accepted part of mining operations. However, this variability is the enemy of high performance (Winchester 2006). Standardised work is a rigorous procedure to standardise, document and progressively improve the way work is done and is applicable to all the other Lean tools. It is implemented through discussing existing practices for a particular work process and documenting a baseline procedure. Through 'kaizen' or brainstorming sessions or through suggestions made by employees at regular meetings, the procedure is incrementally improved Standardised procedures and adherence to them is important if a mine is to remain competitive with international best practice (Dunstan et al. 2006).

Simulation

Simulation is an efficient and cost-effective tool for decision-making and analysing real-world systems and repetitive construction processes. It models the behaviour or properties of processes to predict outcome. Simulation is especially useful where there are complex interactions between processes making analytical solutions too complex to calculate.

Tunnelling and trenchless construction processes are excellent candidates for the utilization of computer simulation due to their repetitive nature. Management of infrastructure, underground, or pipeline projects is challenging because of inherent uncertainties. The most effective way to deal with uncertainty is to collect supplementary information and knowledge. When expensive or infeasible, quantification of uncertainty may be performed using analytical or simulation techniques.

In mining operations simulations have been carried out for many years (Hall, 2000). Hall (2000) comments that simulation is well suited to evaluating the effect of changes in complex dynamic and interrelated systems. Engineering processes can be simulated using a vast array of commercially available computer programs.

RAPID TUNNELLING APPLICATIONS

Lean Production—The North-Parkes Mines Experience (Rio Tinto Practice)

Barry Lavin (Managing Director Northparkes Mines) reports:

> *"Northparkes Mines, an underground block-caving copper mining operation in central New South Wales, recently began developing first stage of a new underground mine at its E48 project. This involves excavating 10000 meters of tunnels using conventional drill and blast mining methods. Reintroduction of underground development presented challenges to project team. The majority of issues were associated with mine services, equipment and work procedures and many of them were recurring."*

Development of underground excavations follows a cyclical process that is repeated every 12 to 24 hours (Dunstan et al. 2006). The tunnelling cycle, undertaken by a crew of five or six miners, consists of:

- Drilling a pattern of blast holes into the rock face;
- Charging blast holes with explosives and firing;
- Ventilation (Cleaning blasting fumes)
- Mucking out broken rock; and
- Supporting the new section of tunnel with ground support elements including meshing, rock bolts and spray-on concrete

Advance rates vary between three and five metres per cycle. This was the task that Lean was called in to control and improve. A key feature of Lean is its ability to manage a large number and variety of issues simultaneously using visual prompts to assist the communication of issues. A Lean Information Centre was established in the project's shift change centre (Figure 5). The metrics that the tunnelling teams chose to track were safety, environment, employee availability, cycle completion times, weekly tunnelling targets and utilisation of resources (Dunstan et al. 2006).

Lean has proved to be a flexible and adaptive management tool. It is currently being used to track more than 100 issues simultaneously (Dunstan et al. 2006). It also allows for communication of tunnelling rates and metrics. This could improve communication with team leaders and crew members and let them see where issues are occurring. As a result, crew members are more willing to contribute to identifying and solving issues that cause delays in the production cycle.

The Lean process facilitates a structured response to productivity issues, which has improved the efficiency and effectiveness of shift changes. Overall, the benefits derived from implementing Lean Information Centres at Northparkes have been significant (Dunstan et al. 2006), with the process contributing to a 56 per cent improvement in the cycle time within the first 30 days of adoption (Figure 6). They have provided a structured approach to improving productivity. The main benefits are that tunnelling targets and performance against those targets are visible. Tunnelling teams are actively involved in identifying and solving causes of delay.

Figure 5. Lean information centre (after Dunstan, Lavin & Sanford 2006)

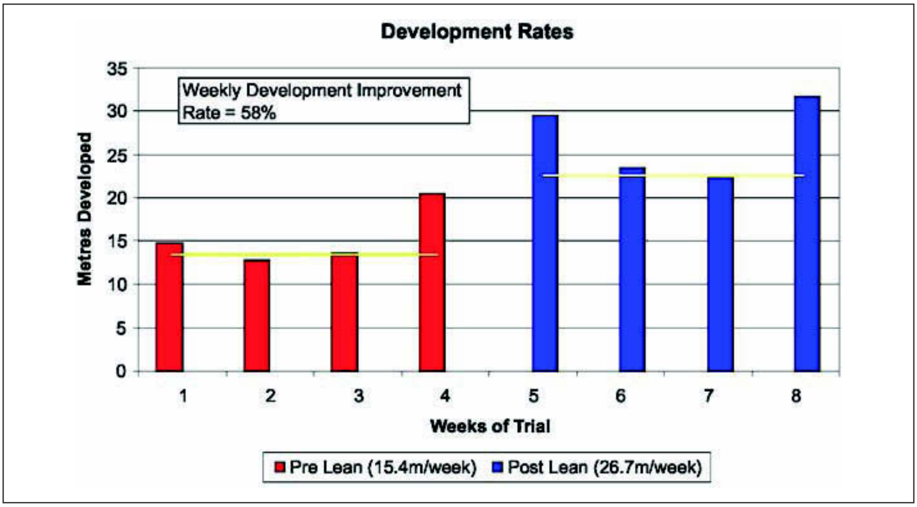

Figure 6. Northparkes tunnelling rates (after Dunstan, Lavin & Sanford 2006)

Six Sigma Application—Cadia East Rapid Tunnelling Technologies

Willcox (2008) Reports on a pre-feasibility study being undertaken by Newcrest Mining Limited, the Cadia East implementation team has developed an access decline to the proposed underground operation. Willcox (2008) discuss the components of Six Sigma methodology were applied to support the systematic changes and demonstrated that tunnelling rates improved 60 per cent above the comparable single heading benchmark. The initial step involved breaking down the tunnelling cycle into the elements. The initial forecast of cycle time is 12 hours based on these elements (Table 2).

Table 2. Tunnelling cycle time (after Willcox 2008)

Element	Time
Fumes clearance	30 mins
Water and inspect	30 mins
Bog # 1 (300 t)	90 mins
Hydroscale and shotcrete	100 mins
Bolt (27 bolts)	120 mins
Bog # 2 (200 t), clean up	90 mins
Drill face (70 holes)	150 mins
Charge and fire	90 mins
Total time	12 hrs
Advance (assume 85%)	269 m/rn

Willcox (2008) found a number of improvements through lateral thinking exercises, by breaking down face utilisation and face efficiencies and their contribution to the advance rate. The potential improvements were then ranked using impact, likelihood and `Pareto' rankings. Cycle times and the individual components were analysed for each month with comparison to expectations. Common cause events such as pumping issues (Figure 7) were identified, with positive and negative contributions to cycle times discussed and actioned.

Box plots (Figure 8) were used as an additional graphical method to present cycle components, essentially showing the distribution of the data by using the median, quartiles and the extremes. The box shows the middle 50 per cent of the data.

Overall Willcox (2008) found Six Sigma improvement processes have supported the adoption of emerging technologies at Cadia East. Accurate long round, high performance drills coupled with emulsion explosives and high-capacity materials handling

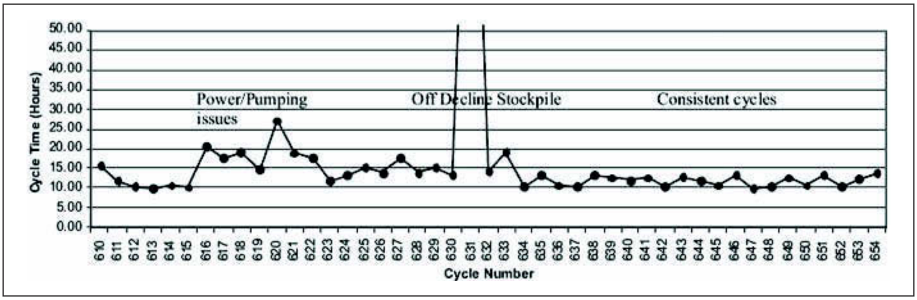

Figure 7. Cycle times—December 2006 (after Willcox 2008)

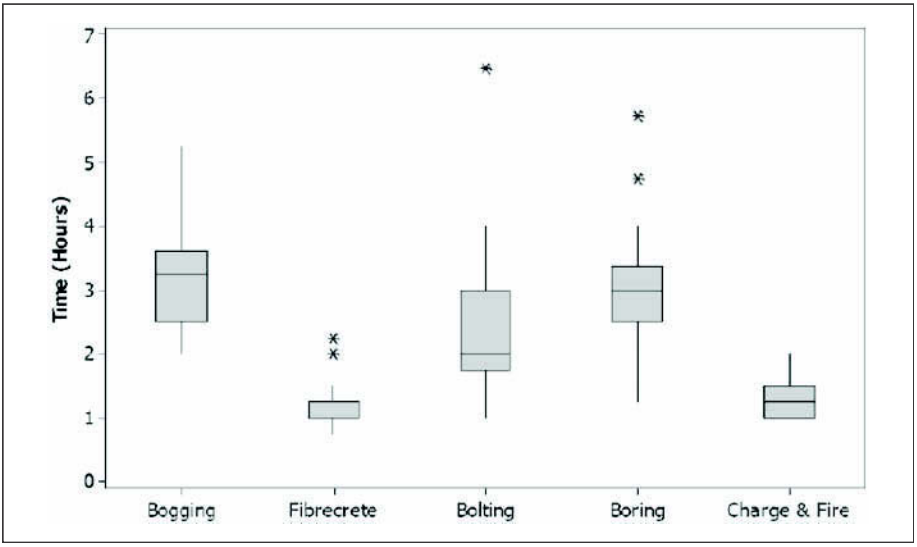

Figure 8. Box plot for cycle components—December 2006 (after Willcox 2008)

have demonstrated single heading tunnelling rates over 8 m/d (50 per cent above the current Australian benchmark of 5.25 m/d) are now practically possible (Figure 9).

Benchmarking

Table 3 contains 8 drill and blast tunnelling case studies used to estimate underground development benchmarks. Neumann (2001) collated the majority of the case studies presented in Table 3. The median advance rate for the 8 case studies was 7.0 m/day and the average was 6.8 m/day. It is important to note that Table 3 contains both single and multiple heading tunnelling case studies. Multiple heading developments have faster average advance rates because of better equipment utilisation. Differences between mines can also be attributed to differing operational, productivity and cost priorities (Neumann 2001).

Benchmarking of not just the overall system performance, but also the individual processes across numerous operations has identified ground support as the process with the most potential to increase tunnelling rates. A survey by Laurentian University Mining Automation Laboratory (LUMAL, 1997) Figure 10 shows that the greatest

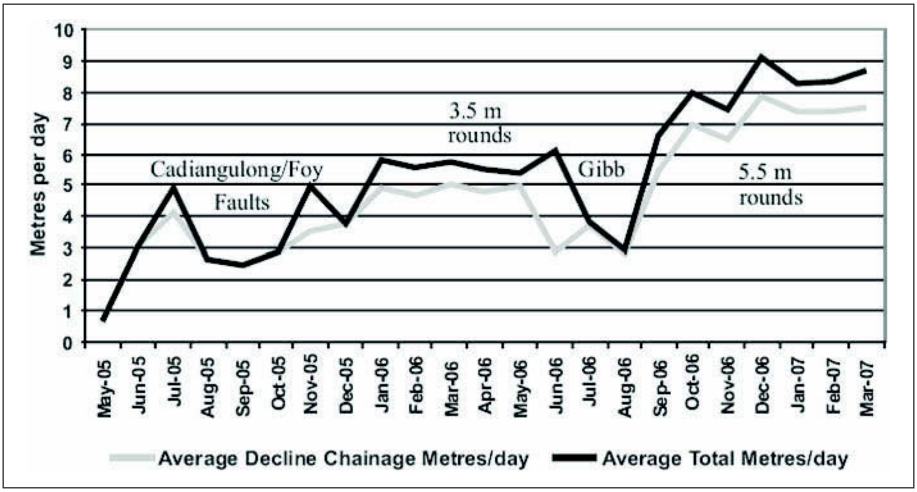

Figure 9. Cadia East single heading tunnelling rates 2005–2007 (after Willcox 2008)

Table 3. Drill and blast benchmark case studies (Nuemann 2001; Stewart et al., 2006)

Case Study	Country	Average Advance Rate
PT Freeport (Barber et al. 2005)	Indonesia	9.0 m/day (63 m/week)
Craviale Tunnel (Kalamaras et al. 2005)	Italy	5.5 m/day (38.5/week)
Kidd Creek mine (Neumann 2001)	Canada	5.3 m/day (37 m/week)
Holt McDermott mine (Neumann 2001)	Canada	7.2 m/day (50 m/week)
Creighton mine (Neumann 2001)	Canada	5.0 m/day (35 m/week)
Brunswick mine (Neumann 2001)	Canada	5.8 m/day (40.6 m/week)
Dome mine (Neumann 2001)	Canada	7.4 m/day (51.8 m/week)
Musselwhite mine (Neumann 2001)	Canada	8.9 m/day (62.3 m/week)

amount of tunnelling cycle time (36–46%) is spent on support installation. This observation is supported by evidence presented (Peake and Rupprecht 2002) from the South African underground mines. For 30 years the Norwegian University of Science and Technology (formerly known as University of Trondheim, The Norwegian Institute of technology) has been collated, analysed and reported on tunnelling design, performance and cost data for both drill and blast and TBM tunnelling. These studies indicate that for a 6m by 5m face, ground control comprises 32% of the tunnelling cycle time (Figure 11) (Johannesson 1995).

Process Mapping Application—Channel Tunnel

At the Channel Tunnel Rail Link project located in the United Kingdom, contractors responsible for rebuilding St. Pancras Station are integrating Lean Construction and Six Sigma in order to achieve critical construction milestones (Koerckel and Ballard 2005). These include distributed real-time production planning and control; tunnelling, use and continuous improvement of standard processes; active measurements of the planning system performance and action on root causes of failures; and cross-functional collaboration.

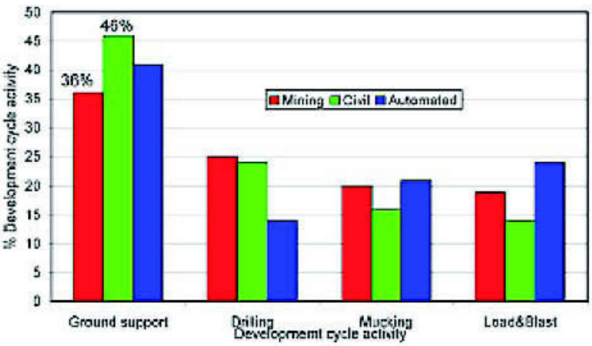

Figure 10. Comparison of tunnelling cycle activity times in drill and blast (Source: Peake & Rupprecht 2002)

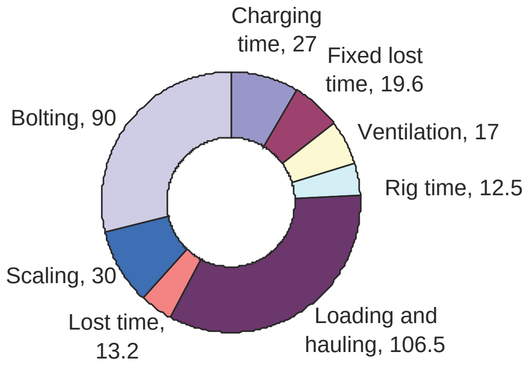

Figure 11. Cycle time times for drill and blast tunnelling for a 6m by 5m face based on NTNU Prognosis for 30m^2 prognosis. Total cycle time=375 minutes. Ground control (scaling and bolting) represents 32% of cycle time.

Strategic Project Solutions (SPS) has developed production control software for implementing the Last Planner System (Ballard 2000) along with other "lean" and modern business principles and theories. The SPS software, SPS Production Manager, is a web resident database, allowing coordination across all specialists, those on site and off site, and enabling data collection and analysis.

According to Koerckel, Ballard and Espanad (2005), all work groups met daily to review and commit to a production plan for the day and to record completions and non-completions for the previous day. The "work flow reliability" for the project, shown in Figure 12, has improved from 70% to 80% over an 18 month period. Notable also is the reduction in variation.

On top of these individual items, by using SPS Production Manager & 3D prototyping to improve their control of the works and their short term planning, the West Deck team has targeted a 10% productivity improvement over the East Deck.

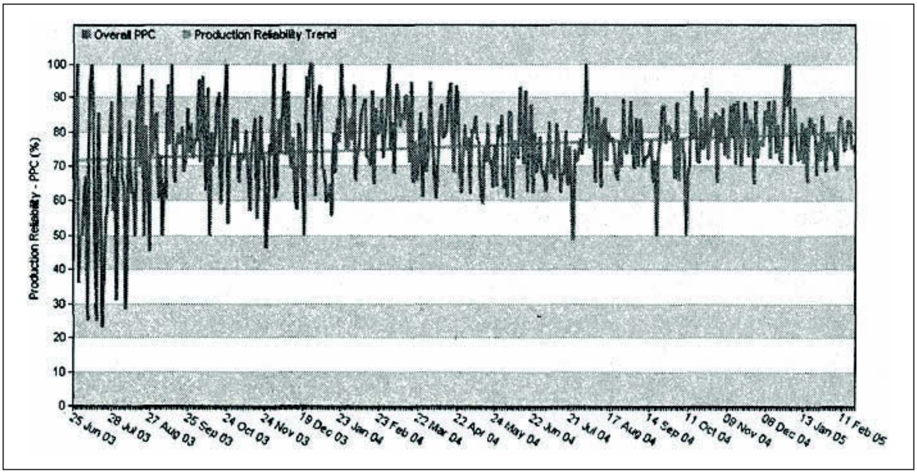

Figure 12. CTRL Production reliability graph to 22 Dec 2004 (Source: Koerckel & Ballard 2005)

CAMIRO Drill and Blast Cycle Simulations

Stewart et al. used benchmarking in combination with simulation results to estimate a theoretical limit for underground development rates of 19m/day. This theoretical limit assumes that it is theoretically possible to achieve the following technical developments and advances while also assuming that the simulated 78% increase can be directly translated to the 6.8 m/day benchmark average (Stewart et al. 2006):

- Shielding to eliminate ground support time
- Successful long round drilling in all ground conditions.
- Halve set-up times
- 3 boom jumbo can be configured to operate effectively at cross-sectional area of 35m^2 to 40m^2.
- Container truck
- Reduce explosive loading time by 30 minutes.

Simulation results for an idealised scenario including; halved set-up times, elimination of ground support time, reduced drilling preparation time, using a 3-boom jumbo, independent firing and reduced explosive charging time has the potential to increase development rates by 90% to 10.2m/day (from the simulation base case 5.4m/day). If the 90% improvement directly translates to the average advance rate for the drill and blast case studies from report by Stewart et al. (7.0 m/day), this scenario would increase advance rates to a theoretical limit of 13.3 m/day.

DISCUSSION

Benefits

The case studies presented in this paper demonstrate how systems engineering methods have been used to improve underground tunnelling rates across a variety of projects and using a variety of methods. In summary, systems engineering methods have been attributed with the following improvements or benefits:

- North Parkes achieved a 58% improvement in cycle time using Lean.

- Application of Lean software for the Channel Tunnel Rail Link project increased production reliability from 70% to 80%.
- Six sigma supported application of emergent technologies that resulted in single heading tunnel rates over 8 m/day (60% above the Australian benchmark of 5.3 m/day).
- Simulation has been used to prioritise rapid tunnelling research areas to those with the most potential to improve tunnelling rates.
- At Kidston mine, tunnelling m/manshift increased by 25% from 0.31 to 0.39 m/manshift.
- Use of simulation software to predict advance rates enables better tunnelling design and planning.

Implementation

Implementation strategies are keys to obtaining benefits from system engineering methods. Based on the case studies presented in this paper, both Lean and Six Sigma appear most advanced in terms of implementation strategies, while benchmarking and simulation are less developed in this regard. Both benchmarking and simulation appear to be primarily undertaken by specialist outside consultants for the purpose of decision making and mine planning. Hall (2000) reports how simulations have been used by mine planning engineers to analyse truck and loader fleet requirements for different mining scenarios, while CAMIRO (2002) used simulation to prioritise research areas. Hall and Harper (2005) recognised the importance of bringing together a site benchmarking team including a "site champion responsible for coordinating different departments" who was considered key to successful implementation (Hall and Harper, 2005). The "site champion" role is key and yet, implementation strategies are not defined for this role. Implementation of benchmarking outcomes depends upon the leadership, authority and ability of the "site champion." This contrasts with Lean which has detailed strategies for operational implementation of improvement recommendations.

Six sigma process mapping steps have been shown to be an effective method for identifying processes where lack of quality control results in delays to the tunnelling cycle. That being said, the complexity of the rapid tunnelling cycle processes and process interaction is such that it relies considerably on experience and understanding to identify critical to quality factors. Hughes (2001) experienced difficulty applying Six Sigma with the level of rigour usually associated with the method. The issue of system complexity could be overcome by combining Six Sigma with a higher level analytical method such as benchmarking or simulation.

A common feature of all systems engineering methods is their reliance on reliable process information upon which to base analysis and improvement. Hall (2000), Hall and Harper (2005) and Hughes (2001) all discuss problems with data reliability and availability. Automated data acquisition/capture systems require much data checking and validation. The possibility exists to incorporate automated data validation and checking algorithms/programs which would enable more timely response to process issues, in much the same way that minerals processing plants use real-time data for process control 24 hours a day 7 days a week.

Sustainability

Sustaining the benefits of system engineering into the future offers long-term benefits as opposed to one-off improvements. Communicating benefits, performance and results of analysis both to management and operators are factors mentioned by Spears (Spears 2001), Dunstan (2006), Hughes (2001), Hall and Harper (2005) and Hall (2000) as being integral to ongoing or sustained implementation.

Implementation of Lean manufacturing boosted tunnelling rates by providing highly visible targets, performance monitoring, as well as, actively involving tunnelling teams in identifying and solving the causes of delay. Lean's use of boards to display performance metrics in tables uses a style of communication familiar to underground employees and was shown to work well. In addition, employees involved in different processes are invited to participate in the process, and the system engineering method becomes part of the system. By contrast, six sigma's performance graphs are more abstract, and therefore more difficult to communicate.

While Lean has demonstrated benefits in terms of ongoing implementation, ideally it should not be seen as a stand-alone systems engineering solution for improving rapid tunnelling. It is conceivable, or even likely that over time a different set of performance metrics should be used. For example, as tunnelling becomes deeper truck availability may become a new limit on tunnelling rates.

It is clear that all system engineering methods discussed use a project or study team, often using consultants from off-site. A limitation of using one-off project or studies is that systems engineering is implemented in a static way often to a situation that may no longer exist. As technology to capture data in real-time advances the possibility exists to create real-time dynamic system engineering methods that can respond quickly and potentially make system engineering part of the system. It is realistic to suggest that developing automated data validation algorithms would capitalise on system engineering benefits by making sustained implementation easier. In minerals processing plants this has been the case of decades. While there are practical challenges to developing a dynamic system engineering solution for rapid tunnelling, the benefits in terms of improved advance rates are well worth the expenditure.

CONCLUSIONS

Application of systems engineering methods in tunnelling and mine tunnelling has been shown to improve tunnelling rates. And, the repetitive cyclic nature of underground development was well suited to systems engineering methods.

Combining higher level analytical system engineering method such as, simulation and benchmarking, with a method with well defined implementation strategies such as, Lean or Six Sigma, offers the potential to deal with the complexity of tunnelling process interactions while also offering practical and proven methods for implementation.

More reliable and faster data capture and reporting was identified as key to sustained implementation of system engineering methods. Faster and more reliable data also offers the potential to continually improve tunnelling rates by incorporating systems engineering methods into the system itself.

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LYON–TURIN HIGH SPEED RAILWAY LINK—ITALIAN PART BASE TUNNEL MIXED SHIELD TBM PROPOSAL

Y. Boissonnas ▪ Amberg Engineering Ltd.

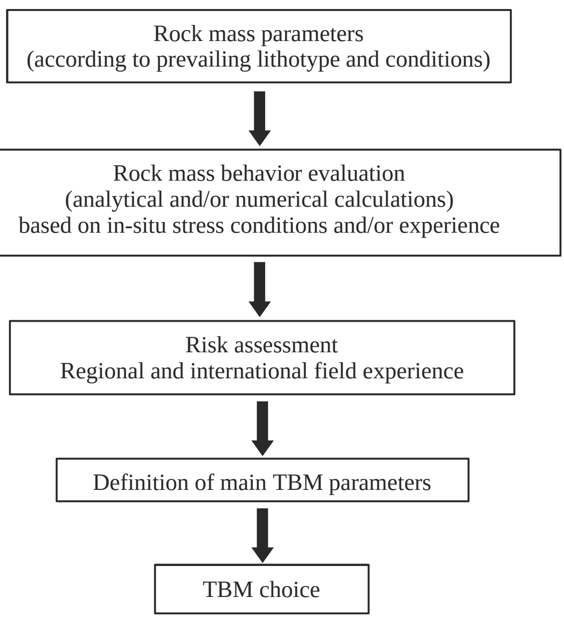
A. Mignini ▪ Amberg Infraestructuras SA

ABSTRACT

This paper provides an overview on the studies carried out during the preparation of Lyon–Turin high speed railway link detailed design on the proposed mixed shield TBM excavation for approx. 9 out of the 12 km section of the Italian part base tunnel. Different geological formations and geomechanical conditions are expected to be encountered, including faults, high overburden, hard and highly fractured rocks as well as loose permeable soils with high water pressures. An overview on calculations of critical sections and TBM specifications is provided.

INTRODUCTION

The approach followed in the choice of the TBM can be summarized in the following flowchart:



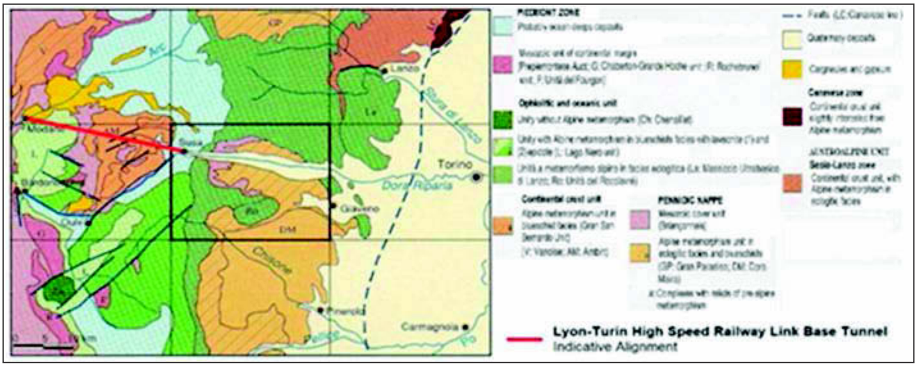


Figure 1. Piemonte geological map

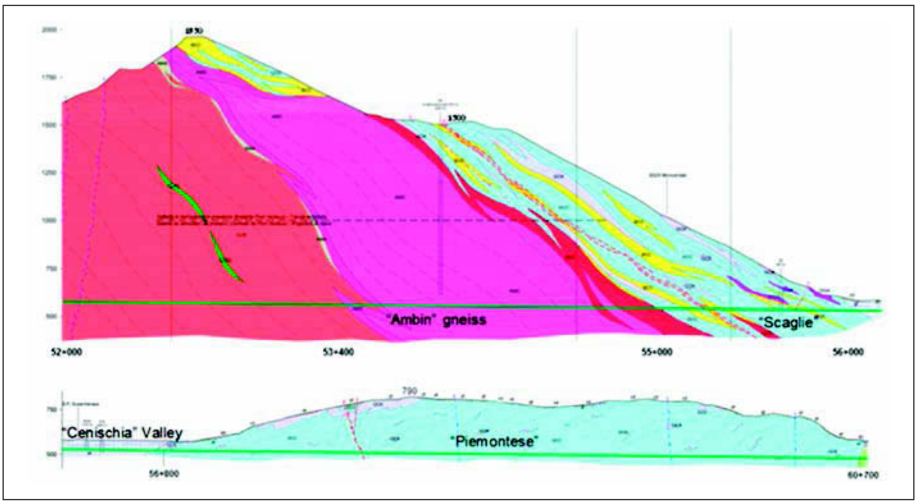


Figure 2. Lyon–Turin Base Tunnel Italian part—Geological-geomechanical profile

GEOLOGY

The regional geological map of Piemonte is reported here, with the indicative alignment of Lyon–Turin base tunnel (Figure 1).

The proposed mixed TBM will bore through the following different zones, starting East (Susa) and going West (direction France–Italy border) (Figure 2).

Following are the geomechanical parameters of different prevailing rock mass units expected along the Italian side base tunnel (Table 1).

Very different rock mass conditions are expected along the alignment, with several hazards as squeezing (Zhao K. 2012) in faulted zones or spalling and rock burst in competent and high stressed rock mass under high overburden (Janutolo Barlet M. 2012). With regard to in-situ stress evaluation, reference is made to the investigations carried out in the past years as already reported in the first preliminary design geomechanical report (APR). It shall be noted that the investigations so far carried out are not many, particularly in the Ambin region, and therefore only an overall trend is available, pending more precise and detailed information expected to come from the excavation of the Maddalena exploratory adit.

Table 1. Lyon–Turin base tunnel Italian side—rock mass units and geomechanical parameters

Unit	Faciès type: description litho.		UCS (MPa)	GSI	Cover (m)	RQD	RMR	γ (kN/m^3)	E_i (GPa)*	E_{rm} (GPa)*	C (MPa)*	F (°)*	Tensile (MPa)	mi*
B	Quartzites (tQ, QSE) Quartzitic conglomerates (r-tCG)	min	46.7	59	400	—	54	26.1	14	6.9	1.7	43	3.3	13
		max	209.1	75	1600	—	74	28.1	63	51	11.5	51	22.1	20
		moy	132.4	67	1000	—	64	27.1	38.5	29.0	5.3	46	13.6	17
		rèf.	132	67		—	64	27.1	38.5	29.0			13.6	17
C	Marbres, Calcaires, Dolomites & Argillites (Cb, CDng, Cng, cs-e, DGA, Jm, jmC, j1, j4-7, l1-4, tC, tCd, tD, tsD, t5S, t6, t7)	min	21.2	50	400	—	52	25.9	7.4	2.3	0.8	27.5	1.8	6
		max	224.3	71	1610	—	73	29.2	78.5	59	10.4	43	10.7	10
		moy	76.2	61	1005	—	63	27.6	42.95	21.0	4	37.0	6.9	8
		rèf.	76	61		—	63	27.6	38.5	21.0			6.9	8
D	Gneiss & Schists (AMA, AMC, AMD, AME, AMF, GCK, GS, S, Cl, SV, TCS)	min	3.8	48	20	—	45	22.3	3.1	0.85	0.07	37	2.5	6
		max	386.1	75	2230	—	77	28.7	96.5	78.7	18.7	51	17.4	18
		moy	73.2	62	1115	—	61	25.5	28.0	15.3	3.8	39	9.4	12
		rèf.	99	62		—	61	25.5	28	15.3			9.4	12
J	Faults & Cargneules (Ksb, K BCC)	min	5	15	0	—	10	23	—	—	—	—	—	6
		max	20	35	2230	—	30	26	—	—	—	—	—	8
		moy	13	25	1115	—	20	24.5	—	—	—	—	—	7
		rèf.	12.5	25		—	20	25.0	—	—	—	—	—	7
Unit	**Faciès type: description litho.**		**UCS (MPa)**	**GSI**	**Cover (m)**	**RQD**	**RMR**	**g (kN/m^3)**	**E_i (GPa) ***	**E_{rm} (GPa) ***	**C' (MPa) ***	**F' (°) ***	**n**	**mi ***
K	depots non-consolidés (ac, af, at, EJ, Ez, gi)		—	—	0–60	—	—	20–22	—	0.05–0.2	0–0.01	33–37	0.3	—

Table 2. Lyon–Turin base tunnel Italian side—in-situ stress k_0

Pk	Zone	k_0	H [m]
52+000 ÷ 53+400	"Clarea" Micaschists	1.8	1500–1000
53+400 ÷ 54+700	"Ambin" Gneiss	1.6	1000–650
55+000 ÷ 56+000	"Scaglie"	0.6	650–50
56+000 ÷ 56+800	"Cenischia" valley	1.0	50–55
56+800 ÷ 60+700	"Piemontese"	0.6	290–500

Table 3. Lyon–Turin base tunnel Italian side—sections checked

Zone	Ambien	Ambien-Scaglie	Scaglie	Piemonte
Section	**C2**	**C3**	**C4**	**C6**
Pk [km+m] Rp [mm] Prevailing Unit Geology	51+550 5100 UGD(7) Sequence of micaschists and fine gneiss alternating with amphibolites	54+950 5000 UGD(5) Tectonic contact transition zone Ambin–Scaglie alternating with AMC and QSE	55+300 5000 UGD(2) Philladic calceshists, gneiss, tectonised rubbles or cargneules	57+950 5000 UGD(2) Decametric – eptometric alternation of GCC, GCCk e/o GCK

However, it can be noted that vertical in-situ stress is close to its lithostatic value, therefore it is assumed as γ*H, whereas horizontal stress varies according to geology and location. Following k_0 (σ_h / σ_v) values are summarized for easy reference (Table 2).

ROCK MASS BEHAVIOR (PK 52+000 ÷ 60+700)

Ground Squeezing

In this chapter an estimate of rock mass plastic radius R_p and longitudinal displacement profile (LDP) in 5 representative sections namely C1, C2, C3, C4 and C6, is provided, based on analytical formulation of convergence-confinement ground reaction curves (Hoek 1999). Following are reported location and geology of sections checked (Table 3).

The parameters adopted in the analytical formulation, are equivalent "fitting" cohesion and friction angle values (Mohr-Coulomb) based on GSI and Hoek-Brown compressive strength UCS.

In each section, UCS and mi are assumed as characteristic of rock mass unit, thus are kept constant, whereas GSI index varies from very competent rock (70) to expected decametric fractured/faulted zones (35), taking into account a "wall effect."

Where two different lithotypes prevail, the worse is taken as reference (conservative approach). Where no precise data are available, a UCS medium to minimum value is considered. Deformation module for intact rock (Ei) is considered as average between maximum and minimum.

With regard to in-situ stress, this is calculated as lithostatic value $\gamma{*}H$ multiplied by k_0, where $k_0 > 1$ (section C1 and C2), whereas is left equal to lithostatic $\gamma{*}H$ where $k_0 < 1$ (sections C3, C4 and C6).

Calculations and Results

Beginning of plastic zone is calculated based on Mohr-Coulomb effective stress formulation:

$$\sigma'_1 = \sigma_{cm} + k\sigma'_3 \quad \textbf{(1)}$$

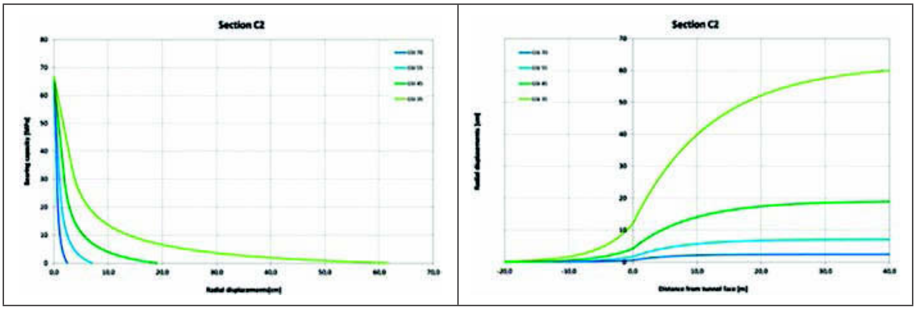


Figure 3. Reaction curve and displacement longitudinal profile (LDP) at Section C2

An internal supporting pressure pi is assumed acting onto the circular excavation with radius r_0.

The plastic zone around the excavation will start to form when the internal pressure is below the critical supporting pressure defined as:

$$p_{cr} = \frac{2p_0 - \sigma_{cm}}{1 + k} \tag{2}$$

Where $p_i > p_{cr}$, no plastic zone takes place and the behavior of rock mass around the excavation is considered elastic with a radial displacement given by the following equation:

$$u_{ie} = \frac{r_0(1+n)}{E_m}(p_0 - pi) \tag{3}$$

Where E_m is the Young's modulus and v the Poisson ratio. Where $p_i < p_{cr}$, a plastic zone takes place around the excavation with a radius rp given by the following equation:

$$r_p = r_0 \frac{[2(p_0(k-1)+\sigma_{cm})]^{1/(k-1)}}{(1+k)((k-1)p_i + \sigma_{cm})} \tag{4}$$

With a radial displacement u_{ip} toward the center of the excavation given by the following equation:

$$u_{ip} = \frac{r_0(1+\nu)}{E}\left[2(1-n)(p_0 - p_c)\left(r_p/r_0\right)^2 - (1-2n)(p_0 - p_{cr})\right] \tag{5}$$

Based on the above approach and on the GSI index range considered (35÷70), ground reaction curves and longitudinal displacement profile (LDP) have been calculated for each section (Figure 3). According to LTF expert panel recommendations, TBM excavation shall stop at chainage 52+000 i.e., just before section C2, which is therefore considered "worst case" scenario for ground squeezing (Figure 7).

SOIL BEHAVIOR—CENISCHIA VALLEY (PK 56+000 ÷ 56+800)

Hydrogeology

Cenischia Valley consists of homogenous horizontal sand and gravel deposits with intercalation of small bands of silts and sandy silts and decimetric bolders. The aquifer flows perpendicular to the tunnel and water table is expected to vary from approx. 5m to 25m below surface, due to the proximity of Pont-Ventoux hydropower plant draw-down (Figure 4). Permeability is estimated in the order of 4.5 e^{-6} m/s.

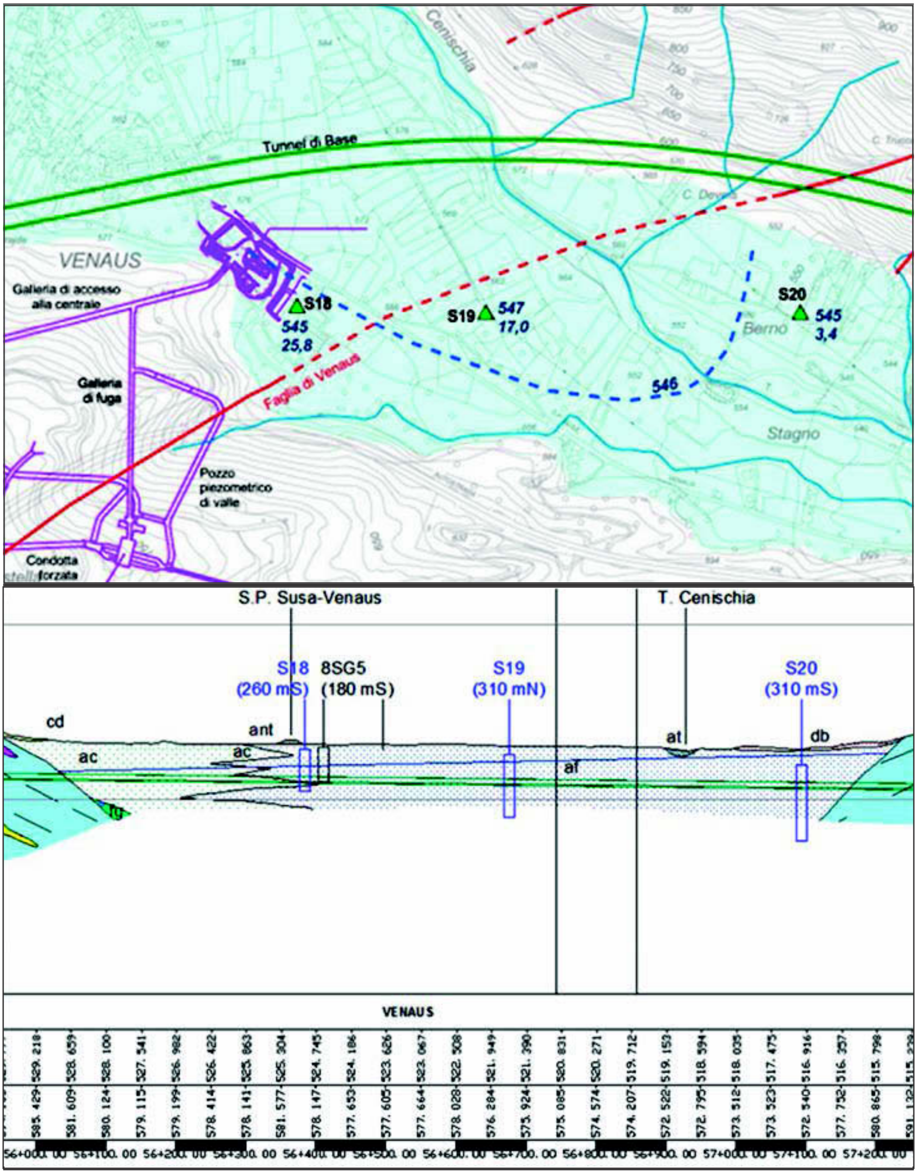


Figure 4. Cenischia hydro-geology, plan view, and longitudinal profile of aquifer along tunnel alignment

Face Pressure Calculation

According to limit equilibrium approach based on Horn formulation for granular soil (Anagnostou G. Kòvari K. 1996), a minimum working target pressure P_{target} is calculated as the ULS effective stresses transmitted in the short term at the tunnel crown by the water and the soil skeleton plus a variation, according to the following formula and sketch diagram (Figure 5):

$$P_{target} = (C - Z_w)\gamma_w + (Z_{s_1} - C)\gamma_{sl} + F_0\gamma' D - F_1 c' + variation \quad \textbf{(6)}$$

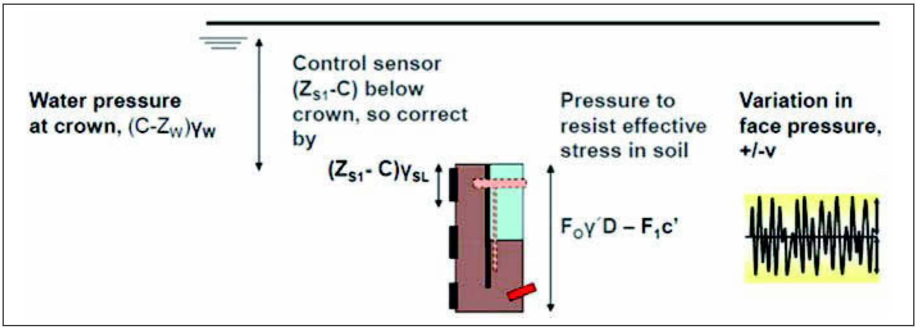


Figure 5. Limit equilibrium formulation (Anagnostou G. Kòvari K. 1996)

According to Anagnostou & Kovari charts, dry soil and submerged soil density ratio is taken as $\gamma_d/\gamma' = 1.6$, whereas F_0 and F_1 are factors depending on ϕ' and *C/D* ratio. No cohesion is considered in this type of soil for calculation purposes, therefore the effect of *C'* is assumed as 0. Water table Z_w is assumed as varying from 5 to 25 m below surface. A slurry density at tunnel crown γ_{SL} = 1,15 kN/m^3 is assumed, considering sensor P_1 is near the bentonite feeding pipe and more dense slurry and excavated material tends to accumulate at the bottom of the chamber by gravity; γ_{SL} = 1,3 kN/m^3 is assumed to derive target pressure at tunnel axis. Friction angle ϕ' of 31,5° with a safety factor of 1,2 is assumed. No surcharge on the surface is assumed at the time of tunneling.

A variation in face pressure of ±0,5 bar is assumed, considering pressure will be adjusted by slurry without air bubble. Following are the calculated target pressure at TBM crown (Table 4).

SQUEEZING, SWELLING, AND WATER—“SCAGLIE” ZONE (PK 55+000 ÷ 56+000)

This is a very critical zone of tectonic discontinuity characterized by discontinuous bands of cargneules and highly sheared zones expected to intersect the tunnel alignment around Pk 55+300 and 55+700, with possible clayey-sandy faulted rocks. A pluridecametric fault has been detected around Pk 55+300. Mixed face, anisotropic rock mass behavior accompanied and/or alternating with frequent excavation face instability, squeezing and swelling ground conditions are expected together with the risk of high water heads up to 300 m and high pressure water ingress through sheared and faulted zones, due to Pont Ventoux Hydropower Plant drawdown towards the Cenischia Valley (Venturini G. Damiano A. et al. 2001).

TBM CHOICE

Mixed Shield Drive Configuration

According to above foreseen hydro-geological and geo-mechanical conditions and risks, following is the proposed mixed shield TBM drive configuration along the tunnel alignment (Figure 6):

1. “Piemontese”: hard rock open mode excavation (PK 60+700–57+200)
2. “Cenischia”: 1st modification to slurry shield and closed mode excavation (PK 57+200–56+000)
3. “Scaglie”: 2nd modification to hard rock shield and open mode excavation (PK 56+000–55+000)

Table 4. Target pressure at TBM crown under Cenischia Valley

PK	Zone	D m	q kPa	C m	Zo m	Zw m	Zst C	γ_w kN/m^3	γ_1 kN/m^3	γ'_1 kN/m^3	ϕ_1 Deg	χэ1 kPa	$\gamma_{SL\ P1}$ kN/m^3
56+400~ 56+800	Cenischia Valley	10	20	48	53	5	0.5	9.8	19	9.2	31	1.5	11.5
56+000~ 56+400	Pont Ventoux HPE	10	20	45	50	25	0.5	9.8	19	9.2	31	1.5	11.5

PK	$\gamma_{SL\ Pasis}$ kN/m^3	γ kN/m^3	γ' kN/m^3	$\phi_{1.2}$ Deg	Fo	P1 bar	Var. bar	Passe bar	Pax. min	Pax. max	C/D
56+400~ 56+800	13	19.3	12.1	26.3	0.25	4.6	±0.5	5.2	4.7	9.1	4.8
56+000~ 56+400	13	19.3	12.1	26.3	0.25	2.3	±0.5	2.9	2.5	8.6	4.5

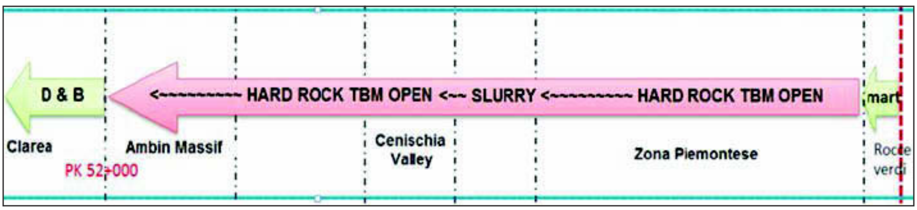


Figure 6. Lyon–Turin Base Tunnel Italian side—TBM drive configuration

4. "Ambin": hard rock shield and open mode excavation (PK 55+000–52+000)

Thrust Calculation

Nominal TBM thrust is assessed based on pressure on the shield induced by rock-mass deformation. According to above LDP for section C2, a 4 m contact is envisaged with stress induced building up progressively during TBM advance (Ramoni M. Anagnostou G. 2011, Zhao K. Janutolo M. et al. 2012) up to approx. 3 MPa according to reaction curve at 10m distance from the excavation face, taking into account 5 cm conicity (Figure 7).

Requested thrust force F_f to overcome friction can be expressed by the following equation:

$$F_f = \mu p 2\pi R L \tag{7}$$

Where μ = friction coefficient it is suggested to use μ = 0.15–0.3 during TBM advance and μ = 0.25–0.45 when excavation starts after stoppage (Ramoni M. Anagnostou G. 2011). A value of 0.3 is adopted (Zhao K. Janutolo M. et al. 2012). Such value does not take into account any additional reduction (e.g., bentonite injection).

A shield radius R = 4.90m is considered, based on 5 cm shield conicity on radius. A contact length L = 4 m is assumed.

According to the above a F_f = 105 MN is required plus the cutterhead thrust F_N, that can be calculated as follows according to assumed overbore configuration (approx. 50nos.17"cutters + 20nos.19"cutters):

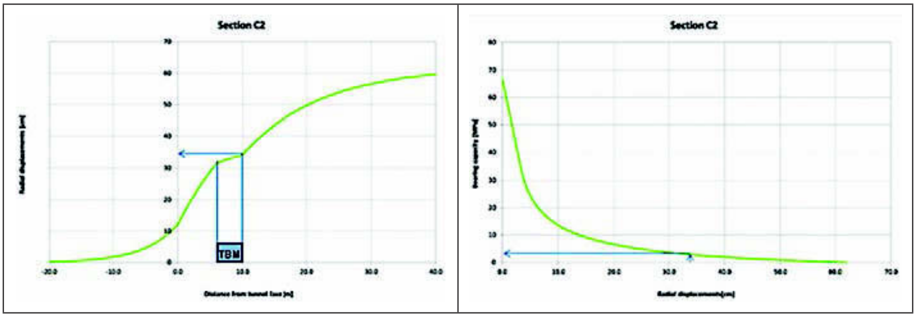


Figure 7. Radial displacement around TBM shield at section C2

$$F_N = 50 \times 270 \text{ kN} + 20 \times 300 \text{ kN} = 19.500 \text{ kN} \quad \textbf{(8)}$$

to be reduced down to approx. 15 MN, considering a non-contemporaneity factor of 0.7–0.8 thus giving a nominal total thrust of approx. 120 MN. It has to be noted that ground reaction curves-LDP method tends to overestimate thrust value with respect to axial-symmetric or 3D models (Zhao K. 2012).

With regard to tunnel bore through granular deposits of Cenischia Valley, TBM shield is expected to be practically fully in contact with ground. However, contact pressures around the shield during TBM advance, as induced by effective stress in the ground, are expected to be far lower than in above squeezing rock conditions. Assuming a contact pressure p=138.4 kPa and a contact length of 9 m over an overall 10 m shield, a friction force F_f = 11.6 MN is expected.

With regard to exceptional thrust, a value of 150MN is indicated by comparison with other projects (Pelaez M. Arroyo J. C. et al. 2009, Mendaña F. 2004, Gonzalez J. F., Gandìa J et al. 2004, Werner B. Dudouit F. 2009, Hoek E. 2001). Torque is expected in the range of 16÷21 MNm with exceptional torque 25÷30 MNm.

Mixed Shield TBM Main Features

Proposed mixed shield TBM shall be single shield with articulated retractable cutter-head and longitudinal thrust onto precast segment ring (lateral grippers may be considered only for roll correction expected to be performed in hard and squeezing rock conditions.

Nominal diameter is expected to be 9.950–9.970 mm with cutterhead 50–70 mm bigger than foreshield. A max. 50–70 mm overbore and a shield conicity of up to 50 mm on radius are recommended. Power is expected between 4 and 5 MW (12÷14 electrical VOF motors 315–350 kW).

CONCLUSION

Different geomechanical conditions expected to be encountered are analyzed in this paper (high overburden, faults, hard and highly fractured rocks and loose permeable soils with high water pressures). According to the above, it is proposed to choose a mixed shield TBM able to operate in open mode under high overburden and through highly unstable and squeezing ground conditions with possible high water ingress, as well as in closed mode with bentonite injection pressure balance through non cohesive soils under high hydrostatic pressure.

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LONG-DISTANCE, INTER-BORO MUCK CONVEYANCE CHALLENGES

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ABSTRACT

Productive mining of eight TBM running tunnels, associated wye caverns and twin 1200-ft long by 60-ft wide main caverns for Metropolitan Transportation Authority Capital Construction's East Side Access Project Contracts CM-009 and CM-019 demanded a reliable, durable and flexible conveyance system. Specifically, a flexible system accommodating removal, handling and conveyance of over 400,000 cubic yards of muck derived from drill-and-blast, roadheader, and TBM excavation. Paper discusses the robust and reliable system built to withstand the rigors of a five-day per week, 24-hour per day mining operation extending from multiple headings beneath streets of Midtown Manhattan to the mucking shaft located over four miles away in Long Island City, Queens. Lessons learned from using one main 36-inch width conveyor belt with up to five unique loading points and several different mucking system configurations using a jaw-type rock crushing plant are extensively discussed.

OVERVIEW AND INITIAL DESIGN OF CONVEYANCE SYSTEM

As part of Metropolitan Transportation Authority Capital Construction (MTACC's) East Side Access Project Contracts CM009 and CM019, the Dragados-Judlau Joint Venture (DJ/JV) mined a series of eight running tunnels, ten wye and two main station caverns, associated cross passages, vertical shafts, and inclined escalator wellways to provide commuter rail service connection from Long Island to Grand Central Terminal. This paper describes the various challenges to productive mining that were met and overcome in conveying muck generated from multiple headings under mid-town Manhattan through existing tunnels under the East River to the mucking shaft located four-miles away in Long Island City, Queens. This extensive multi-level rock excavation effort was performed using both conventional (drill-&-blast) and mechanized (roadheader and Tunnel Boring Machine) techniques. Successful and productive excavation advance mandated a robust and reliable muck conveyance system including a total system belt length exceeding 25,000-ft. An innovative and flexible system was required to permit future modifications for handling muck produced from newly mined headings as well as various types of muck generated. In total more than 800,000 CY of muck was removed by the system, which additionally was supplemented by diesel-powered muck cars.

From the project outset, a durable and reliable automated muck conveyance system was required to accommodate the future vast and long-distance generated excavation spoils. This system included the following main components: Surface (overland and stacker belt), vertical (at Bellmouth shaft), underground (fixed and transfer belts). Specifically, a 36-inch-wide fixed belt manufactured and engineered by Robbins was installed in the existing (out-bound) lower-level 63rd Street tunnel under the East River (approximately 8,500 ft) which initially connected from Manhattan at an existing

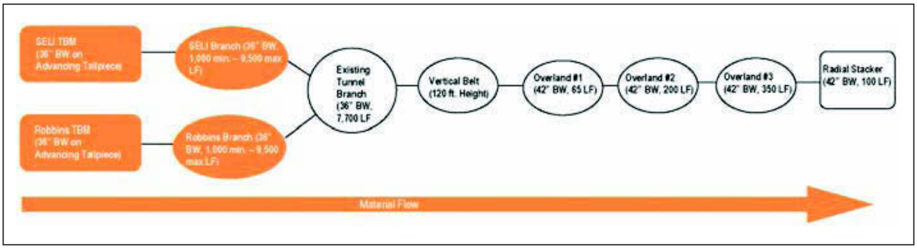

Figure 1. Initial muck conveyance system schematic

bulkhead under Second Avenue crossing under the East River and Roosevelt Island to Long Island City, Queens, discharging onto a vertical belt at the Bellmouth Shaft. This mucking shaft was outfitted with a 120-ft high Robbins vertical belt which discharged onto three sections of overland conveyor belt and a radial stacker. The second overland conveyor section included a fully-enclosed box truss spanning 37-m (120-ft) over Northern Boulevard and provided approximately 6-m (20-ft) vertical clearance over the active five-lane roadway in Long Island City, Queens. To accommodate continuous 5-day × 24-hour/ day per week TBM mining operations as well as restricted haulage truck schedules a radial stacker belt conveyed muck from the third overland belt to stockpiles in Sunnyside Rail Yard. The stacker belt featured a 60° horizontal rotation enabling muck distribution into multiple stockpiles with total 8,400 cubic meters (11,000 cubic yards) capacity.

The initial Robbins system was configured as shown in Figure 1. Both belt cassettes were designed to accommodate one 1,500 ft. roll of 36-inch-wide Depreux conveyor belt supplied by Cobra America and operate continuously while two 22-ft.-diameter hard rock TBMs mined over 22,000 LF each. The initial muck conveyance system was maintained by four full time mechanics, with one maintaining each TBM conveyor branch, one for the existing tunnel branch, and one for all overland belts and the vertical belt. After ten months of continuous TBM mining, Sandvik roadheaders and drill-and-blast methods were used to concurrently open additional headings. This paper will describe both the components determined to create the most favorable composite muck-removal system for multiple headings and the system's evolution as headings either opened or were completed as well as the lessons learned by implementing various components innovatively.

Unique System Conditions and Constraints

The project team quickly realized running two roadheader headings and at least four active drill-and-blast headings ranging from 3.5 to 5 miles away from the site's sole mucking shaft continuously for over three years would require more effective means of mucking than by trains comprised of 20 cy muck cars. California switches were deemed an infeasible option to augment the mucking system for two major reasons. Two of the four active rails were used to supply TBM headings while excavation headings were routinely operating directly adjacent to TBM rail. Waterproofing, rebar, and structural concrete needed to be installed both immediately after completing TBM mining and immediately after excavating major caverns or auxiliary structures. At any given time during the project two or three of the four rail lines were in constant use by either of the other operations. Given the nature of the project, 65'H × 60'W × 1150'L caverns needed a fully waterproofed reinforced concrete arch installed while excavation continued to the south, cutting off over 125,000 bcy of remaining excavation from rail access. Those caveats along with numerous others forced the project team to develop solutions to the muck removal issues.

Figure 2. Crusher plant installation in a 3.5-ft deep pit

CONVEYANCE SYSTEM MAJOR DESIGN CHANGE I

The decision to implement a 42" × 36" jaw-type rock crusher plant was dictated by the ineffectiveness of running muck trains to the shaft for aforementioned reasons as well as crane availability. Running three shifts per day at five production days per week each drill-and-blast heading generated an average of over 900 bcy per week. The typical swell factor for shot rock was calculated by the project team to be 2.0 (900 bcy/wk * 2.0 swell factor = 1,800 lcy/wk/D&B crew). Roadheaders generated an average of 1,200 lcy each, per week. Reconfiguration of the muck conveyance system was required to remove the average muck generation of at least four D&B headings and two roadheader headings running concurrently with TBMs each generating an average of 5,500 lcy/wk, or a total system removal capacity of about 15,000 lcy/wk. The initial Robbins 36" width tunnel belt system was capable of conveying the specified quantity of muck once it was on the conveyor belt, and the jaw crusher plant was capable of handling over 150 lcy of shot rock per hour and crushing it below the maximum allowable particle size of 6-inch-minus, dictated by the bucket size of the vertical belt.

No major structures had been fully excavated by the time the Lippmann crusher plant was introduced to the system, so the decision was made to install it within the limits of a future wye cavern previously excavated by two 22-ft. diameter TBM bores. The plant's dimensions forced the project team's hand to make considerable installation accommodations. The layout of the plant installation allowed adjacent rail traffic or equipment movement while it was being fed by five, six, and eight-cubic-yard load-haul-dumps (LHDs). Another requirement was the capacity to discharge onto and integrate into the muck conveyance system, again because there would never be enough rail availability to run a continuous muck train operation. In order to fulfill all parameters the crusher plant was installed in one of the two TBM bores in the future wye cavern allowing for the entire bore to be utilized by LHDs and associated mucking equipment. The feed end of the crusher plant, however, was higher than the maximum dump height of the LHDs. Since the LHDs were used to haul shot rock, it was decided that tailoring the plant's operation to LHD use was preferable over double-handling muck and adding a front end loader. By committing to LHDs, the project team added a 13% grade ramp to the haul road leading to the feed end of the crusher plant. Additional head room was required, however, to accommodate the LHDs' buckets and it was decided to excavate a 3.5'-deep pit in which to install the crusher (Figure 2).

As the crusher plant was integrated into the system with its own 2,500' long fixed belt, take-up unit, and 25-foot-long transfer belt (Figure 3); two roadheaders continued mining adjacent to the TBM branch belts. LHDs with side-dump buckets discharged roadheader fines directly into a hopper above an impact bed on the active TBM belts. The schematic for the described system can be seen in Figure 4. Figure 5 shows how the crusher branch's conveyor belt was installed on a 12-degree incline to allow for rail traffic and equipment to pass underneath.

CONVEYANCE SYSTEM MAJOR DESIGN CHANGE II

After eleven months under the initial robust system configuration, the project's major top heading excavations had been completed and benching operations were set to commence. Due to the nature of the excavation sequences for multi-level caverns, any bench blasting would result in shot rock filling lower level TBM drives or developed lower levels of aforementioned caverns. The crusher plant in its initial position at the benching elevation would cease to be effective. Hence, it was decided to transport the fully assembled crusher plant to a completed wye cavern excavation located 34-feet below. Transporter frames were designed and welded onto the plant's structural truss

Figure 3. Take-up unit and transfer belt on crusher branch prior to 36-inch belt installation

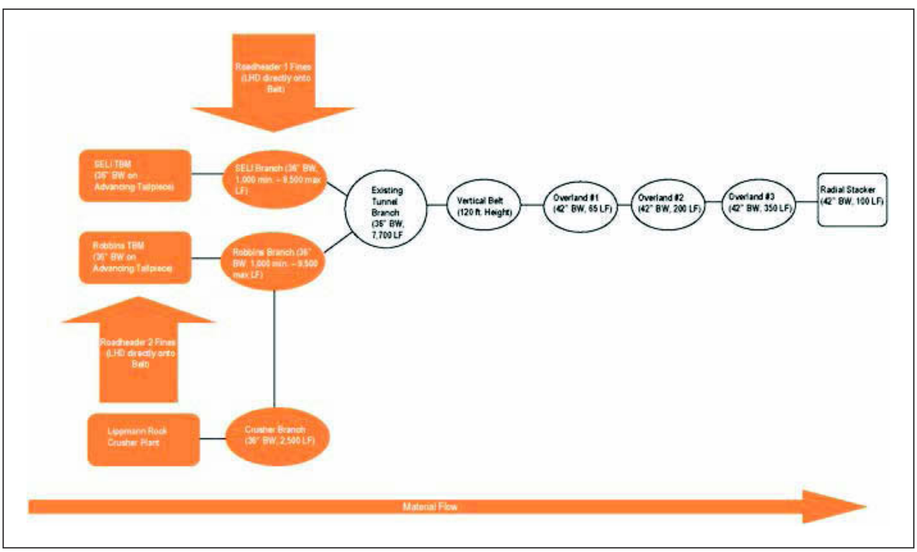

Figure 4. Robust muck conveyance system schematic

Figure 5. Crusher branch belt existing crusher pit on a twelve-degree incline

Figure 6. Fully assembled crusher plant on transporter frames and rail dollies

and high-capacity rail dollies were coupled to the frames. The plant was pulled by locomotive to its final location as seen in Figure 6.

Two major problems identified by the project team were solved and addressed during the system's second major configuration change. First, the challenges associated with introducing shot rock onto a TBM's main discharge belt on top of additional Roadheader fines were able to be ignored because TBM and Roadheader excavation was completed by the time the second configuration change occurred. Although the crusher branch belt, its transfer belt, and the plant itself had an 80% availability rate, its actual availability dropped to 20% because of its dependence on the continuous downstream operation of TBM and every other belt conveyor. The new configuration (Figure 7) reduced belt down-time by eliminating two transfer points and the separate TBM belt. Secondly, when the initial crusher was installed in its 3.5' deep pit any spillage of fines or over-saturated material from the vibratory feeder or crusher discharge belt sat in the pit and could only be hand-mucked. The tight geometry of the initial installation point prevented machinery access to the plant for mucking purposes, and water hoses would only serve to push the wet material around inside the pit.

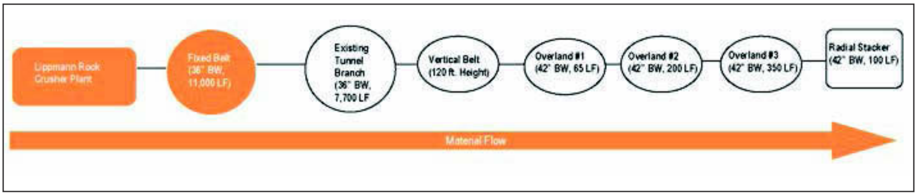

Figure 7. Final muck conveyance system schematic showing belt widths and lengths for each discrete branch

The pit was necessary during the initial installation because of headroom and dump height issues. The final installation utilized a flight conveyor on a 13-degree incline (Figure 8) to allow LHDs to dump into its feed end at ground level and the flight conveyor discharged the shot rock into the crusher plant's vibratory feeder bed at a feed rate of 50 fpm. Though the addition of the flight conveyor introduced more maintenance items to the system, it proved extremely beneficial by eliminating the need for installing a 30-m long by 5-m high ramp in a fully-excavated wye cavern in a critical location on the project. Another problem regarding the crusher plant was the frequency drill steel and other metal fragments would pass through the jaw and puncture the crusher plant's discharge belt. Tearing the belt and installing a new one cost between two to three shifts of downtime, so a pan feeder was introduced under the jaws to deflect any steel passed by the jaws and to lay the material parallel to the belt's feed direction rather than perpendicular to. Since the pan feeder installation meant steel was being accepted to the muck flow, a magnet belt was added at the crusher plant conveyor's head pulley. It allowed for continuous steel removal without the need to stop the belts for manually cleaning the magnet. In an added effort to protect the crusher discharge belt, the project team also decided to through-bolt steel plates to the carrying side of the belt.

During the crusher plant move, improvements were also made to the overall system. The vertical belt underwent weeks of extensive maintenance and bucket repairs. The over 7,300-meters of belt on the main 36"-width fixed length tunnel belt underwent heavy maintenance as well, having all mechanical splices vulcanized. Trouble zones where excessive idler and belt edge wear were identified and rectified by making curve radii consistent. The previous TBM branch belts' drive motors (2 ea. 250-hp units) were converted into booster motors and the previous existing tunnel branch motor (250hp unit) acted as the main drive at the long belt run's head pulley and take up unit, eliminating one transfer point and belt. In its initial configuration muck from the crusher plant needed to be transferred to different belts 12 times before discharging to the muck pile via the radial stacker. In its final configuration crushed rock needed only to be transferred onto six different belts, alleviating the potential for downtime associated with more transfer points and moving parts.

In the system's final configuration, total system availability rose from 20% to over 50%. Weekly mucking capacity of shot rock through the crusher system rose from 4,000 cy/wk to over 6,500 cy/wk. Figure 9 compares the mucking production under the initial and final crusher configuration, respectively. Both actual and average production of crusher plant is shown in Figure 10. The improvements and investment made in the system reduced the need for unscheduled maintenance due to belt tears. They also shortened the project's completion by months because over 200,000 cy of shot rock was passed through the more efficient final configuration.

Figure 8. Final muck conveyance system at flight conveyor feed point

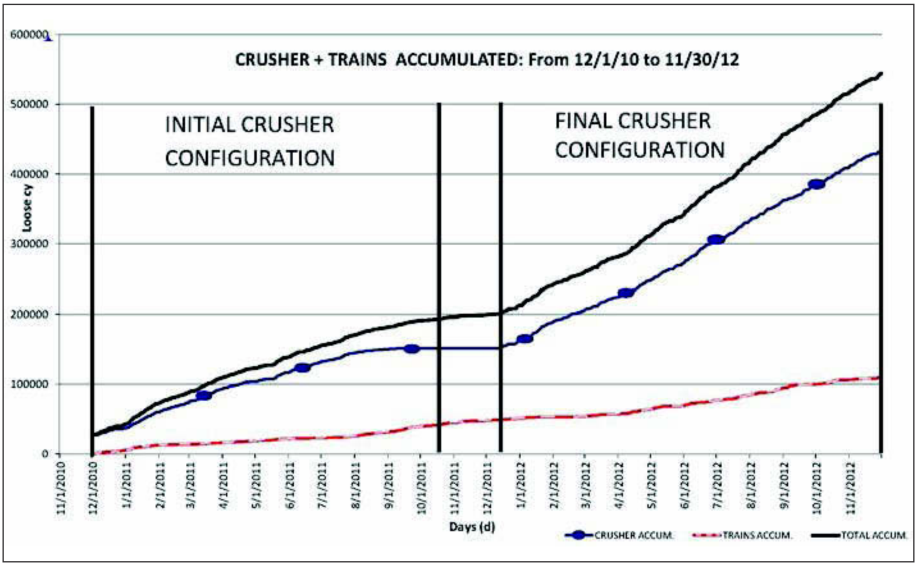

Figure 9. Cumulative mucking system production graph

LESSONS LEARNED AND CONCLUSION

On a project where muck trains are prohibitively time and rail consuming creative thinking became paramount with regards to muck conveyance system design. Key takeaways are as follows and all regard maximizing muck conveyance system availability:

- Minimize muck transfer points
- Minimize instances of routine maintenance items, if possible
- Commit to at least one scheduled maintenance shift per week to address issues which arise during production
- Ensure full auxiliary mucking equipment access around main dump points to alleviate muck buildup and unnecessary maintenance, especially if site geometry does not allow for clean-up conveyors

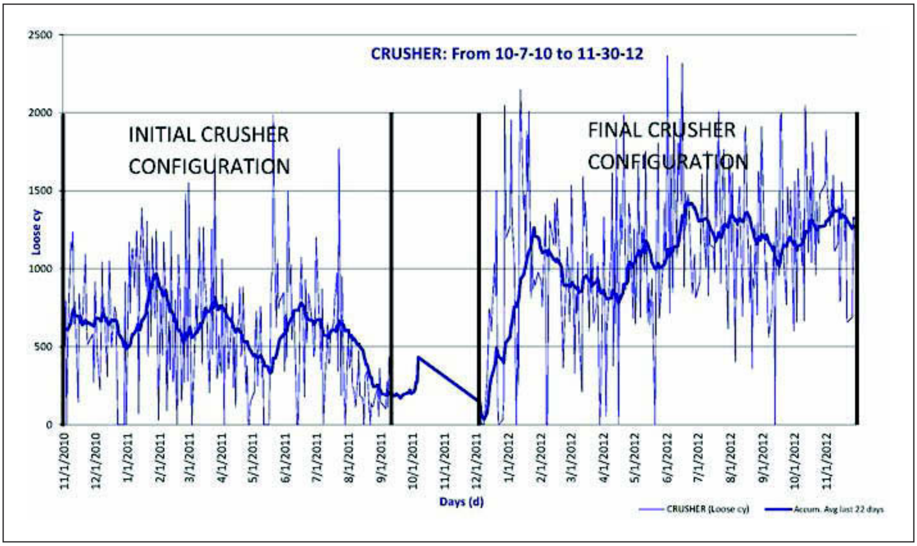

Figure 10. Crusher plant actual and average production

- Perform breakeven analyses regarding capital investments into the system versus schedule and labor savings with an increasingly efficient system to understand which will yield realized returns on investment

Considering the impacts of various muck removal options and the need for continuous operation, an innovative robust system with high availability and reduced labor demands was developed to help realize schedule savings. Especially in the unique New York City skilled labor market, one of the most expensive in the world, reducing schedule and maintenance time equated to high cost savings. The pre-emptive initial installation and the calculated reinstallation of the crusher plant proved that making a significant capital and labor investment at the start of a major (over 100,000 cy) mucking system design change is worth the cost. What was paid for up-front gave a return on investment many times over. The overall project greatly benefitted from using nonconventional techniques to respond to and remove bottlenecks in the muck conveyance system.

INNOVATIVE APPROACH TO MUCK DISPOSAL AND VENTILATION DURING DRILL-AND-BLAST OPERATIONS IN A DENSELY POPULATED URBAN ENVIRONMENT

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ABSTRACT

As part of the ongoing 2nd Avenue Subway extension program which will relieve major crowding for subway riders on Manhattan's east side, MTA Capital Construction (MTACC) has contracted with Skanska/Traylor Joint Venture (STJV) to perform mining and structural concrete work for the 86th St Station. The contract's main objective is to excavate and dispose of approximately 160,000 CY of rock from below one of Manhattan's busiest avenues, 2nd Avenue, with measures to mitigate the impact on the public and traffic during the operation. Working in a densely populated urban neighborhood requires special attention to be paid to the methods used for lifting the excavated rock out of the two access shafts, each located in narrow work zones on Second Avenue, and subsequently transferring the material to trucks for disposal off-site. A fine balance must be maintained in order to provide a high production system for the removal of rock to meet schedule constraints, while at the same time addressing public concerns about the impacts of construction on their local environment. The solution developed for this operation is a mechanized mucking system with an overhead gantry crane for shaft hoisting and a separate dumping crane which loads 30 cubic yard dump trailers for off-site disposal.

This paper will also describe the use of an innovative approach to ventilation during drilling and blasting operations in order to address the air quality concerns of a highly sensitive and densely populated local community. A special ventilation system was developed to minimize the visible dust and reduce smoke and fumes created by drilling and blasting. This system uses both a dedicated supply air system and exhaust air system at each of the two access shafts for the cavern excavation. The exhaust air system incorporates a wet dust collector using scrubber technology in order to treat the dust laden air created after a blast before discharging it out into the environment.

BACKGROUND ON THE 86TH STREET STATION PROJECT

The Upper East Side of Manhattan in New York City is currently served by one north-south subway route, the Lexington Avenue Line. The east side of Manhattan was previously served by two elevated transit lines, the Second Avenue "El" and The Third Avenue "El," but these lines were demolished in the 1940s and 1950s to make way for commercial and residential high-rise development with significantly increased population density. As a result, the Lexington Avenue Line became overcrowded, and as the sole remaining line, it now carries a daily ridership that is approximately equal to the combined ridership of the rail transit systems of Boston, Chicago and San Francisco.

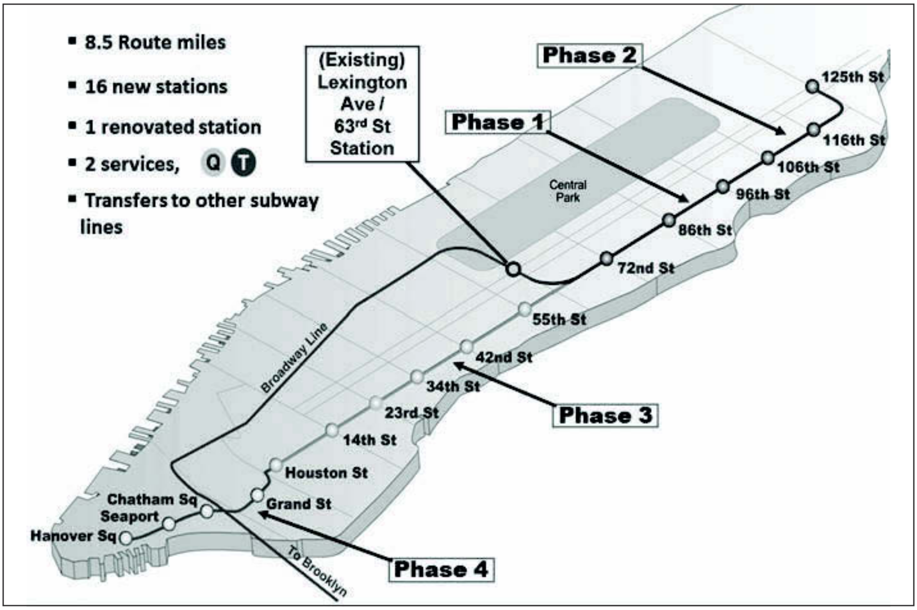

Figure 1. Second Avenue subway project route and phasing

To address the need for additional subway transit on the East Side of Manhattan, the Metropolitan Transportation Authority (MTA) revived a plan shown in Figure 1 to construct a new north-south subway line under Second Avenue, extending 8.5 miles from 125th Street in Harlem at the north end to Hanover Square in Lower Manhattan at the southern terminus. Construction of a Second Avenue Subway Line was actually first proposed in 1929, even before the two elevated lines were torn down, but did not advance significantly until the 1960s when a detailed design was developed, and construction of some sections got underway in the early 1970s. Unfortunately that work had to be stopped in the mid-1970s when New York City experienced a serious financial crisis.

The rebirth of the Second Avenue Subway Project began in 1995 when MTA New York City Transit started with a Manhattan East Side Alternatives (MESA) Study. This was followed by environmental impact analyses and in 2001 a full length subway alternative was selected. The Final Environmental Impact Statement (FEIS) was completed in 2004. Engineering design development divided the project into four phases, which are shown in Figure 1. Final design for Phase 1 was started in 2006, and in March 2007, the first construction contract was awarded.

Phase 1 includes twin bored running tunnels, and three new stations at 96th Street, 86th Street and 72nd Street. The alignment is initially linked to an existing station at 63rd Street and Lexington Avenue. By modifying this station, the first section of the Second Avenue Line can be merged with the Broadway Line to enable riders from the Upper East Side to continue on to midtown and downtown destinations.

Phase 1 has a target cost of $4.45 Billion and an estimated daily initial ridership of 213,000. The target completion date is December 2016. The work has been divided into 10 contracts, covering tunnel and station mining and excavation, heavy civil work, station finishes, and systems work. The 86th Street station extends between 83rd and 87th Streets, and includes north and south entrances, and two ancillary buildings housing station ventilation equipment. Contract C-26008 (5B), the subject of this paper, covers mining of a 980-ft long rock cavern for the station at 86th Street and heavy civil and structural work. The arrangement of the underground spaces for station, entrance structures and ancillary buildings is shown in Figure 2. The Contract was awarded to a Joint Venture of Skanska USA Civil Northeast and Traylor Brothers (STJV) in August, 2011 at a cost of $302 Million. The Contract duration is 37 months.

MUCK CONVEYANCE SYSTEM

Different Options for Muck Removal

When the quantity take-off was complete and the numbers were tabulated, the volume of muck to be removed stood out as a significant challenge: 160,000 bank cubic yards. Critical decisions were needed on how best to get the rock out of the shafts and into trucks for removal from the job site. This process required many discussions within the project team that considered what would be the fastest, the most cost effective, and the most reliable methods to remove the rock, and further what would provide the most versatility in performing the work, and most importantly, how could the muck removal be performed with the least impact on the local community? The debate progressed to a focus on two options, the first being the proven method of lifting the muck out with a large crane, storing it temporarily on site in a muck bin, and from there loading the muck into trucks with a rubber-tired loader. The second option was to use a relatively new method with a mechanical muck conveyance system that would lift the rock onto an elevated storage platform, transfer the rock to an overhead dumping location, and subsequently load the rock into trucks below at street level.

The first option involved the use of a large hydraulic crawler crane in conjunction with a muck bin positioned alongside of the crane, lifting buckets of rock to be dumped into the muck bin. This conventional approach would be similar to that used at many projects in the New York City area, such as for the soil excavation of the tunnel boring machine launch box between 91st and 95th Streets along Second Avenue as well as for the station cavern excavation for MTA New York City Transit's No. 7 Subway Line Extension Project. Advantages of this method included the ability to constantly muck out of the access shafts with the crane, providing temporary storage in the muck

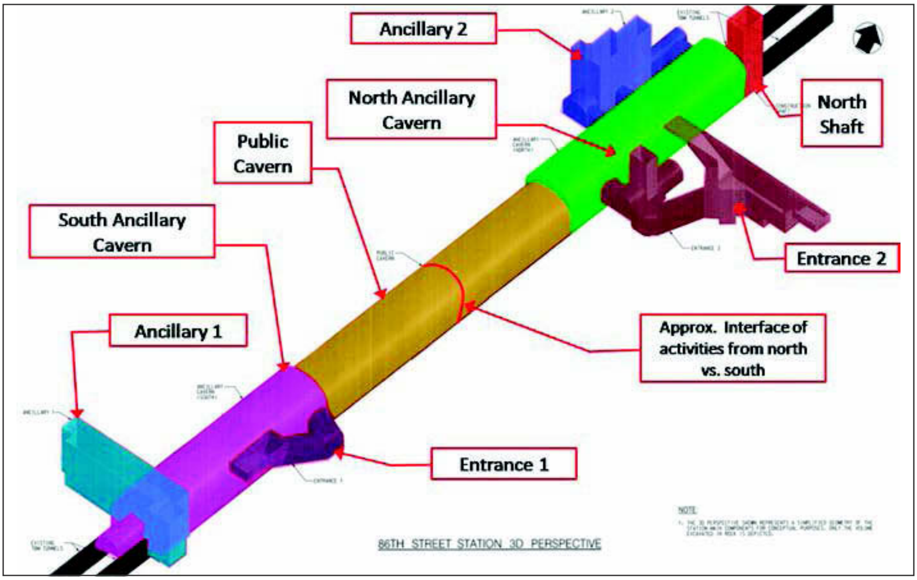

Figure 2. Arrangement of underground spaces for 86th Street Station

bin area, and only needing limited equipment and area to set up and start the work. Disadvantages of this method would be the need to provide and maintain two cranes (one at each access shaft) with capability to pull muck from the shaft at a high rate and raising and lowering major equipment such as drill jumbos, excavators, and loaders weighing as much as 83,000 pounds. Further, the use of a large crane at street level would greatly limit the area for surface stockpiling of the muck within the long, but narrow work zones around the shafts.

Alternatively a mechanized mucking system approach was developed for both access shafts of the job. Conceptually this followed the lead of the 72nd Street Station cavern mining contract, where the contractor developed a mechanized system in an enclosed structure surrounding each shaft site. The mechanical mucking system would be custom designed to address differences in site conditions at each shaft, related to the requirement to maintain four lanes of traffic along Second Avenue, the existing buildings adjacent to the work area, and a pedestrian access routes. Initially, the mechanized handling system concept was not the preferred choice, not only because it was an unproven method of muck removal for both local industry and project team members, but also because the approach would require an extensive effort to design, fabricate, manufacture, and install the system in a limited timeframe. However, positive aspects would be the opportunity to create a system extending the length of an entire city block (approximately 200 feet) rather than to limit the major working area to a specified radius around each access shaft, in addition to establishing separate work areas within the system for loading and unloading equipment and materials, storing rock for disposal, and loading haul trucks. Another factor supporting the mechanized system concept would be the ability to switch quickly from lifting muck containers from the shaft to lowering and raising equipment and materials at any time, thereby providing operational flexibility without adversely affecting the mucking production.

After a careful review of the two options, it became apparent that the mechanized system was the best choice for the project on the basis of the efficiency of muck handling and shaft hoisting operations and impact on the surrounding neighborhood.

Working with the Community

Two major concerns when making any decisions relating to the Second Avenue Subway 86th Street Station Contract are maintaining public safety and mitigating construction impacts on daily life in the community. With these objectives in mind, the muck handling system was designed, manufactured, and customized on site to achieve what was believed to be the most community sensitive system. Going with the mechanized system allowed an electric power source to be used rather than requiring multiple diesel-powered pieces of equipment and generators. The system set up at each shaft used electric power for the gantry crane, the moving of the muck containers on the elevated storage platform, and for the container dumping crane. The alternative to electric power would have been performing the work with a diesel powered crane and a minimum of one excavator at each shaft location running essentially continuously from 7 AM until 10 PM (15 hours) a day, five days a week over 11 months, for a total of 3,575 hours. This would clearly have been a nuisance to the community and impacted air quality. In addition to the reduction of diesel emissions, using electric powered equipment provided significant noise reduction in comparison to operating diesel powered cranes and excavators.

Regardless of the type of power used, the one noise source that was inevitable would be the initial impact of rock hitting the bed of the haul trailers as they were loaded. However, this noise would not be continuous, being limited to at most once every 10 minutes for approximately 5 seconds. The mechanized handling system also allowed the truck loading station to be enclosed in a sound-attenuated structure, another advantage for mitigating impacts on the community that would have been a difficult to match if the truck loading were to be performed by an excavator which would need to rotate from a muck storage bin to the trailer. Further to this point, the use of the muck system limited the noise of impact at street level to the first container load dropped into the truck. In comparison, with the conventional crane/excavator mucking approach, there would be the noise from dumping the rock into the surface muck storage bin in the work zone, then scooping up the rock from the storage bin, and finally the placement of the rock into the dump trailer, meaning a three-fold increase in impact noises occurring at the street elevation.

The configuration of the muck handling system also contributed to making the work areas more community friendly. The ability to keep all work in line with the system meant there was no need to rotate a suspended load out beyond the edge of the work zone to reach either the storage or lifting area as would be the case with a conventional crane, and therefore temporarily stopping pedestrian or vehicular traffic along Second Avenue would not be required. The number of lifts during a single shift would need to average 40 muck boxes raised and lowered or eighty lifts, and also a minimum of ten equipment and material lifts up and down, thus totaling an average of 100 occasions where a conventional crane would require either pedestrians or a minimum of one lane of traffic to be held for the duration of the lift. By eliminating this impact, the mechanized system greatly reduced potential traffic congestion on Second Avenue.

The muck system structure would not only be designed with traffic in mind, but also for the buildings located directly alongside the systems where tenants would be looking at the muck system out of their apartment windows for more than two years. With this visual impact, another key design factor for the project team was making the system as unobtrusive as possible, without adversely affecting operational efficiency. With this in mind, the system was designed with as low a profile as possible, with only limited sections that would need to be taller to maximize light reaching the apartment windows and minimize blocked views. Accordingly, the height of the elevated part of structure was limited to a level (13'-6'' at the tightest location) that would allow for the muck hauling trailers, equipment, and concrete trucks to pass underneath with ease. The tallest element of the system was the gantry crane, topping out at 45 feet above the street. However, the crane, which would need to move back and forth on rails along the elevated deck, was left open around the sides, with the exception of the upper ten feet of the gantry structure which was covered to protect the electrical hoisting equipment and to reduce any noise generated by it. Although this upper section was enclosed, the gantry crane would be moving back and forth and therefore not block window views at one location. The one stationary and fully enclosed section would be the truck loading station, which needed to be fully enclosed to reduce the noise of dumping into the trailers. This section was limited to a maximum height of thirty five feet above the street and a length of 30 feet.

Muck System Design Details and Variations for Different Conditions at the Shaft Sites

The designs of the muck conveyance systems were based on the principal purpose of removing rock from the station cavern excavation in the most efficient manner achievable. As mentioned previously, the volume of rock to be removed from the two access shafts was 160,000 CY in place which translated into an expected swelled volume of approximately 240,000 CY. The preferred choice of using 30 CY capacity truck trailers to transport the rock to the disposal sites would mean processing 8,000 truckloads. The plan to use these trailers was an important criterion in the design development of the system by driving the muck container size to hold a volume that would maximize the load volumes actually carried by the trailers and therefore maximizing the productivity of every step involved in the excavation cycle from the filling of the containers in the shafts to the time consuming trip to and from the dump sites. When this was taken into account, the weight of the rock also needed to be considered, as 30 CY of rock at an assumed bulk density of 2 ton/CY would weigh approximately 60 tons. When this was compared with the maximum weights of equipment to be hoisted with the same system and also to consider the heavy usage of the system, a single load of 60 tons was considered too large, and an impractical weight for a muck container to lift on a continuous basis. Therefore the boxes for the muck were sized at a volume of 15 CY or 30 tons.

Once the size of the muck boxes was determined, the remaining components of the muck system could be designed, such as the main hoisting gantry, the drive components for moving the boxes of muck along the elevated deck, and finally the dumping crane system. The company selected by STJV for the design and fabrication of the superstructure and components for the two muck handling systems was MCT Consult USA, LLC of Greenwich, CT. The main hoist gantry was sized in conjunction with the equipment to be lifted which included an 83,000-lb two boom drill jumbo rig and an 80,000-lb Caterpillar 980k wheel loader, leading to the use of a 44-ton rated hoist manufactured by DeMag Corporation. The components of the box moving system ("carousel") consisted of hydraulic powered drive wheels which propel the rock boxes along tracks along the elevated deck of the system between the main hoist and the dump hoist for truck loading station. A hydraulic power pack with separate power supplies for each of three rows of drive systems would be utilized for the carousel, which allowed for a maximum of 15 muck boxes to be stored on the deck for a total capacity of 225 CY of material. The remaining major mechanical component that needed to be designed was the dumping hoist, which was planned as a sideways rolling system that would lift the muck box to the dumping location inside the enclosed structure where the trailer would be waiting below. The dumping hoist would then lower the box onto a set of supports that would roll it over to an orientation of maximum 50 degrees off vertical allowing the rock to fall into the trailer. Once two muck containers were dumped in this fashion, the trailer below could exit the system and a new trailer would move into the loading station.

After the main components were determined, the design of the structural framework and supports of the muck system needed to be completed. Due to the existing utilities running below Second Avenue and two differently shaped but equally challenging work zones, this proved to be a time consuming task. For the north shaft system, the work zone had to fit around the existing access shaft footprint with a work zone width that was 44 feet at the north end but which narrowed to 28 feet at the south end to allow for four lanes of traffic between the work zone and a second work area located on the other side of Second Avenue. The transition point for the width of the north shaft work zone occurred at a critical mid-block location where the trucks would need to enter the site. Therefore the spacing of vertical supports for the muck system had to allow for a clear span of 48 feet would be required to allow trucks to turn into the site to reach the loading station. With the tight layout of the work zone, available locations for vertical supports were limited. The width of the carousel structure and dumping hoist equipment were controlled by the need to allow for three rows of muck boxes and perimeter walkways, thereby pushing the columns to the outer limits of the work zone. A plan view of the north shaft muck conveyance system is shown in Figure 3 and a photograph of the constructed system is shown in Figure 4.

At the south shaft, the work zone would be a constant 34-foot width for the entire length of the muck conveyance system structure. The southern end of the structure, however, would need to span over the access shaft and decking previously installed with an unsupported length of 58 feet. This constraint proved to be difficult with respect to finding proper supports of this part of the structure as it would need to pass over this 60-foot section with the self-weight of the steel supports, the weight of the gantry structure and system components weighing approximately 60,000 lbs, the 83,000-lb weight

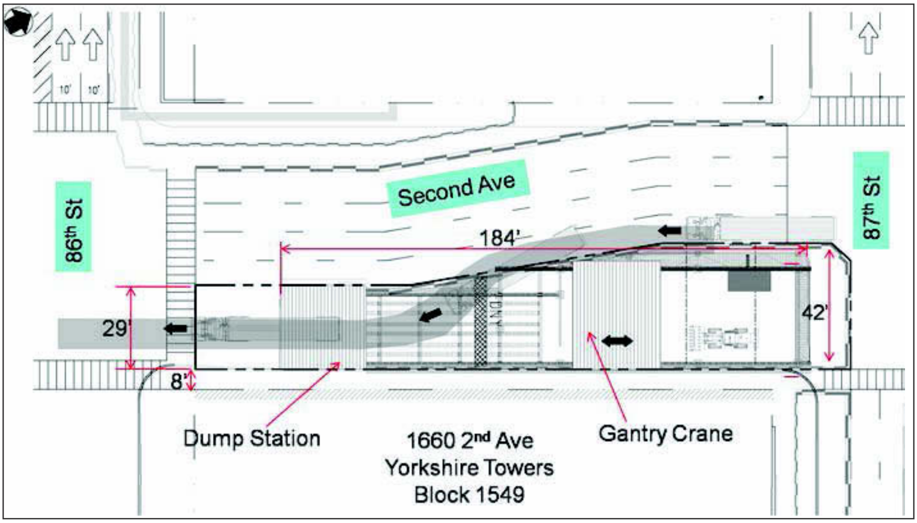

Figure 3. Plan view showing layout of north shaft muck conveyance system

Figure 4. Photograph of completed north shaft muck conveyance system

of the two-boom jumbo drill, and finally the consideration of a fully loaded dump trailer with 30 CY of material passing over the shaft on a deck at the same time. In addition to the heavy weights and long span, the steel framework providing the rails for the gantry system could not deflect by more than 0.2 inch between each of the four supports of the gantry. This led to the development of a truss framework for the support of the structure to pass over the south access shaft. In later stages of the system design, it was determined that there were two sections of the rock mass below in the shaft which would not be required to be excavated to the final depths and these could be used as sites for column supports that would help reduce the unsupported length of the main structure.

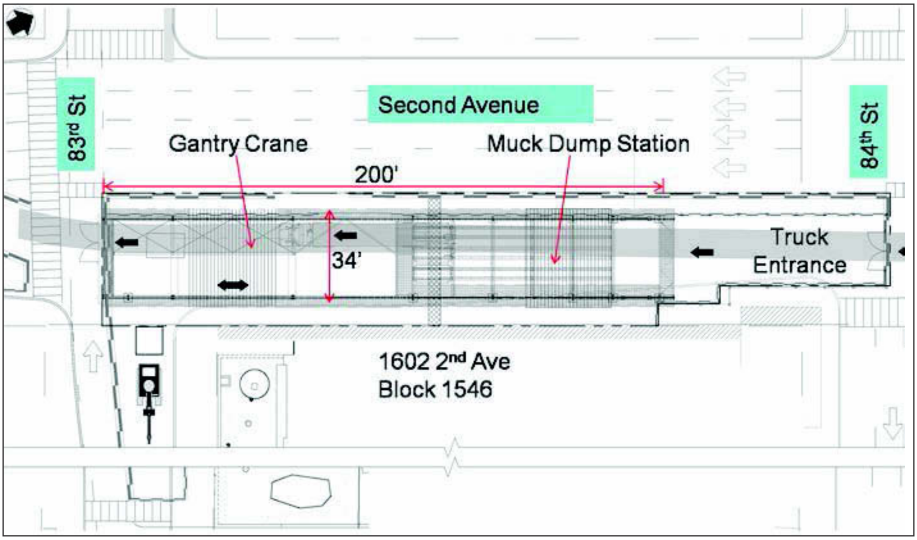

Figure 5. Plan view showing layout of south shaft muck conveyance system

A plan view of the south shaft muck conveyance system is shown in Figure 5.

Muck System Performance

Based on the contract schedule requirement that the 160,000 CY of rock be excavated for the 86th Street Station cavern mining contract in 11 months, the average production parameters are as follows:

- 160,000 CY — In-situ rock volume to be excavated
- 240,000 CY — Swelled volume of rock to be removed
- 16,000 containers — Total number of muck boxes of swelled rock to be hoisted out of the shafts
- 8,000 trailers — Total number of 30 CY dump trailer trucks to be loaded for muck removal
- 1,450 containers — Required monthly production for muck boxes hoisted out of the shafts
- 725 trailers — Required monthly production for dump trailer trucks to be loaded
- 66 containers — Required daily production of muck boxes
- 33 trailers — Required daily production for dump trailer trucks

To meet these production parameters, over a 15-hour allowable daily window for surface work activities, an average of one box every 25 minutes would need to be hoisted out of the shafts to maintain the contract schedule. However, the need to lift equipment and materials and move personnel in and out of the shafts for the blasting operations results in the actual time available for muck removal being reduced to approximately 12 hours in a day, which would require a cycle time for lifting boxes to one box every 20 minutes. Actual performance data collected to date indicate that the typical time required to lower a muck box into the shaft, load the box, hoist it back out and return it to the elevated carousel deck takes approximately 13 minutes, which would appear to be plenty of time. The other variable in achieving the required production rate for rock removal is the time needed to dump a box into a trailer. Actual data for this part of the mucking operation indicate that it takes approximately 8 minutes for the dumping hoist to lift the box from the carousel, move it to the truck loading station, dump the box, and return the empty box back to the carousel. The fact that the time to dump a box is quicker than the time it takes to fill a box is preferable in the sense that the dumping of the boxes should not delay the gantry crane from lowering an empty box into the shaft and having a place to put the full box on the storage deck carousel. However, the one additional variable affecting actual production is the cycle time required for loaded trucks to reach the muck disposal sites and return, which can be affected by regional traffic congestion during the morning and evening rush hours.

At time of writing, substantial information has been collected to compare the actual production rates for the muck handling system with the anticipated required production rates. A period of three weeks during the peak rock removal time has been checked as a comparison to those which were required. During this three week period, the actual number of days of mucking operations was 14 days. The production achieved clearly demonstrates the capability of the muck conveyance systems as built and operated to perform at the level required to support the contract schedule.

- 11,710 CY — In-situ rock volume excavated
- 17,565 CY — Swelled volume of rock excavated
- 1,171 containers — Total number of muck boxes of swelled rock hoisted from the shafts
- 586 trailers — Total number of 30 CY dump trailer trucks leaving the site
- 84 containers — Removed on a daily average (compare to the average required daily rate of 66 containers)

Owner and Community Reaction

The muck conveyance systems were constructed in the spring and early summer of 2012. The MTA was concerned about how the community would react to these major structures, and made several presentations at community meetings before the systems were installed to show the impacted residents what they would look like and how they would operate. After the structures were erected, and the systems started to operate, MTA requested STJV to install additional acoustical paneling on the sides of the structures to screen operations on both the upper and lower levels. Doors were also added to the truck loading station structures to further reduce the noise levels resulting from loading of the dump trailer trucks.

Now that the muck conveyance systems have been in place and operated for several months, the experience with the community has been largely positive. Although neighborhood residents would prefer not to have these structures adjacent to their apartments, the systems essentially appear and function as the community was told to expect, and consequently there have not been many complaints. It is clear that the impact of these systems on the community is far less than would have occurred if STJV had employed the conventional boom cranes, surface muck storage bins, and diesel-powered equipment for loading trucks for their mucking operations.

VENTILATION SYSTEM

Ventilation for Dust Mitigation in a Sensitive Community Environment

Now that a community sensitive muck conveyance system had been developed for each access shaft site for the 86th Street Station cavern mining, it was determined that the ventilation system for the drill and blast operations would also need to address community concerns. A large part of the planning for this job was getting to know the neighborhood in which we would be working. Fortunately because the 72nd Street Station cavern mining contract for the Second Avenue Subway Project was already well underway when the 86th Street Station cavern mining work was awarded, we were able to learn about the feelings that the local community had towards the construction going on in their neighborhood. We quickly learned that we would be dealing with an affluent community that was well informed about the MTA's requirements for environmental controls during construction. Control and mitigation of dust and smoke generated by blasting had become a very serious issue for the 72nd Street Station work, with numerous complaints from residents and several reports by the news media on air quality related to the subway construction. Residents in the vicinity of the 86th Street Station work were highly aware of what had been happening with the 72nd Street Station mining operation. With that background, it was quickly concluded that, just like the mucking operations, a conventional ventilation system typically used for a drill and blast operation would not be sufficient to mitigate community concerns about air quality. We knew that once drilling and blasting operations commenced, we would be watched very closely, and that additional measures would be necessary to complete the job on schedule.

The contract specification for dust monitoring on site required that particulate concentrations were to be measured using temporary monitoring stations which would utilize dust track monitoring devices capable of measuring particle sizes less than 10 micrometers in size (PM-10). The temporary stations would be located at the upwind and downwind perimeters of the work zone. The particulate level limit which needed to be satisfied was 100 micrograms per cubic meter (mcg/m³). If this limit was exceeded, the dust track monitors would send out an alert and dust suppression measures would have to be implemented.

Choosing the Right System for the Job

As the initial stages of the job progressed and we learned about the specific community issues associated with building a subway in the Upper East Side of Manhattan, it was decided to move ahead and research a more advanced ventilation system for the job. The general concept was a system that would employ two fans at each access point to the cavern excavation below. One fan would be a dedicated supply air fan that would provide fresh air to the workers at the heading during normal operations, and the other fan would be a dedicated exhaust fan that would be used after a blast to capture the air and direct it to the method of treatment before being dispersed into the environment. The initial thought for the method of treating this air was a dust scrubber system in order to remove the majority of the particulates out of the air before exhausting it. As research into different types of scrubbers progressed, two specific scrubber systems were considered.

The first system evaluated was the simpler of the two systems; it consisted of a connex box to which the ductwork from the exhaust fan would be connected. Inside the box were baffles which increased the distance and time the dirty air had to travel to pass through the box, and mounted inside was a simple sprinkler system similar to one that could be found at a local home improvement store. As the dust laden air traveled through the system, the large particles would be mixed with the water and collected in a basin at the bottom of the box, and this water could then be sent either back to the muck pile or treated for proper disposal. An outlet hole in the top of the box covered by a protective screen allowed the remaining air to be exhausted out into the surrounding environment.

A visit was made to a construction site where this system was being used to observe it in action. Measurements were taken with the dust track system which was to be used on our site and it was determined that this scrubber system was capable of meeting our specification requirements for particulate matter. However, it was observed during the site visit, that there was still visible dust coming out of the top of the unit and dust residue could be seen in the area immediately surrounding the point of exhaust. Another disadvantage of this system was that there was no back up data available indicating the particle sizes that it was capable of removing, and it was important to be able to show the MTA and, if necessary, the community hard data from a proven system that had been used before in the industry. Since this was a custom unit built by the contractor using it specifically for their job, no prior testing had been done that would yield the essential performance data.

The second system that was found and ultimately selected for the 86th Street Station work was a wet dust collector system made by Schauenburg. This type of system is commonly used on tunnel boring machines and in mining operations; it treats the dirty air created from the mining process to a point where it is actually acceptable for the workers to breathe. For this alternative, we were able to present back up data to the MTA that was available from studies that have been done for these units as they have been in use for years. The backup data showed that these units were capable of providing an effluent particulate concentration as low as 1 microgram per cubic meter, while the contract specification required we stay below 100 micrograms per cubic meter. It was immediately clear that this system would go above and beyond what we would need to meet the specifications for the job, but at a considerably higher cost then the first simpler option described above. After presenting the options to the MTA, it was decided to go with the more advanced Schauenburg system, with MTA support by change order for the additional costs involved with purchasing this system to achieve the enhanced level of dust mitigation expected by the community after the blast-generated dust problems experienced with the 72nd Street Station cavern excavation.

Final System in Use

Having selected the specific wet scrubber system to be used for the cavern mining work, we were able to advance the design of the complete ventilation system. At each access shaft, a VFD-controlled, 66-inch, 125-horsepower, axial Jet-air fan supplied by Mining Equipment was used with steel duct to provide the supply air to the workers during normal operations. This fan provided about 100,000 CFM to the heading. After a blast, the supply fan would be turned on to clear the dirty air from the blast from inside the cavern and direct it towards, and eventually up, the access shaft. In addition to a supply air fan, a dedicated exhaust fan feeding a Schauenburg scrubber unit also was installed at each access shaft to the cavern excavation and steel duct was run about twenty feet down into each shaft off of the fan. The fans used for the exhaust were 48-inch, 150-horsepower, axial Jet-air fans also supplied by Mining Equipment. After a blast, a soft start controller would turn on both the scrubber unit and the exhaust fan that feeds it, and as the air being flushed out of the cavern by the supply fan comes up the shaft it is collected by the exhaust fan and fed directly into the scrubber unit for treatment. This all would be done with shaft covers in place to prevent any leakage of fugitive dust into the environment.

Once the air enters the scrubber, it is initially sent through a venturi unit fitted with spray nozzles around the perimeter of the opening. The spray nozzles are fed by a high pressure pump which provides a very fine spray to allow for smaller dust particle removal. The dusty air is mixed with these fine water droplets in order to saturate the dust laden air in preparation for the secondary treatment. The secondary treatment is provided by a filter bank consisting of PVC spin filters which mechanically separate the saturated dust particles created in the initial treatment. The spin filters consist of a plastic tube with a stationary spinner mounted at the intake end. As air carrying the saturated dust particles is forced through the inlet of the filter tube, it engages the stationary spinner, which creates a centrifugal force sufficient to send the dust particles to the periphery of the tube, thereby separating them from the air. The now clean air

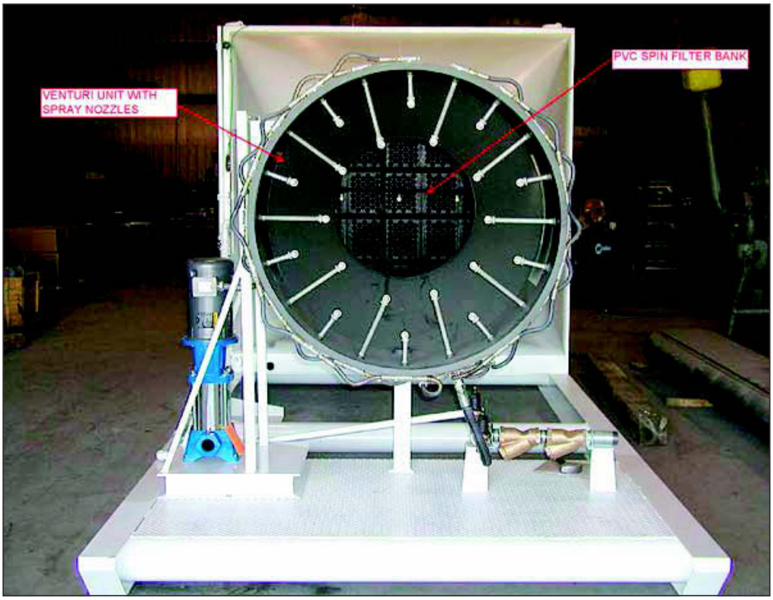

Figure 6. End view of wet scrubber system showing venturi unit with spray nozzles and PVC spin filter bank

passes through a duct located at the center of the outlet end of the tube. The extracted water and dust are drained through the bottom of the spin filter bank and are collected in a sump tank located at the bottom of the scrubber. This dirty water is then pumped out of the tank and into the muck pile for disposal. The moist clean air finally passes through a demister panel which removes the majority of the remaining water content, such that little to no moisture can be detected in the effluent air dispersed back into the environment.

An end view of the scrubber system showing the venturi unit with spray nozzles and the filter bank of PVC spin filters is shown in Figure 6. A photograph of the system installed at the south shaft is shown in Figure 7.

Ventilation System Performance

The scrubber systems were installed at the north and south shafts in the summer of 2012. At time of writing approximately 60% of the required volume of station cavern rock blasting has been completed on the project and the system has performed better than expected. After a blast, typically no visible dust or smoke can be detected in the area immediately surrounding the access shafts, nor can it be seen exiting the scrubber unit itself. Another positive result was the lack of maintenance required for the wet scrubber systems. Once a week, the PVC spin filters are checked to ensure that they are not becoming clogged with dirt. In the rare cases where the filters show some build-up of material, they are easily cleaned by spraying them down with a water hose. Additionally, the spray nozzles throughout the scrubber system are checked to ensure that they are functioning properly and pressure gauges on the scrubber are a simple means to monitor whether the spray is at the correct pressure to effectively clean the dusty air.

While still in the planning stages, some issues were raised about using this system during a cold New York winter when the temperature drops below freezing almost daily. The concern was that the effluent air would have remaining water content sufficient for

Figure 7. Photograph of wet scrubber system installed at the south shaft

it to condense and fall onto the street below, causing an icing hazard. This has turned out to not be a problem because the effluent air is almost completely dry by the time it is discharged into the air. Another concern was having the entire unit freeze up, since this system requires water for every stage of treatment; specifically, the worry was the spray nozzles at the venturi unit and the PVC spin filters would ice up to the point that no air would be able to pass through, and the collection sump at the base of the scrubber unit could become a block of ice if left too long without being pumped out. In order to address these concerns, shed enclosures were built around each unit with heaters installed inside so that the scrubbers could be kept at a constant temperature even in the winter months. This has proven to be an effective method of dealing with these potential winter season freezing problems.

We have also found that by using this method of ventilation with wet scrubber dust mitigation after a blast, the air quality in the cavern became acceptable to work in more quickly than would have occurred when using traditional ventilation methods. Because the workers are able to go back down into the cavern sooner, cycle times for the drilling and blasting operations have improved.

Owner and Community Reaction

The MTA has been pleased with the performance of the ventilation and wet scrubber systems implemented on the 86th Street Station cavern mining contract for the Second Avenue Subway. Because of the excellent results achieved to date with this dust mitigation method, there have been little to no community complaints related to dust or smoke after a blast which is a significant accomplishment considering the dense urban surroundings in which the job is taking place. The major concern of the community about blast generated dust exposure associated with the earlier 72nd Street Station cavern mining work has not developed for the similar work at 86th Street. This ventilation system has done exactly what it was intended to do, and as a result has assisted in the actual construction of the job, and perhaps more importantly, has avoided what could have been a difficult environmental issue with the actively concerned community.

ACKNOWLEDGMENT

The authors thank Harshad Pandit for contributing to this paper.

RECENT BELT CONVEYOR APPLICATIONS IN THE UNITED STATES

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ABSTRACT

Belt conveyors are state of the art as a stabile muck removing system in the worldwide tunneling business. This paper will review the latest tunneling operations in the USA, the Euclid Creek Tunnel in Cleveland, the Blue Plains Water Tunnel in Washington D.C. and the SR99 Tunnel in Seattle. All these TBM drives are equipped with tunnel belt installations, this paper will analyze the different requirements caused from basic conditions at each particular project and the technical solutions that led to a successful application of belt conveyor systems. Project requirements led to special set ups such as the vertical belt storage cassette, the horizontal double stack cassette, vertical conveyor and complex on surface arrangements.

INTRODUCTION

In 2011 several Tunnel Boring Machines (TBMs) on the U.S market were awarded, most of them with the opportunity to install continuously extendable belt conveyor systems, which is the state of the art of muck removal, hence H+E Logistik GmbH from Bochum, Germany founded the subsidiary H+E Logistics USA Inc. in Sumner, WA. The idea was being present on the U.S. market, closer to clients and to the market itself. Challenging projects were awarded in 2011 for realization in 2012 and 2013, which to mention are the Euclid Creek Tunnel in Cleveland, OH, the Blue Plains Water Tunnel in Washington D.C., and the Alaskan Way Viaduct Replacement in Seattle, WA.

All tunneling projects with conveyors have their own technically demands which have to be considered during design, assembly and operation of conveyor systems. This to fulfill is the challenge but also daily business for conveyor supplying companies. Solutions for the particular demands are only to be worked out in a jointly working process between the contractor and the supplier.

EUCLID CREEK TUNNEL CLEVELAND, OH

Keeping Lake Erie clean is the goal for the enlargement of the sewer district in Cleveland, OH, the Euclid Creek Tunnel (ECT)—now under construction—is part of several tunnels that will be built to drastically reduce the overflows. The McNally/Kiewit ECT Joint Venture is the construction joint venture having an 8.23m diameter Herrenknecht hard rock TBM driving the tunnel to its final length of 5,500m, the winding alignment is shown in Figure 1.

The de-mucking system being capable handling the advance rates of the tunnel boring machine is a conveyor system. Due to the site arrangement the conveyor system comprises of a continuously extendable tunnel belt conveyor, following the TBM to the final length of the tunnel, a vertical conveyor lifting the soil up the 60m shaft to the surface. With on surface conveyors the muck is piled up by a pivoting stacker on jobsite before transported away by trucks. Figure 2 shows the on surface conveyor system arrangement, with the vertical conveyor reaching the surface (left hand side) and the pivoting stacker piling the soil up.

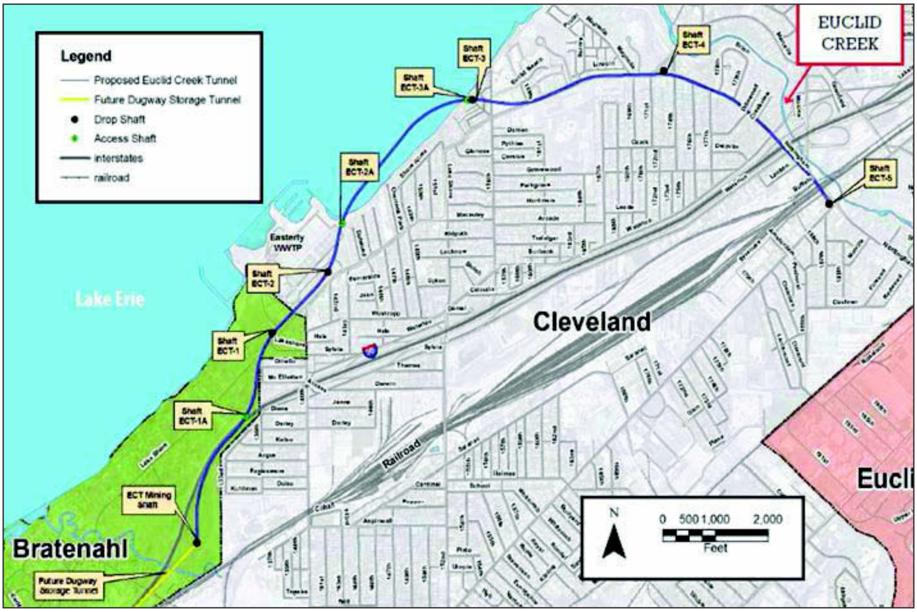

Figure 1. Euclid Creek tunnel project location map (*Source:* Northeast Ohio Regional Sewer District)

All conveyors are capable to transport 1,200t/h and are therefore equipped with 1,000mm wide belts; the exception is the vertical belt conveyor as the shape of the belt itself is different from a rubber belt for horizontal conveyors. The tunnel belt, a textile belt, 1,000mm wide EP 800/4 5+3 is driven by a 184kW main drive located in the access shaft.

The drives of all conveyors in the system are VFD controlled by an Alan Bradley PLC with a main control container placed on top of the shaft. The communication between the PLC in the main control container and the VFDs and the TBM is done Profibus and the emergency installation is controlled by Dubline, which allows having the indication which emergency switch has been activated without checking every single emergency switch for resetting. A touch pane in the main control container and in the TBM visualized all relevant information for the operator.

In addition to the main drive of the tunnel belt conveyor three top strand and two bottom strand booster drives will be installed during the tunneling operation. The booster drives are required due to the winding alignment of the tunnel with several curves of radii between 610m and 426m as the belt tension, which constantly rises with the increasing tunnel length during the TBM drive, has to be reduced in order to track the belt through the curves. Each top strand booster is equipped with a 184kW drive and each bottom strand booster with a 104kW drive, in total 944kW will keep the muck being conveyed from the TBM to the shaft. The shaft is the logistical bottle neck as the de-mucking equipment as well as everything to supply the TBM has to go through it.

Finding space for the conveyor installation that had to fit into the shaft was a back and for as the drive of the tunnel belt conveyor as well as the return station of the vertical conveyor with a foot print of dimension of approximately 10m × 4m × 9m required to place the take up unit on top of the shaft. This setup caused a technically ambitious belt bending installation, because the belt has to move off center from its position in the tunnel but keep the tunnel entrance clear for the logistic traffic. One can see in Figure 3 the tunnel belt conveyor reaching out the tunnel entrance with transfer point

Figure 2. ECT on surface conveyor system

Figure 3. Conveyor installation in the shaft

to the vertical conveyor and the tunnel belt directed to the top of the shaft off centered from the tunnel belt.

The limitation of the site facility on surface required a different set up of the horizontal take up unit. A tailor-made design resulting in a double stacked cassette with a capacity of 600m conveyor belt (Figure 4) allowing the TBM 300m of advance before new belt has to be fed to the cassette.

This design is combination of the advantages of the typically horizontal cassette design with variable capacities up to 800m allowing TBM advance of 400m and the vertical cassette which has a footprint of slightly more than a 20 foot container but with limitation of capacity to 400m.

Following the line of conveying from the tunnel, the vertical conveyor lifts the excavated soil up the 60m deep shaft driven by 2 × 184kW, the belt is a 1,600mm wide steel reinforced belt with corrugated sidewalls and cleats. To protect the people working in the shaft area from dropping spillage, the vertical section is covered. At the top of the shaft the vertical conveyor discharges to the on surface conveyor arrangement. At the head station (Figure 5) with a footprint of 15m × 6m × 11m the belt is transferred to a horizontal position so the gravity could be used to clear the cleats from the soil.

Figure 4. Double-stacked cassette

The belt cleaning device for flat conveyor belts are scrapers, typically installed as a twin set at the discharge drum comprising of a pre-scraper with segments made of polyurethane and a main scraper with hard metal segments. But this effective solution does not work with belts that have corrugated side walls and cleats installed. To support the gravity is the most effective technical solution to clean such belts; therefore a knocking pulley is installed at the horizontal discharge section of the head station.

Figure 5. Vertical conveyor, head station

The conveyor systems ends with the stacker shown in Figure 6 with a discharge height of 14m and has pivoting range of 30 degrees which is enough to pile up around 4,300m³. The drive consists of 2 × 37kW whereas the drive for the horizontal travelling is designed with 2 × 4.6kW

The conveyor setup for the Euclid Creek Tunnel was commissioned already in 2012 and had to overcome its challenges as described in this paper and some more.

ALASKAN WAY VIADUCT REPLACEMENT IN SEATTLE, WA

The Alaskan Way Viaduct Replacement in Seattle, WA also known as the SR99 tunnel because the State Route 99 will cross Seattle beneath in the near future instead of passing by the waterfront of the scenic view of Seattle's skyline. This giant project is in the focus of everybody who is linked to the tunneling business. The Seattle Tunnel Partners Joint Venture formed by Dragados USA Inc. and Tutor Perini Corp. has been awarded for this giant project and chose a 17.45m diameter EPB TBM supplied by Hitachi Zoisen Corp. to drive this 3,000m long tunnel, a H+E Logistics USA conveyor

Figure 6. Pivoting stacker

Figure 7. SR99 tunnel route (Source: Washington State Department of Transportation)

system starting with the TBM belt conveyor will transport the masses of excavated soil to the surface and load into barges. The route of this inner city project is illustrated in Figure 7, starting at the south portal on the left side and ending 3,000m further at the north portal.

The largest tunnel requires a large conveyor system being capable to transport 2,800t/h of excavate soil. The belt conveyor system comprises of a continuously extendable tunnel belt conveyor and a complex on surface conveyor arrangement all with a belt width of 1,600mm.

The tunnel belt conveyor with a 1,600mm wide EP1000/4 5+3 belt is driven with by a main drive equipped with 3 × 362kW located in the shaft of the south portal. A horizontal loop take up with a capacity of 800m of rubber belt grants a TBM drive of 400m before new belt has to be fed into the cassette.

The tunnel belt, following the TBM on its 3,000m long drive beneath downtown Seattle, transfers the spoil in the shaft to overland conveyors which transport it to a barge at the pier as schematically illustrated in Figure 8. As soon as the overland conveyor reaches out the shaft at ground level, a completely enclosed bridge structure designed to protect the conveyor and spoil from weather conditions as well as the surroundings from noise and dust and noise emission, which a major concern at inner city projects, especially at large size projects as this one. The tunnel belt drive in the shaft and the first overland conveyor with bridges and transfer tower to the following second

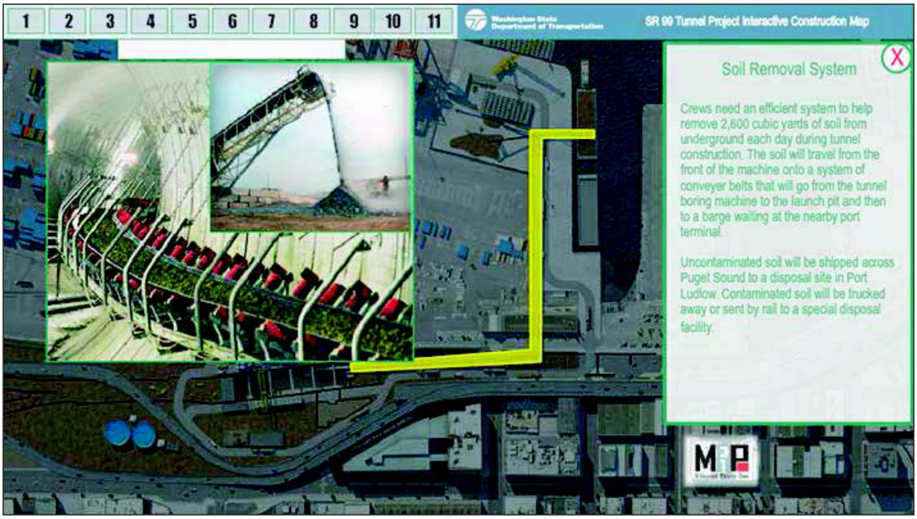

Figure 8. Conveyor route at SR99 tunnel (Source: Washington State Department of Transportation)

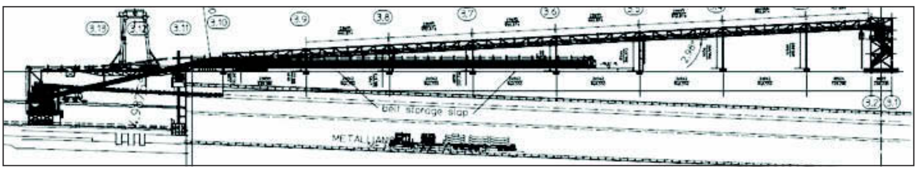

Figure 9. Overland conveyor #1

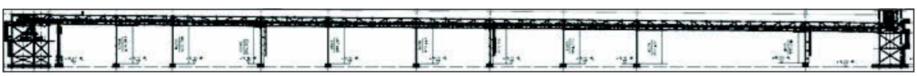

Figure 10. Overland conveyor #2

overland conveyor is shown in Figure 9. The first overland conveyor is driven by 2 × 184kW due to the inclination.

Along the pier at the waterfront the second overland conveyor consisting of enclosed bridge structure transports the spoil for 300m to a complex transfer tower (Figure 10) and is driven by 184kW, which is less than the first overland conveyor due to the horizontal alignment. The bridge structure includes a 46m wide span bridge to overpass an existing building of harbor facility (which is not illustrated in Figure 10).

The transfer tower at the end of the second overland conveyor shelters three transfer points which results in a complex design. From that tower the TBM excavated spoil is either transferred to the barge loading conveyor or to an intermediate stock pile on the pier.

Figure 11 visualizes the conveyor setup at the barge loading are. The priority is on loading into a barge but to be prepared for any event that does not allow loading the barge the intermediate stock pile allows to keep up tunneling operation. Whether the mass flow is guided to the barge loading conveyor or to the stacker is determined by the position of a motorized switch chute installed at the discharge of the second overland conveyor.

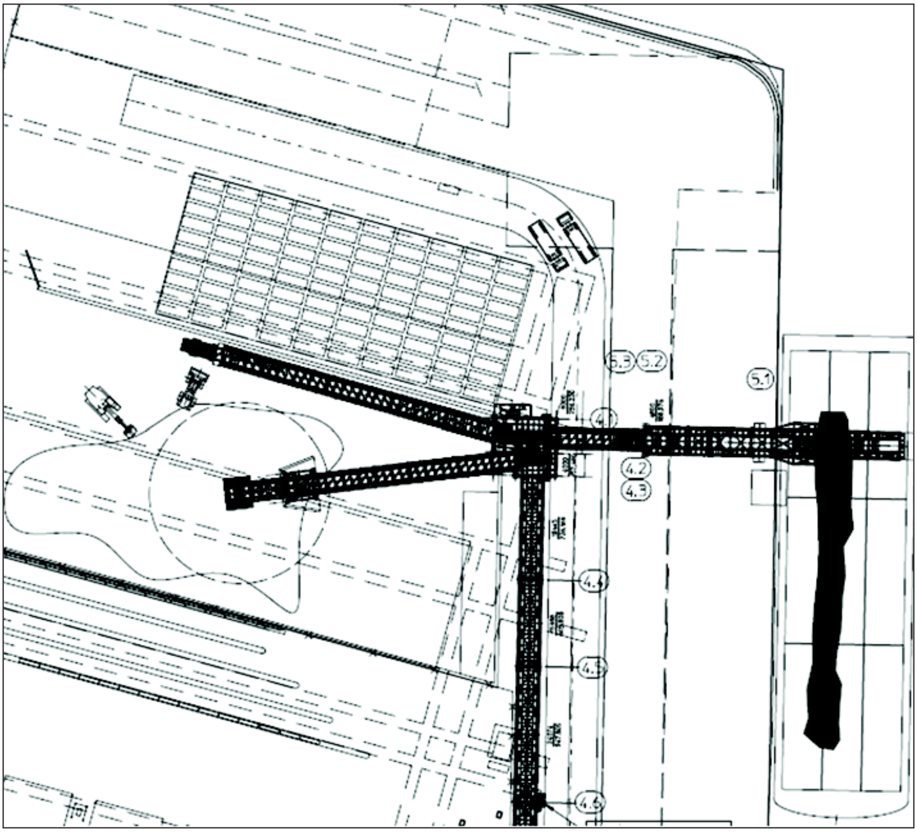

Figure 11. Conveyor setup at barge loading area

As the spoil has to be re-loaded to the barge a conveyor which will be loaded by front loaders at the muck pile, transfers the spoil also to the barge loading conveyor.

The control system for this giant project is as complex as the mechanical setup. The principle of having VFD drives at all conveyors and a PLC system being the heart is the same, also that a Dubline system is used for the communication of the emergency equipment but for the communication of the main control container with the TBM and all conveyor drives fiber optic cable is installed. A camera system allows monitoring all transfer points and visualizes the TBM driver these spots in real time.

As complex as this set up is, it is mandatory that the technically solutions prove the capability of the conveyor belt system from the beginning of the drive to the end.

This important project is as visible in a city as only few in the world, which causes public interest in the progress and success. It is a challenging project but it also proves that the technical solution presented led to being part of it.

BLUE PLAINS WATER TUNNEL IN WASHINGTON, DC

Keeping the rivers clean is the goal of the tunneling job at Washington D.C.'s Blue Plains Tunnel. The construction companies Traylor-Skanska-JayDee with Halcrow form the joint venture for this important project. The winding tunnel (Figure 12) with a length of 7,300m will be driven by an 8m diameter Herrenknecht EPB TBM will be mucked out by a tunnel belt conveyor.

The tunnel belt EP1000/4 5+3 with a width of 1,000mm will be driven by an 184kW main drive. Following the same principle as already mentioned earlier in this paper, due to winding alignment in with radii between 300m and 265m booster drives are required to reduce the belt tension in order to track the belt through the curves. There will be three top strand booster drives installed each with 184kW and two bottom strand booster drives with 103kW. This setup will allow the conveyor system to follow the TBM on its 7,300m long drive excavating 700t/h and even more.

All drives are VFD controlled by an Alan Bradley PLC, whereas the control container will be located on top of the shaft.

The focus for this project lies on the narrow shaft arrangement at the access shaft with only 40m diameter and a depth of approximately 35m. The tunnel belt discharges the excavated spoil into a muck skip carrousel provided by the JV from where the spoil is lifted to the surface. Beside all logistic traffic to supply the TBM the conveyor system has to fit into this narrow area. To safe space a vertical take up is the most efficient solution, allowing the TBM to proceed for 200m before another 400m of conveyor belt has to be filled into the cassette. The small footprint which is approximately the size of a 20 foot container grants a compact setup. As the tunnel belt and the cassette are installed in line and the discharge is just in between both items, a complex belt bending is required to guide the belt from the drive underneath the carrousel into the cassette. Filling new belt into the magazine required with this narrow space additional support on top of the drive unit to place a belt bobbin. The shaft setup is illustrated in Figure 13.

This project shows that there are technical solutions to also install conveyor systems in narrow shafts like this one, in order to be able to profit from the speed of tunnel boring machines. Projects like this confront tunnel belt systems supplier with challenges and keep the process of engineering on.

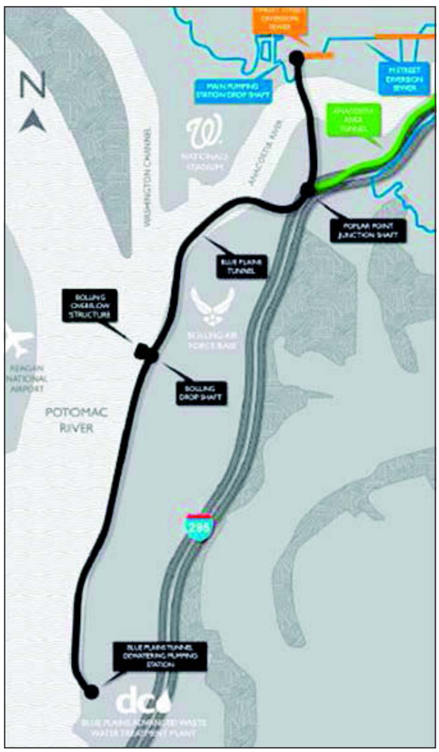

Figure 12. Route of Blue Plains tunnel (Source: District of Columbia Water and Sewer Authority)

CONCLUSION AND OUTLOOK

As the advantage of belt conveyors in tunneling to support the TBM drives all over the world are known, belt conveyors will be part of time and cost improving philosophies. The application of conveyors in tunneling jobs has increased and will be improved due to sophisticated engineering companies who are established in this market with their niche technologies.

It is important to be part of challenging project to prove the capability and to gain experience from these projects in

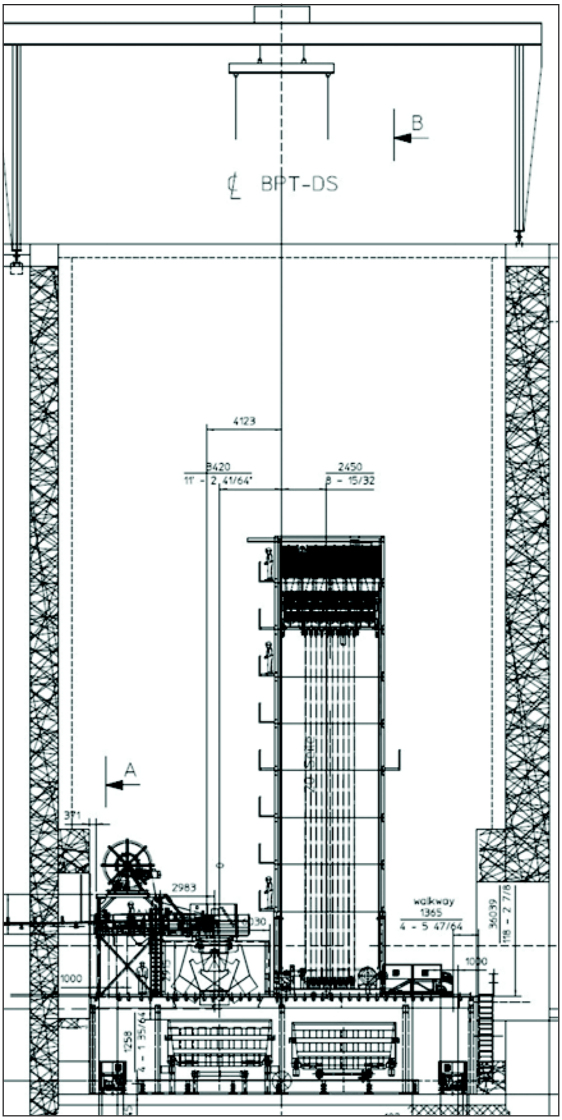

Figure 13. Conveyor installation in shaft

order to keep the technology developing process on and to convince that technology remains state of the art.

SUSPENDED PLATFORM HEADING SYSTEMS FOR SAFE, HIGH-PERFORMANCE TUNNELING

Alfred Moergeli ▪ moergeli + moergeli consulting engineering

Alberto Belloli ▪ Rowa Tunnelling Logistics AG

ABSTRACT

How do you deliver an innovative solution for safe high-performance tunneling? South of the Gotthard Base Tunnel, the world's longest infrastructure tunnel, Switzerland is constructing the 9.3-mile-long High-Speed Railway Ceneri Base Tunnel by Drill & Blast. In the Himalayas, India is constructing the 5.6-mile-long Rothang Pass Highway Tunnel for an avalanche-safe road connection under the Rothang Pass by Drill & Blast. Switzerland-based solutions provider Rowa Tunnelling Logistics AG (Rowa) successfully manufactured, supplied, erected and commissioned five suspended platform heading systems. Tunnel logistics is separated from construction and transportation activities in the invert. True innovation results in safe high performance construction processes, verified in situ through Process/Safety Reviews by moergeli + moergeli consulting engineering (m+m).

SWITZERLAND'S ALPTRANSIT PROGRAM

Switzerland's AlpTransit Program with its main part, the Loetschberg Base Tunnel and the Gotthard Base Tunnel (GBT), has been subject of numerous publications, including various papers and presentations for the North American Tunneling (NAT) and Rapid Excavation and Tunneling Conferences (RETC).

THE CENERI BASE TUNNEL (CBT)

The Ceneri Base Tunnel (CBT) is the southern continuation of the Gotthard Base Tunnel (GBT) and completes the new flat rail link through the Swiss Alps towards Italy.

Similar to the Gotthard Base Tunnel, the Ceneri Base Tunnel (CBT) system consists of two single track tubes which lead from the north portal at Vigana/Camorino to the south portal at Vezia, connected with cross passages about every 300 meters. All four main headings are executed in parallel and have started out of an underground cavern (Caverna Operativa), which was accessed from Sigirino through a 2.3-km-long access tunnel with about 5% decline, driven by a Tunnel Boring Machine (TBM) under a previous contract.

Project Design and Tendering

Expected geological conditions in the northern headings as well as towards the south up to the fault zone "linea della Val Colla" reasonably excluded the use of a TBM. The section from the "linea della Val Colla" to the breakthrough with the heading from the southern portal was deemed suitable for both conventional and shield TBM headings. Two joint ventures bid the project based on drill & blast only. One joint venture submitted a bid as a combination of drill & blast and shield TBM for the southern section. The

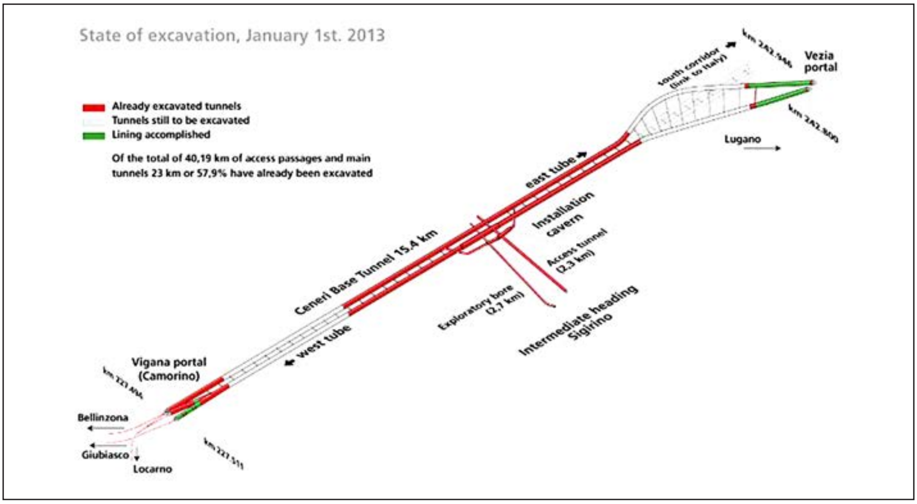

Figure 1. CBT—Overview of the Ceneri Base Tunnel (CBT) project (Courtesy of ATG, www.alptransit.ch)

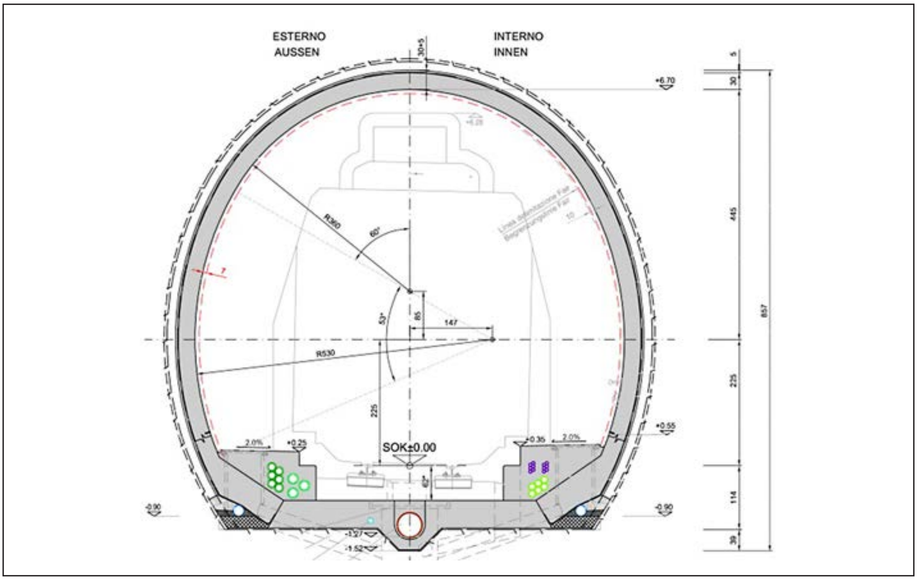

Figure 2. CBT—Standard cross sections for drill & blast (Courtesy of ATG, www .alptransit.ch)

winning joint venture, Consorzio Condotte Cossi (CCC), bid both alternatives, whereas the option with drill & blast turned out to be less costly.

In order to meet the important deadline for tunnel completion all bidders included highly mechanized drill and blast headings in their proposals. This allowed the owner to select the option which

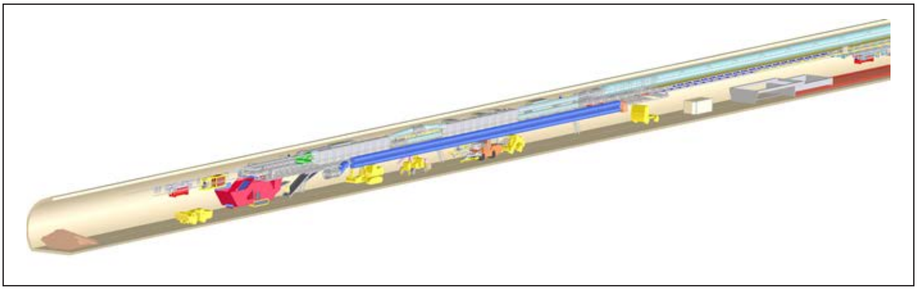

Figure 3. CBT–Highly mechanized back-up system for main headings (drill & blast) (Courtesy of Rowa, www.rowa-ag.ch)

- Meets the time constraints,
- Is economically more interesting,
- Bears the lowest risks in case of deviations from the expected geology.

Previous Experience

When planning the suspended heading platforms for the CBT, as always in close cooperation with the customer, Rowa was able to learn from current experience and valuable insights with recent heading mechanization projects such as suspended heading platforms for the Vereina Tunnel, the Mitholz and Raron sections of the Loetschberg Base Tunnel, as well as for the Amsteg and Sedrun sections of the Gotthard Base Tunnel. Each system basically consists of a

- Ventilation platform,
- Heading platform,
- Invert platform,
- Crusher,
- Conveyors, and
- Monorail (single track suspension rail).

Innovative Ventilation Provides Fast Access to the Face While Delivering a Safe and Healthy Environment

Optimal ventilation and cooling are important measures to guarantee health protection and work safety in tunnels. Fresh air is sucked in at the portal of the Sigirino access tunnel down to the operational cavern, and then blown from there via airducts to the rear of the respective heading platforms. There, fresh air is picked up in two stages with two fans each (max. 2 × 16 m^3/s overall performance) and transferred to the face via two airducts of 900 mm in diameter. For this project, for the very first time Rowa constructed a ventilation platform which can be moved independently from the heading platform. It carries the spiral airducts of the blowing ventilation and can be telescoped up to 40 meters, up to approx. 30 meters behind the face.

Therefore, supply of fresh air is optimized and blasting fumes flushed are vacuumed off in front of the heading platform by a suction fan (max. 28 m^3/s overall performance), transferred to the rear of the heading platform and from there by a second fan through an exhaust airduct of 1,800 mm in diameter to the access tunnel. This way blast fumes are not passing through the heading platform and the invert construction site behind the heading. Before blasting, the ventilation platform is retracted back to the crusher and, therefore, out of danger. All fans are remote controlled. The ventilation platform can be extended via friction wheel drive at the touch of a button from the rescue container after the blasting. Tunnel crews are rarely exposed to unhealthy blasting fumes, while the ventilation platform enables to resume work at the face much faster.

Ambient rock temperatures are growing with increasing depth of the tunnel and high performance construction equipment produces additional heat. Ventilation at the Ceneri base tunnel (rock covers of up to 800 meters) was therefore supplemented by cooling systems situated on the heading platforms.

Simultaneous Trailing of Complete Infrastructure and Sufficient Room for Construction Equipment

All infrastructure equipment is located on the 135 m long heading platform. In particular, the most important installations include

- Dedusting unit with 1,200 m^3/min performance,
- Fans for blowing and suction ventilation,
- Air compressor,
- Water pressure booster system,
- Emergency generator (200 kVA performance),
- Transformer for conversion of medium voltage (16,000 V) to 380 V and 220 V respectively,
- High voltage cable drum,
- Storage containers each for electricians and mechanics,
- Lifting devices,
- Airduct cassettes for blowing and suction ventilation,
- Maintenance area for the monorail, container for contractor's supervisors.

Exactly like the ventilation platform, the invert platform and the conveyors, the entire heading platform moves on suspension tracks suspended from the tunnel roof with chains and special adaptors connecting to friction rockbolts of the Bellex 120 Forte type Three stepping units with appropriate hydraulics and controls are placed along the heading platform, while three additional ones are placed alongside the towing conveyor, in such a way that the entire heading platform can be pulled forward at the touch of a button.

In the invert, below the heading platform, tunnel crews have a second working level and free space for working, maneuvering and parking at their disposal. Safety has been increased substantially due to predefined parking and working areas, rationalization of all work flows and generous illumination.

Jaw Crusher, Towing Conveyor and Transfer Conveyor for Muck Removal Logistics

Rowa and the client Consorzio Condotte Cossi have implemented a logistics system which allows for direct muck transport from the face to the portal of the access tunnel at the touch of just a few buttons—and, including the subcontractor for material handling—all the way to the muck disposal site. Muck is hauled after each blast over a distance of 50–70 meters by a wheel loader from the face to the mobile crawler type crusher with primary screen, main screen and metal separator. The 630-meter-long and 800-mm-wide towing conveyor for muck removal is suspended below the heading platform. The climbing area behind the crusher can be lifted before moving the suspension platform. The towing conveyor drops the muck onto a transfer conveyor which is towed every 330 meters, simultaneously with the extension of the continuous conveyor.

Figure 4. CBT—Telescopic ventilation platform for optimal fresh air supply to the face and efficient flushing of blasting fumes (Courtesy of Rowa, www.rowa-ag.ch)

Figure 5. CBT—Increased safety due to predefined parking and working areas, rationalization of all work flows and generous illumination (Courtesy of Rowa, www.rowa-ag.ch)

Figure 6. CBT—Cross conveyor drops the muck of the Eastern tube onto the continuous conveyor in the Western tube, (Courtesy of Rowa, www.rowa-ag.ch)

Figure 7. CBT—Monorail for direct supply to the heading area (Courtesy of Rowa, www.rowa-ag.ch)

Cross conveyors are transporting the excavation muck of the Eastern tubes through cross passages to the continuous conveyors in the Western tubes.

Figure 8. CBT—15-ton-capacity heavy duty crane over the invert construction area (Courtesy of Rowa, www.rowa-ag.ch)

A Monorail Guarantees Direct Supply of the Heading

Supply of the heading with rock support material, shotcrete in mixing containers, wear and tear material as well as explosives is guaranteed by the monorail, which bridges the invert construction site and the heading installation. The monorail moves on an additional track suspended with chains and eye bolts from the roof of the tunnel, connected to friction rockbolts of the Bellex 240 Forte Mono type. Bracings absorb induced acceleration and deceleration forces. Materials are delivered by truck through the completed tunnel invert in dedicated receptacles. The 15-ton-capacity monorail carries all loads directly to the heading with a maximum speed of

1.8 m/s. Interim work stations along the 400 meter-long-rail section can be supplied as well.

Tunnel Invert Is Constructed Simultaneously with the Heading

Simultaneously with the heading operation, a self-moving formwork is used to pour the invert in one cast.

The in situ concrete is supplied by trucks and transported to the site by a 15-ton-capacity heavy duty crane suspended from the invert platform. Heavy repair parts for the construction equipment can also be transported above the invert construction site by the travelling crane. The 66-m-long invert platform can be moved by a total of eight friction drives. The relative movement versus the heading platform amounts to approximately 50 meters, therefore considerably reducing the interdependencies between the two construction sites.

First Lessons Learned

Of the altogether approximately 40 km of single track tubes, cross passages and tunnels which have to be excavated for the Ceneri Base Tunnel (CBT), more than half of it was already completed at the end of 2012. High mechanization of the drill & blast headings was the method of choice in order to economically meet the project deadline in a country with very high labor costs as Switzerland and master expected and encountered the geological conditions.

High mechanization of the drill & blast tunneling initially required a substantial investment from the contractor, especially as four identical Suspended Platform heading systems were ordered from Rowa.

However, the upfront investment continually pays back by increased performance, enhanced safety and better working conditions in all four headings.

THE ROTHANG PASS HIGHWAY TUNNEL (RPHT)

At 3,980 meters above sea level (a.s.l.), the Rohtang Pass belongs to the highest mountain passes open for traffic worldwide and provides the only road connection from the North Indian province Himachal-Pradesh to the border region Ladak. An 8.8-kilometer-long, horseshoe-shaped tunnel at approximately 3,100 meters a.s.l. is currently under construction, which will open the Manali-Leh-Highway for traffic during the entire year, shortening the drive over the narrow and dangerous mountain pass by several hours. The tunnel will offer enough space for an 8-meter-wide two-lane road, as well as a 1-meter-wide footpath on either side. Underneath the main road, a 2.25-meter-high and 3.6-meter-wide escape tunnel is integrated into the tunnel cross section.

Extraordinary challenges of the RPHT project are high altitude, extreme climatic conditions, as well as the geological conditions of the Himalayas. The overburden is 600 meters on average and 1,900 meters maximum. Three fault zones are expected, as well as squeezing rock conditions in certain areas. Additional challenges are the removal of more than 800,000 m^3 of muck, as well as major inrush of water (up to 3 million liters/day in June 2012). For these reasons and due to the very large cross section, the owner and the project designer have excluded a tunneling solution with a shield Tunnel Boring Machine (TBM) and have instead opted for a conventional heading method by drill & blast with flexible adjustment of the heading, as well as the required rock support. Moving large TBM parts on winding roads would have been a serious problem as well.

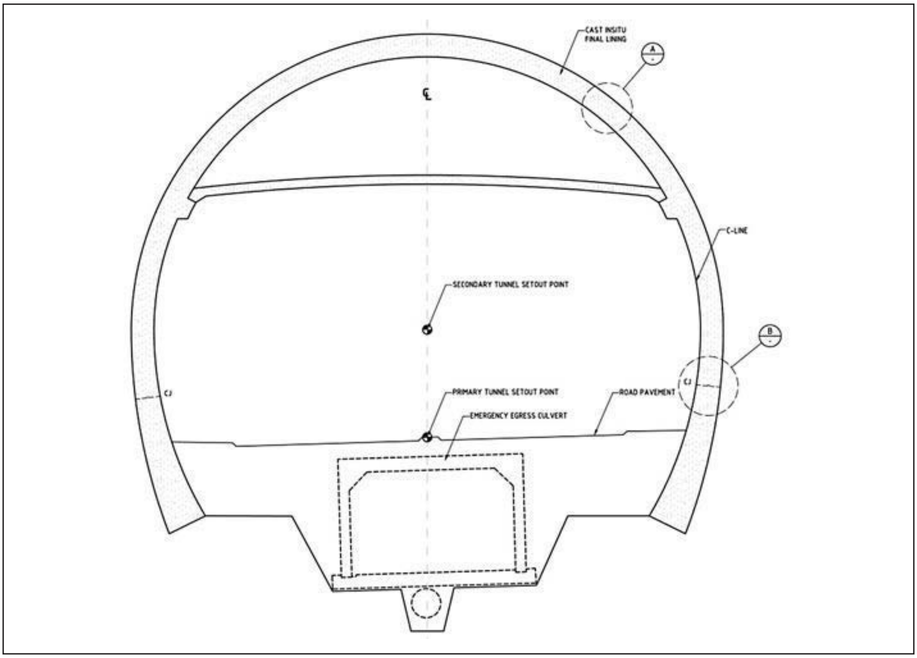

Figure 9. RPHT—Standard cross section of the Rohtang Pass Highway Tunnel (RPHT) (Courtesy of Border Roads Organisation (BRO), www.bro.gov.in)

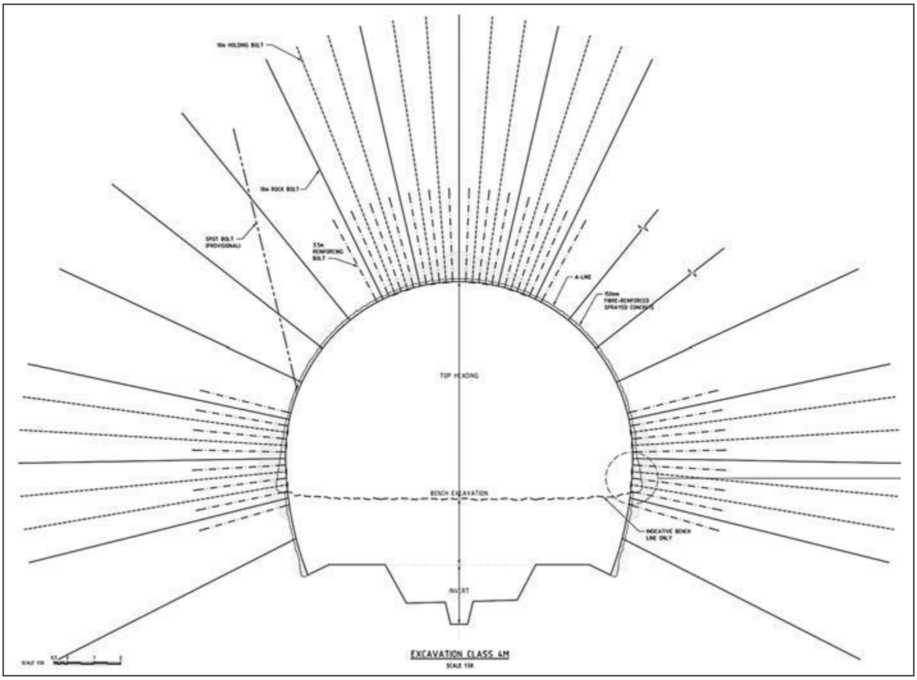

Figure 10. RPHT—Multiple headings and rock support measures for excavation class 4M (Courtesy of Border Roads Organisation (BRO), www.bro.gov.in)

Simultaneous Tunneling in Top Heading, Bench and Invert, Including a Fast Ring Closure

Due to the very large cross section of up to nearly 135 m^2, and due to the geological conditions, multiple headings are the preferred solution. In a first step, the top heading with approximately 83 m^2, and in a second step the bench with roughly 33 m^2, are excavated by drill & blast. Then, the invert follows.

Rock support comes with rock bolts, fiber-reinforced shotcrete and—according to rock class—steel arches. To achieve an early ring closure, excavation and rock support in the bench and invert areas must follow the top heading close by. In addition, the precast elements of the escape tunnel are set. Without special measures and equipment, supply and removal logistics of all work levels would become a very complex challenge and represent the limiting factor for heading performance. For this purpose, the Strabag–Afcons Joint Venture (SAJV) targets a high mechanization of all drill & blast processes and requested Rowa to develop, manufacture, supply, assemble, commission and start up a tailor-made suspended heading platform.

Suspended Heading Platform

All logistics is supported by a highly mechanized, 370-m-long suspended heading platform. It basically consists of

- Heading platform on suspension tracks with equipment and infrastructure,
- Two jaw crushers,
- Three towing conveyors, and
- One continuous conveyor.

Top heading, bench and invert excavation, as well as the construction of the escape tunnel, are independent construction sites for the most part. Their performance may differ from regular heading progress. In order to minimize the interdependencies, the selected platform length allows for a relative difference of 60 meters between work places, without creating logistical bottlenecks. Due to the extraordinary length of the heading platform, four platform sections, each with a pair of stepping devices and corresponding dilatation areas, are controlled according to the sliding-floors principle.

Innovative Ventilation Provides Fast Access to the Face While Delivering a Safe and Healthy Environment

Fresh air is sucked in at the South portal and blown via air ducts with 2,800 mm in diameter to the rear end of the heading platform. There, part of the fresh air is picked up with a fan and transferred to the front of the platform by an air duct of 2,000 mm in diameter. For the efficient flushing of blasting fumes, additional air is blown to the face with a dedicated fan and an air duct of 900 mm in diameter, suspended from the left suspension track. On the right side, next to the blasting protection, blasting fumes are vacuumed by a suction fan as well as the deduster and transferred through two air ducts of 900 mm in diameter to the rear end of the heading platform. During top and bench/invert heading, the tunnel crews and all workers on the rear invert and escape tunnel construction sites are rarely exposed to unhealthy blasting fumes, while the ventilation platform enables to resume work at the face much faster.

Jaw Crushers, Towing Conveyors, and Continuous Conveyor for Logistics of Three Headings

Rowa and the client, Strabag AG–Afcons Joint Venture (SAJV) have implemented a logistics system which allows mucking out from the face all the way to the discharge tower of the muck disposal site at the touch of just a few buttons.

Figure 11. RPHT—Rationalization of work flows and increased safety due to a second level for working and parking below the heading platform (Courtesy of m+m, www.moergeli.com)

Figure 12. RPHT—Continuous conveyor and discharge tower at the muck disposal site (Courtesy of Rowa, www.rowa-ag.ch)

Figure 13. RPHT—Jaw crusher in the top heading area (Courtesy of Rowa, www .rowa-ag.ch)

Figure 14. RPHT—Heading platform, jaw crusher and towing conveyor for bench/ invert excavation (Courtesy of m+m, www.moergeli.com)

Muck is transported after each blast over a distance of 50–70 meters from the face of the top heading to the mobile crawler type crusher by a wheel loader with side tipping bucket.

The muck crushed by the jaw crusher, with a maximum end grain size of 200 mm, is transferred onto the climbing area of the 217-meter-long towing conveyor located on the suspended platform. Simultaneously, a second identical jaw crusher is loaded with material from the bench, and from the invert breakouts, respectively.

All muck is loaded onto a second 110-meter-long movable (with 60 meters travelling distance) towing conveyor suspended from the heading platform. Both towing conveyors drop the muck onto a third, 245-meter-long suspended towing conveyor, which transports the material to the continuous conveyor. Both are installed within the escape tunnel.

Therefore, the entire carriageway is clear of tunneling equipment and all lining works can proceed without interruption. The continuous conveyor is elongated every 200 meters.

Heavy Duty Crane, Material Handling Cranes, and Various Pipelines Ensure Direct Supply of the Headings

A 12-ton-capacity, self-propelling heavy duty crane provides supply of rock support, operational and wear and tear material as well as explosives above the invert construction site to the top, and bench headings. Delivery trucks are driving over the top of the completed escape tunnel. Any workplace on the way to the rear end of the bench crusher can be supplied with construction material. From this point, a 2 × 1.6-ton-capacity travelling crane supplies the top heading. A 1.6-ton cross-travelling, electric-chain hoist is available behind the climbing area of each towing conveyor for material handling. A lengthwise travelling electric-chain hoist takes care of all material handling from the transport vehicles to the heading platform, to the storage/workshop containers on the heading platform.

Shotcrete for rock support is supplied via truck to the rear end of the heading platform, and transferred by two shotcrete pumps to the invert construction site, or to the rear end of the top heading crusher. A switch in the concrete pipes enables an efficient distribution between the two target areas. Additives for shotcrete are supplied via truck and transferred through pipes into the two additives tanks on the heading platform. Fuel for construction machines can also be transferred and stored temporarily in two fuel tanks on the heading platform.

Figure 15. RPHT—Towing conveyor and continuous conveyor in the escape tunnel (Courtesy of Rowa, www.rowa-ag.ch)

Figure 16. RPHT—Shotcrete manipulator with swiveling and telescopic spraying arm, covering a working area of 12 meters in length and a 210° sector (Courtesy of Rowa, www.rowa-ag.ch)

Invert Construction by a Moveable Shotcrete Manipulator, Heavy Duty and Travelling Cranes

A moveable shotcrete manipulator suspended from the heading platform reinforces walls and invert. This innovative solution consists of a frame with integrated lengthwise traction drive, and a swiveling and telescopic spraying arm. The shotcrete manipulator can be used along a 60-meter-long platform section. Its working area covers 12 meters in length and a 210°-sector. The appropriate shotcrete pump with additives tank is located above on the heading platform. The same equipment allows for the provision of invert concrete for the escape tunnel later on. For a remote location as the RPHT project, an especially robust design and manufacturing process and easy-to-operate processes were major factors for the planning.

The precast escape tunnel elements delivered by truck are transferred and placed with the heavy duty crane. For this operation, the crane can travel 1 meter crosswise. After the invert area next to the escape tunnel has been filled up with concrete and compacted in layers, the kickers can be poured with insitu-concrete. A cross- and lengthwise moving travelling crane suspended from the heading platform is used for transferring construction material, as well as repositioning the formwork. The same 2 × 2.8-ton-capacity crane can install precast line segments later on.

Figure 17. RPHT—Placing of the escape tunnel precast elements with a heavy duty crane (Courtesy of Rowa, www.rowa-ag.ch)

Figure 18. RPHT—Travelling crane for construction material, positioning the kicker formwork and placing precast line segments (Courtesy of m+m, www.moergeli.com)

First Lessons Learned

Until the end of 2012, approximately 2.082 kilometers of tunnel has been excavated. Upon completion, the RPHT with its 8.8 kilometers will undoubtedly be the world's longest tunnel higher than 3,000 meters a.s.l. In the past months, all functions of the heading installation supplied by Rowa could be put into operation successfully. Even though the learning phase takes time, the local miners were quickly able to adapt to the increased demands of high mechanization and the entire system has proven to deliver expected results even under such challenging conditions. The chosen construction sequence enables multiple headings and invert lining to be carried out simultaneously and makes the miners' work safer and more productive. At the end, all of these factors offer a fair chance to meet the project's deadlines—if encountered geology will match previously expected conditions.

It is a complex task to choose the best heading method. Each tunnel needs a different approach. With the illustrated suspended heading platform, Rowa is pursuing the continuous development of mechanization in tunneling construction. This leads to a rationalization of work flows and, therefore, to significant productivity increase. And last but not least, all persons involved will profit from enhanced working conditions and increased safety and health.

PROCESS/SAFETY REVIEWS

Underground construction with its multiple hazards in confined space sets the perfect stage for using Process/Safety Reviews (PSR's). Why?

PSR's are basically Safety Audits following the complex procedures and processes in high-risk environments. Conservatively applying true-and-tried safety rules in such environments will definitively kill any innovation and productivity. There is no way a tunnel can be constructed legally by just following strictly all of OSHA rules and regulations. Just think of working under suspended loads continuously … And, as always, if productivity gets hampered, crews begin to cut corners and fight against safety solutions instead of actively using them for their own protection.

This is where PSR's deliver their real value added. However, it takes

- A thorough understanding of best current practices in tunneling,
- Patience to monitor and understand crew activities and interaction in real-time,
- Intercultural competencies to communicate with the workforce in situ adequately,
- A productivity oriented approach to help stabilizing established processes applied underground,
- Innovation to develop solution packages producing a level of safety comparable to OSHA requirements that local Authorities Having Jurisdiction (AHJ's) can authorize,
- Readiness by contractor and equipment suppliers to strive for optimization of the project.

PSR's (have to) achieve comparable results in safety & health, just the other way round. Instead of directly enforcing safety (following the well know and best intended "Safety first" commitment), PSR's start to enhance productivity first by optimizing (and therefore stabilizing) the production processes. However, this results in safe work practices accepted by the crews underground and acceptable by all involved AHJ's. Therefore, safety is not the beginning, but the end result of a process.

A stable process is always a productive process. And a productive process always becomes a safe process. We have demonstrated this so many times.

ACKNOWLEDGMENTS

The authors thank and acknowledge the competent support by:

Figure 19. AlpTransit Gotthard Ltd (ATG) (Courtesy of ATG, http://www.alptransit.ch/en/home.html)

Figure 20. Border Roads Organisation (BRO) (Courtesy of BRO, http://www.bro.gov.in)

Figure 21. Consorzio Condotte Cossi (CCC) (Courtesy of CCC, www.condotte.com)

Figure 22. SRABAG—AFCONS Joint Venture (SAJV) (Courtesy of SAJV)

Figure 23. Rowa Tunnelling Logistics AG (Rowa) (Courtesy of Rowa, www.rowa-ag.ch)

Figure 24. moergeli + moergeli consulting engineering (m+m) (Courtesy of m+m, www.moergeli.com)

Figures 25–27. Every day moving into places where no human being has ever been before (Courtesy of m+m, www.moergeli.com)

However, the biggest thanks goes to all crews on site, safely managing the unforeseeable (= Risk Management in Action) in high risk environments as their daily routine.

Therefore, always … "One small step for (a) man, one giant leap for mankind…" (Neil Armstrong, 2:56 UTC on July 21, 1969—First person to walk on the Moon).

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[All latest website checks by m+m/am, January 9, 2013]

PART 15

Precast Tunnel Linings

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FIBER-REINFORCED GEOPOLYMER CONCRETE— AN INNOVATIVE MATERIAL FOR TUNNEL SEGMENTS

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Malcolm Chappell ■ Halcrow Pacific Pty Ltd.

ABSTRACT

Fiber reinforced geopolymer concrete (FRGC) has no Portland cement or steel reinforcement, but instead used synthetic fiber reinforcement and geopolymer binder.

This paper discusses the development of FRGC during a project part-funded by the Victorian Government and a consortium of five organisations. The work involves laboratory and field trials and the production of prototype tunnel segments which have been subject to fire testing.

The results indicate that FRGC has the potential to produce precast concrete products with increased durability and substantially reduced carbon emissions compared to the Portland cement based concrete with steel reinforcement.

INTRODUCTION

This paper examines the combined use of two emerging technologies in the production of precast tunnel segments: geopolymer binder and fiber reinforcement.

Precast concrete is the predominant material for segmental tunnel linings, with a typical 6m internal diameter (ID) tunnel utilising approximately 4,500 cubic metres of concrete per kilometre. The major contribution of concrete to underground infrastructure is accompanied by significant embodied greenhouse gas emissions. Concrete in construction reportedly generating more than 5% of worldwide carbon dioxide (CO_2) emissions, over three-quarters deriving from the Portland cement binder. Approximately half the CO_2 emissions from Portland cement are associated with energy used in the heating and grinding processes. The remaining emissions derive from the chemical de-carbonation of the limestone. The cement industry has made considerable improvements in energy efficiency and use of alternative fuel sources. However, if worldwide emissions targets are to be met, some radical changes are required to further reduce the CO_2 emissions derived from the use of concrete. Geopolymer binder contains no Portland cement but instead uses industrial by-products such as fly ash and slag which are activated by an alkaline agent. The hardened binder includes alumino silicates similar to those produced by Portland cement based binders, but no calcium compounds. There are currently no international standards for the use of geopolymer in construction, but guidance was recently published by the Concrete Institute of Australia (CIA 2011).

Steel fibers have become the preferred reinforcement for segmental linings constructed in the last 10 years. Halcrow has been at the forefront of this development (King 2005, Angerer and Chappell 2008 and Harding and Chappell 2012). In addition to increased productivity by the elimination of steel fixing, steel fiber reinforced concrete (SFRC) has enhanced durability due to the absence of galvanic and stray current corrosion (ACI 1996) and reduced impact damage during handling. There has also been development in the use of non-corrodible synthetic macrofibers as an alternative to steel fibers, primarily for sprayed concrete linings in mines, but also for potential use

Table 1. Key issues to address in the adoption of FRGC technology

Geopolymer	Synthetic Fibres
Absence of structural design parameters Practical constraints (e.g., controlling workability, setting time and strength development) Uncertainty over long-term performance (e.g., permeability and diffusion properties and acid resistance)	Absence of structural design parameters Uncertainty over long-term performance (e.g., creep) Urgent need to identify appropriate test methods and limits to control properties but avoid unacceptably high rates of non-compliance

in segmental linings. Fiber reinforced segments have a lower resistance to bending moments than conventionally reinforced concrete and the lower elastic modulus of synthetic fibers influences their ability to control cracking and deflection under load. These effects need to be considered in the design and handling of segments. Guidance on the use of steel and synthetic fibers was published by the UK, Concrete Society in 2007 and a design method based on work by Rilem is included in New Zealand Standard NZS 3101.

The combination of geopolymer binder and synthetic fiber reinforcement presents the opportunity to produce concrete with enhanced durability and reduced environmental impact. However, it necessitates the design, testing and construction procedures to be modified compared to bar reinforced Portland cement based concrete. This paper summarises the findings of a 3 year Australian project to develop Fibre Reinforced Geopolymer Concrete (FRGC) precast products for underground infrastructure funded by Victoria's Science Agenda Investment Fund and a consortium of five organisations (Halcrow Group, Elasto Plastic Concrete, Zeobond, University of Melbourne and Humes).

METHODOLOGY

The introduction of new products in the construction industry is controlled by the understandably conservative nature of the engineering profession and the need to meet existing industry specifications. Some of the issues relating to adoption of geopolymer and fibre technology are indicated in Table 1. In order to address these issues, the project pre-empted the normal approvals process by testing the FRGC products against the requirements of a typical performance specification. In addition, the project developed guidance on structural and durability design and produced prototype products.

Typical performance requirements for fiber reinforced tunnel segments were adopted as shown in Table 2. The tests include American and Australian strength and durability tests, such as residual flexural strength and apparent volume of permeable voids (AVPV), as well as European tests for water penetration and chloride migration. Specimens were also subject to exposure tests to acid, chloride and sulfate solutions for 2 years.

Following laboratory testing, field trials were undertaken at the precast plant to allow prototype segments to be produced as well, as large beams for bending tests to validate the design method. Standard and accelerated curing were used to determine the effect on the early strength gain for demoulding. Prototype segments were subject to simulated hydrocarbon fire to assess spalling resistance. ASTM C1550 round panels were tested for toughness and cracked panels were centrally loaded in a rig to monitor deflection due to creep.

Table 2. Summary of performance specification requirements

Parameter	Requirement
Strength	
Minimum 28-day cylinder strength (MPa)	50
Minimum cylinder strength for demoulding (MPa)	10
Minimum 28-day tensile splitting strength (MPa)	4.2
Minimum 28-day flexural strength (MPa)	4.6
Minimum 28-day equivalent post-crack residual flexural strength $F_{e3.0}$ (MPa)	3.2
Durability	
Maximum AVPV rodded (%)	13
Maximum 56-day chloride migration coefficient (m^2/s) Maximum 91-day chloride migration coefficient (m^2/s)	4×10^{-12} 2×10^{-12}
Maximum sorptivity (mm)	8
Maximum 56-day drying shrinkage (microstrain)	600

DISCUSSION

Laboratory Trials

The initial laboratory trials assessed the workability characteristics of FRGC mixes using different fibre types, and doses of synthetic fibres from 8–12 kg/m^3. A Portland cement based concrete containing 8 kg/m^3 of synthetic fibres and geopolymer concrete with 40 kg/m^3 steel fibres provided two control mixes. The Portland cement control mix was based on an existing production mix, with 20% fly ash in the binder and a water/binder ratio of less than 0.4.

The synthetic fibres are manufactured from polyolefin and are 60mm long and 0.5–1 mm in diameter with an embossed profile. The steel fibres are formed from cold drawn high tensile carbon steel and are 60 mm long and 0.75 mm in diameter with hooked ends.

Based on the initial trials a geopolymer mix with 8 kg/m^3 of synthetic fibre was selected for further development. This mix gave a 100 mm target slump 60 minutes after mixing (allowing for permissible tolerances). The main laboratory mixes were 0.35 m^3 in size to allow a large number of specimens to be produced. The key findings from the laboratory trials are discussed below.

Strength

The strength results are summarised in Table 3.

It can be observed that the compressive and tensile splitting strengths of the geopolymer concrete are lower than those of the Portland cement based control and the typical specification requirements. However, flexural strength is of primary importance in the performance of tunnel segments, and the flexural strength and equivalent post-crack residual flexural strength value at 3 mm deflection of the geopolymer with synthetic fibres slightly exceeds those of both the Portland cement based control with synthetic fibres and the geopolymer mix with steel fibres. This is shown in Figure 1.

Conventional concrete mixes with steel fibres have shown a tendency to become brittle as the concrete strength increases due to fibre rupture rather than gradual pull-out. The good equivalent post-crack residual flexural strength values of the geopolymer mix with synthetic fibres is encouraging, and this value would not be expected to be reduced by long-term strength gain of the concrete in the same way as steel fibres because of the lower elastic modulus of synthetic fibres.

Table 3. Summary of strength results for laboratory trials

Parameter	Conventional Concrete with Synthetic Fibres	Geopolymer Concrete with Steel Fibres	Geopolymer Concrete with Synthetic Fibres
28-day cylinder strength (MPa)	52.5	46.0	49.5
1-day cylinder strength for demoulding (MPa)*	25.0	24.0	25.0
28-day tensile splitting strength (MPa)*	4.8	4.0	3.4
28-day flexural strength (MPa)*	5.5	6.4	7.4
28-day equivalent post-crack residual flexural strength $F_{e3.0}$ (MPa)*	3.7	3.8	3.9

* denotes accelerated curing

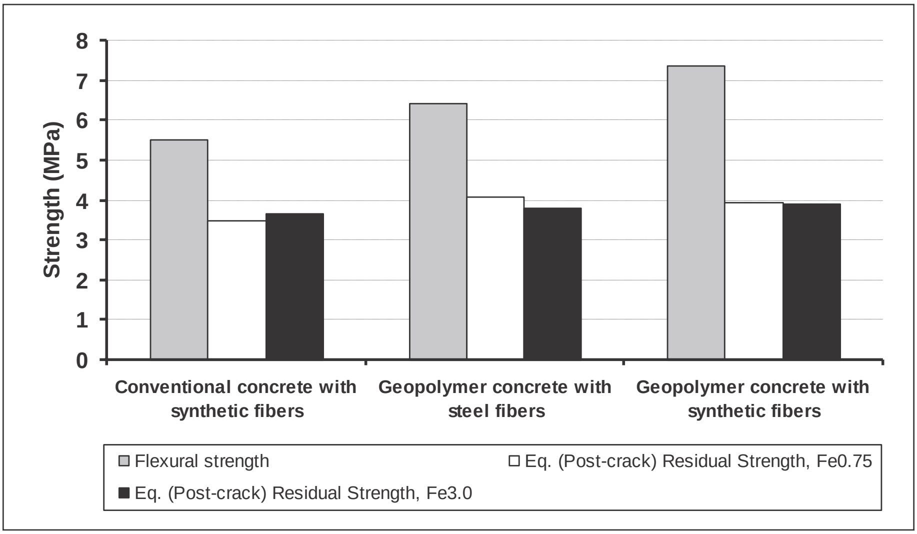

Figure 1. Flexural strength results

Durability

The durability test results are summarised in Table 4.

The Apparent Volume of Permeable Voids of the geopolymer mixes is higher than that of the Portland cement based control, and also exceeds the specified limit. In contrast, the chloride migration, sorptivity and dying shrinkage of the geopolymer mixes are better than those of the Portland cement based control.

Exposure of specimens to acid and sulfate solutions and periodic abrasion resulted in similar deterioration to FRGC as the control mix containing Portland cement and 20% fly ash. In contrast, the chloride ingress into FRGC is much lower than that of the control and this is consistent with the chloride migration test results.

Geopolymer concrete lacks a conventional capillary pore structure and this may mean that parameters which are heavily influenced by capillary porosity and capillary transport of moisture, such as sorptivity and diffusion, could be beneficially influenced. However, because there is no hydration in geopolymer concrete it is important that the water content is carefully controlled to minimise overall porosity to benefit AVPV and strength.

Field Trials

Field trials of up to 2.5 m^3 size were undertaken at a precast plant. Tunnel segment moulds were already available at this plant from a recently completed 2.4 m ID tunnel with a 200 mm thick lining. The objective of the field trials was to produce prototype FRGC segments, as well as larger specimens for further testing, including large beams for bending tests. A conventional concrete control mix with steel fibres was also included.

The field trials used a FRGC with 8 kg/m^3 of synthetic fibres and Portland cement based concrete with 40 kg/m^3 of steel fibres (with and without the addition of 1 kg/m^3 of synthetic microfibers for improved fire spalling resistance).

Four rectangular bolted segments (each approximately 0.4 m^3 and 0.8 tonnes in weight) and four smaller tapered key segments (each approximately 0.1 m^3 and 0.2 tonnes in weight) were produced from the FRGC mix. Figure 2 shows the typical condition of the demoulded segments and a sawn cross-section. The surface of the segments displayed acceptable finish and the cross-section indicated good uniformity and compaction. However, care has to be taken to evenly disperse the fibres during mixing and to appropriately cure the concrete.

Strength

The compressive strength development of the FRGC mix is shown in Figure 3. Early strength development of the FRGC was very good and segments were successfully stripped to allow a single casting cycle every 24 hours at an ambient temperature

Table 4. Summary of durability test results for laboratory trials

Parameter	Conventional Concrete with Synthetic Fibres	Geopolymer Concrete with Steel Fibres	Geopolymer Concrete with Synthetic Fibres
AVPV rodded (%)	13	17	14
56-day chloride migration coefficient (m^2/s)	3.5×10^{-12}	Not tested	1.1×10^{-12}
91-day chloride migration coefficient (m^2/s)	1.9×10^{-12}	Not tested	0.9×10^{-12}
Sorptivity (mm)	9.0	6.1	6.2
56-day drying shrinkage (microstrain)	530	240	400

Figure 2. Prototype tunnel segments and sawn cross-section

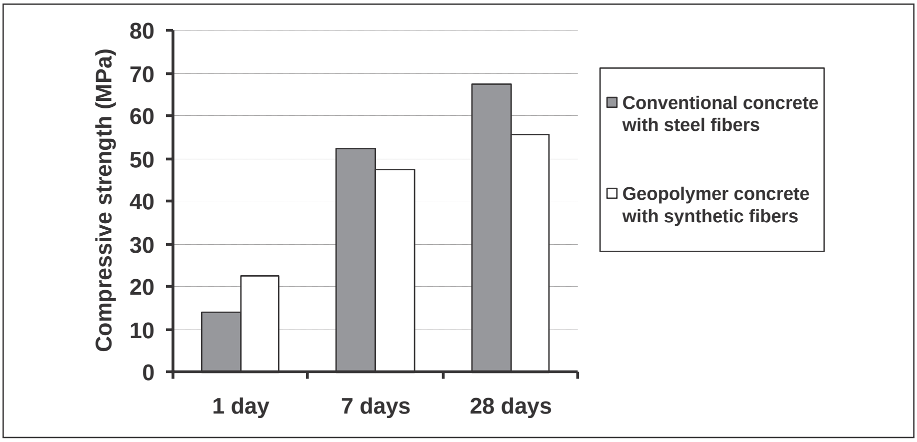

Figure 3. Compressive strength development

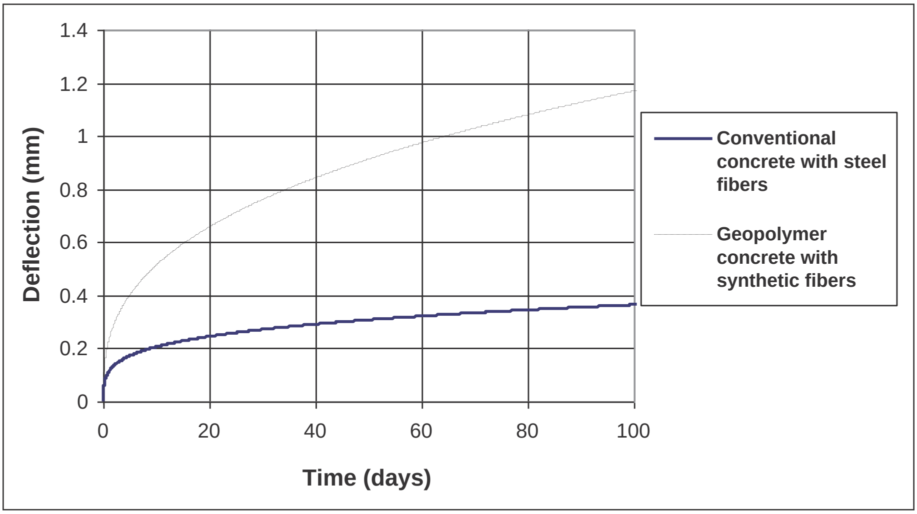

Figure 4. Creep data

at casting of 22°C without heat curing. Compressive strength and tensile splitting at 28 days in the FRGC mix is approximately 20% lower than conventional SFRC control mix. Flexural strength and residual flexural strength in the FRGC mix are 5–10% lower than the SFRC control, but met the performance requirements.

Creep

ASTM C1550 round panels were loaded to induce cracking and then a load representing 20–30% of the static capacity was applied over 100 days the central deflection monitored (see Figure 4).

The creep deflection of the FRGC mix is approximately a factor of three greater than the conventional SFRC control mix and creep coefficients are approximately 1.5 for the control mix and 4.3 for the FRGC mix. This indicates that creep deformation needs to be taken into consideration in the design when there is a potential for cracked FRGC segments to be subject to sustained bending.

Beam Tests

Notched beams of different dimensions (100×200×700 mm, 250×200×1750 mm and 400×200×2800 mm) were cast from the FRGC mix and subject to bending at approximately 3 months age using a modification of the three -point method in TC162-TDF (Rilem 2003). Figure 5 shows the testing arrangement for the largest of the beams. The load and crack mouth opening deflections were recorded and the results were analysed to verify the size factor in the NZS 3101 design method. This factor takes into consideration boundary effects in thinner sections, including a predominant fibre orientation. The beam tests mimic research undertaken in the Brite EuRam Project "Test and Design Methods for Steel Fibre Reinforced Concrete."

The tests confirm that there is a size factor effect for FRGC similar to that for SFRC, although its magnitude is influenced by the fibre type and should be established for the intended fibre in order to modify the NZS 3101 design method.

Fire Tests

Four segments were subject to a simulated hydrocarbon fire involving exposure to furnace temperatures of up to 1100°C over approximately 1 hour. Table 5 summarises the condition of the intrados exposed to fire testing.

The steel fibre mix without synthetic microfibers displayed extensive explosive spalling, whereas the geopolymer concrete with macrosynthetic fibers had little or no spalling indicating acceptable fire performance. However, the potential impact of a fire event upon the performance of the synthetic macrofibres should be assessed as part of the design process.

Figure 5. Bending test on FRGC beams

Table 5. Summary of fire tests on segments

Segment	Performance
Conventional SFRC	Explosive spalling to approximately 40% of exposed area
Conventional SFRC and synthetic microfibers	No spalling
FRGC (6 months old)	Non-explosive spalling to approximately 10% of exposed area
FRGC (8 months old)	No spalling

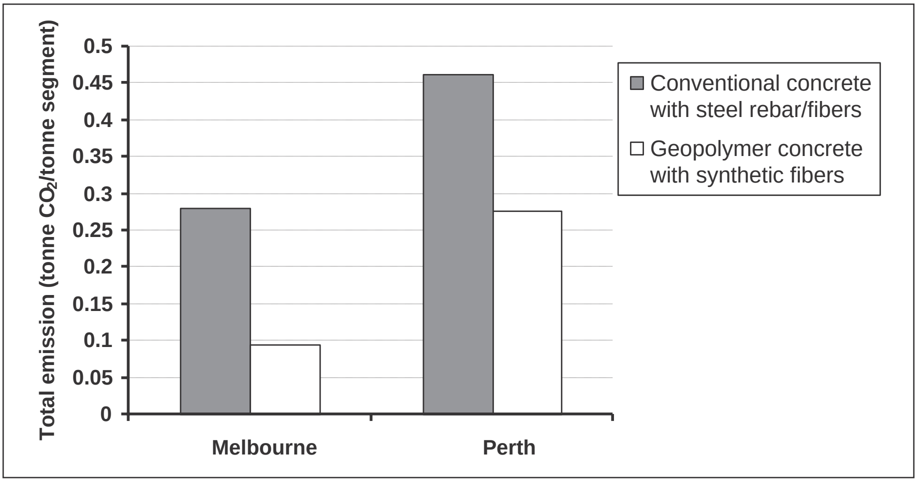

Figure 6. Carbon emissions associated with FRGC and conventional concrete segments delivered to sites in Melbourne and Perth

CARBON EMISSIONS

Reducing the carbon emissions of concrete is a key driver for the project. In order to assess the impact of using FRGC compared to conventional concrete, two scenarios were considered:

a. Casting segments at Echuca, Victoria for transport by road 220 km to a project site in Melbourne, Victoria; and

b. Casting segments for transport at Echuca, Victoria by road 3,310 km to a project site in Perth, Western Australia.

The first case represents a realistic supply situation for precast segments, whereas the second case is intended to represent a maximum transport distance in order to assess the influence of haulage on carbon emissions.

The carbon emissions were calculated using published values for converting energy and fuel to CO_2. The calculations assumed that the strength and durability performance of the FRGC and conventional concrete mixes are similar. The FRGC concrete has 8 kg/m^3 of synthetic fibres and conventional concrete assumes 40 kg/m^3 of steel reinforcing bar or steel fibres.

The calculations allow for the embodied carbon in the constituent materials (including obtaining and processing the raw materials), transportation to the precast plant, production of the segments and their transportation to the project site. The calculations do not assess the effects of carbonation, or the carbon emission associated with demolition and reuse of tunnel segments.

Figure 6 shows the comparison between FRGC and conventional concrete. The CO_2 emissions of segments using FRGC are 34% and 60% of the values for conventional concrete segments delivered to sites in Melbourne and Perth respectively, representing a reduction of up to approximately 70% in emissions. The CO_2 emissions for FRGC segments transported to Perth are slightly less than those associated with conventional concrete segments delivered to Melbourne, indicating that binder and reinforcement type predominate over transportation.

Table 6. Summary of FRGC characteristics

Characteristic	**FRGC Comparison to Conventional SFRC**
Compressive strength	Equal or better at early ages 20% lower at later ages (50MPa possible with care)
Tensile splitting strength	20% lower consistent with compressive strength
Flexural strength	Slightly lower
Residual flexural strength	Slightly lower
Drying shrinkage	Reduced
Creep	Factor of three times higher for cracked FRGC panels
Fire resistance	Improved resistance to spalling compared to SFRC without synthetic macrofibers. Effect of fire event on synthetic macrofibers to be assessed in design.
Durability: AVPV Sorptivity Chloride migration Acid resistance Sulfate resistance	 Slightly higher Reduced Significant reduction in chloride ingress Similar Similar
Carbon emissions	70% reduction allowing for practical transport distances
Design code	NZS 3101 but with modified size factor and allowance for increased creep
Specification	Similar to SFRC but no need for chloride migration testing if synthetic macrofibres used Additional care over control of water content and curing
Cost	Cost up to 10% lower than conventional SFRC

SUMMARY AND FURTHER WORK

The findings of the project are summarised in Table 6. An estimate of cost of FRGC compared to conventional concrete has been included based on the experience of the consortium partners in Victoria. The cost will be influenced by the availability and cost of suitable industrial by-products and alkali activators.

The next step in the development of FRGC following the successful laboratory and field trials is to undertake project trials. It is proposed that precast FRGC products using synthetic or steel fibres are used in non-critical applications, such as a back shunt tunnel or precast headwall. The products would be designed to NZS 3101 and subject to load testing as necessary. The products would be inspected and tested periodically and/or include sensors, such as corrosion ladders, to allow long-term monitoring of performance.

CONCLUSIONS

A 3 year study has been undertaken to develop concrete which has no Portland cement or steel reinforcement, but instead used synthetic fiber reinforcement and geopolymer binder.

Laboratory and field trials have been undertaken to assess the strength and durability of FRGC against a typical performance specification for fibre reinforced concrete segments and control mixes using Portland cement and 20% fly ash. Prototype segments have also been produced.

The work indicates that acceptable FRGC mixes and segments can be produced. Flexural and residual flexural strength are slightly higher and lower than control mixes using synthetic and steel fibers respectively and met the specified requirements. Durability and fire spalling performance are significantly enhanced.

Compressive strength and tensile splitting strength are lower than the control and care is needed over control of water content and curing. Creep of cracked FRGC ASTM C1550 round panels is a factor of three higher than the control and this should be allowed for in the design.

The embodied carbon dioxide emissions associated with synthetic fibre reinforced FRGC segments are approximately 70% lower than conventional concrete using steel reinforcement for a realistic transport distance.

The next stage in development of FRGC are project trials using precast FRGC products in non-critical applications to allow their long-term performance to be monitored.

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DESIGN, CONSTRUCTION, AND PROCUREMENT CONSIDERATIONS FOR TUNNELING SEGMENTS OF MTA BALTIMORE RED LINE

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ABSTRACT

The Maryland Transit Administration (MTA) Baltimore Red Line Light Rail Transit (LRT) Project, a proposed 14.1-mile twin-track east-west transit link across Downtown Baltimore between the Federal campus at the Center for Medicare and Medicaid Services and the Bayview Medical Center campus of Johns Hopkins University, features two tunnel sections totaling about four miles. During preliminary engineering, the Program Management Consultant (PMC) and General Engineering Consultant (GEC) teams explored alternative contracting methods including Design/Build (DB), Design/Bid/Build (DBB) (baseline), Design/Build/Operate/Maintain, and Design/Build/Finance/Operate/Maintain under several financing models including Public-Private Partnerships (P3) to deliver the Project. Challenges and risks presented and evaluated included contract packaging interfaces, difficult ground conditions (residual and transition soils to mixed-face and hard rock), overlying urban development including sensitive structures, site constraints, and mining techniques.

PROJECT OVERVIEW

The Baltimore Red Line presently under design is a 14.1 mile, east-west LRT corridor that will connect west Baltimore County and east Baltimore City. It will pass through downtown Baltimore and include connections to MTA's existing transit system—Urban and Express Bus, Metro Subway, Central Light Rail Line, and Maryland Area Regional Commuter train service (MARC rail). The corridor will also provide access to the Centers for Medicare and Medicaid Services and Social Security Administration (large office complexes), University of Maryland Medical Center, the Veterans Administration Hospital, the Johns Hopkins Bayview Medical Center and associated campus of National Institute of Health, and the local attractions—Baltimore's Convention Center, Gwynns Falls trail, Inner Harbor and Fell's Point.

The alignment (Figure 1) is composed of 8.7 miles of surface rail, 4.7 miles of tunnels (1.0 mile-long portal-to-portal Cooks Lane Tunnel [CLT] and 3.2 miles-long portal-to-portal Downtown Tunnel [DTT]), 0.7 miles of aerial structures, 14 surface stations, and five underground stations. The revenue service is projected to start in March 2021.

DESCRIPTION OF TUNNEL SEGMENTS

As stated, the MTA Baltimore Red Line LRT alignment features two tunnel segments: the CLT and the DTT.

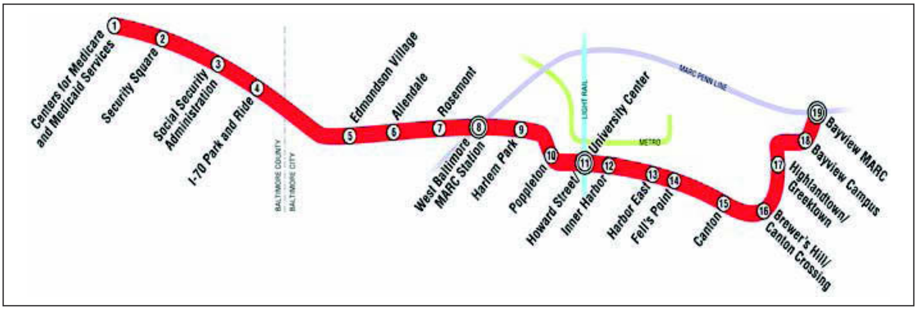

Figure 1. Baltimore Red Line light rail alignment

The 1.0 mile-long CLT Segment extends from a west portal located at the highway ramp for I-70 southeast to the east portal, which is the intersection of Edmonson Avenue (US Route 40) and Glen Allen Drive. This tunnel segment includes approximately 4,800 feet of twin bored tunnels, 500 feet of cut-and-cover tunnel, seven cross passages and approximately 1,800 feet of retained cut. The existing I-70 EB/Park and Ride Ramp is to be abandoned under the current design.

The DTT segment stretches approximately 18,000 feet from a west portal located at the intersection of Mulberry Street and North Fremont Avenue, just south of US Route 40, to the east portal, which is the intersection of Boston Street and South Montford Avenue/Hudson Street. The DTT segment consists of twin mined running tunnels, two cut-and-cover portal sections, five underground stations, and associated mined cross passages, and a pedestrian tunnel connecting the Red Line LRT with the Baltimore Metro Line. As the alignment approaches the Howard Street/University Center Station, it will cross below the existing CSX railroad tunnel. This mined section features several horizontal curves, and will reduce surface construction in Baltimore's narrow and congested downtown streets.

The feasibility of a single-bore large diameter tunnel versus the twin-tunnel concept is currently being re-evaluated as part of preliminary engineering.

GROUND CONDITIONS

CLT Segment

The 4,800-feet long mined portion of the CLT Segment lies below the ground water level, and anticipated ground conditions include high strength, highly abrasive rock (Ground Class I, II, and III), Transition Group (Ground Class IV and V) underlain by rock (forming mixed-face), and fault zones. Specifically, the Transition Group consists of completely weathered and highly permeable material, and is expected at the western end (≈800 feet) and eastern end (≈400 feet), respectively. Between these two ends of Transition Group, about 67%, or 3,200 feet, of the tunnel is located in generally competent rock. Three distinct fault zones were identified during the subsurface exploration.

In general, the distribution of ground types as a percentage of the total tunnel volume is as follows:

- Rock—71%
- Transition Group—24%
- Other materials (fill and residual soil)—5%

Mixed-face conditions (Transition Group over rock) are expected near tunnel ends and aggregate about 30% of the alignment.

Generally, the groundwater table along the alignment lies within 30 feet below ground surface. The rock permeability is generally controlled by fracture flow; rock permeability per se is low.

DTT Segment

The DTT Segment's tunnel excavation will also be located below the groundwater table and pass through highly variable ground conditions anticipated to range from hard rock, highly weathered to completely decomposed rock, and saturated soft ground (granular and/or cohesive soils of varying degrees of density and consistency).

The general anticipated ground conditions are shown in the following breakdown of ground-type as a percentage of total tunnel volume:

- Rock—35%
- Cretaceous Group Soils—34%
- Transition Group—25%
- Other materials (Fill, residual soils, post-Cretaceous deposits)—6%.

In general, rock is predominantly located within the westerly tunnel reaches, the Transition Group is concentrated in the central to easterly reaches, and the Cretaceous Group soils towards the easterly portion of the tunnel.

The groundwater table generally lies near top of transition zone west of Martin Luther King, Jr. Boulevard and within about six feet of mean sea level in the overburden east of Howard Street. Low permeability is generally expected in the intact rock, with the highest permeability anticipated in the fractured rock associated with fault or sheared zones. Similarly, typically low to moderate permeability in the Transition Group is expected to increase substantially at local open relict fractures. Hydraulic connectivity between the Inner Harbor and neighboring soils is inferred; hence, brackish groundwater and a tidally influenced groundwater behavior are anticipated during construction excavation.

Rock

The various metamorphic and igneous rock types present include amphibolites, gneiss, schist, marble, pegmatite, and vein quartz. The predominant Ground Class III rock mass may be characterized as "Moderately Blocky to Very Blocky and Seamy." Several faults and shear zones may exist. Rock mass permeability is expected to be less than 10^{-4} cm/sec, except at discontinuities or fracture zones where permeability is expected to be higher and where localized inflows would occur.

Cretaceous Group

The Cretaceous Group is primarily granular in nature with high permeability sands/gravels (>10^{-2} cm/sec permeability). Hence, under the existing high groundwater conditions positive face stability and groundwater control will be critical.

Transition Group

Within the Transition Group (Ground Classes IV, V), material may be characterized as completely decomposed (weathered) whereby the host rock mass has been altered. Descriptions of this altered material include: an extremely weak, completely decomposed rock; weakly bonded (cemented) particulate material (soft/ decomposed); a gradational material between the lesser weathered (Grade III) state and fully weathered (approaching Grade VI, Residual); and a porous moisture-sensitive material susceptible to slaking and disaggregation when exposed (unconstrained). The remaining relict structure exhibits the following:

- Reduced shear strength characteristics due to occurrence of clay infilling.
- Generation of instability due to unfavorably oriented structure and potential block release surfaces. For example, sub-vertical structure in curved haunch profiles as might be occurring in a Sequential Excavation Method (SEM) excavation.
- Heterogeneity (variability) that may be difficult to sample in a representative manner.
- Occurrence of weaknesses or zones that control overall behavior (key blocks).

Because of a short stand-up time, immediate ground support and continuous positive tunnel face support is essential for excavation in Transition Group material. These materials are unstable under the groundwater table.

Mixed Face and Mixed Ground Conditions

Mixed face conditions are defined as Rock overlain by Transition Group; Rock overlain by Transition Group; and soils derived from any combination of Cretaceous, Post-Cretaceous and/or Residual soils. Mixed face conditions are predominately in the western and central tunnel reaches. These conditions are not continuous; they total approximately 5,900 linear feet.

Soft ground conditions are generally encountered to some extent within all tunnel reaches. Some soft ground conditions occur as mixed ground with the tunnel face exhibiting contrasting soil properties such as:

- Transition Group/Cretaceous,
- Residual Soil/Cretaceous,
- Transition Group/Residual Soils/Cretaceous, or
- Granular Cretaceous/Cohesive Cretaceous.

Brief Ground Class Description

Ground Class I (Rock): Massive to moderately jointed; unweathered to slightly weathered rock and fracture spacing exceeds six feet; rough or irregular to smooth and planar joint surfaces; unaltered to slightly altered fracture surfaces, with non-softening mineral coatings; and no obvious planar weakness zones with alteration products.

Ground Class II (Rock): Moderately blocky; unweathered to moderately weathered rock, and fracture spacing two to six feet; or one set of slickensided, polished fracture surfaces present within the excavation horizon, or one planar weakness zone containing clay or disintegrated rock, with a thickness of disintegrated rock or alteration products less than six inches.

Ground Class III (Rock): Moderately blocky to very blocky and seamy; slightly to moderately weathered rock, and fracture spacing less than two feet; or multiple sets of slickensided, polished fracture surfaces, or multiple planar weakness zones with fillings of disintegrated rock or alteration products less than six inches thick, or a single planar weakness zone with infilling greater than six inches, or weak rock type (e.g., talc schist, biotite schist, or chlorite schist).

Ground Class IV (Transition Group): Highly weathered rock; recovered with rock sampling equipment; more than half of the rock material matrix weathered to a soil; fresh or discolored rock present as corestones; some open grain boundaries, but individual grains intact; does not disintegrate when agitated in water.

Ground Class V (Transition Group): Mix of sand, silt and clay; medium dense to very dense; stiff to hard; completely weathered rock; recovered with soil sampling equipment; all rock material decomposed or disintegrated to soil; all feldspars and biotite decomposed to clay; disintegrates when agitated in water.

Ground Class VI (Residual Soil): Mix of sands and fines; medium dense to very dense; stiff to hard; all rock material converted to soil; no corestones.

DESIGN AND CONSTRUCTION ISSUES

Cooks Lane Tunnel

There are a number of critical design and construction issues that have been identified and studied:

- Alignment with a narrow right-of-way (ROW) with a 90° small radius turn,
- Residential neighborhood,
- Relatively deep alignment due to terrain and LRT operational grades,
- Rock with Transition Group (completely to highly weathered) material creating appreciable lengths of mixed face conditions,
- High groundwater table,
- Transition Group material anticipated as being highly unstable during excavation below the groundwater table,
- Narrow pillar if twin bored tunnels are utilized, and
- Potential asbestos type minerals in the rock.

The CLT is confined with a narrow ROW to avoid passing under residential properties. Tunnel width has been designed to fit within a 55 feet-wide corridor. The alignment has a right angle turn with a relatively short radius (see Figure 2). The radius is currently held at 650 feet for LRT design speed considerations. This radius would accommodate a typical subway size Tunnel Boring Machine (TBM) and easy suit any SEM excavation methodology. However, with this radius any tunnel will pass under a residence (see Figure 3) in mixed-face ground conditions. It is to be noted that a larger single bore TBM would increase the impacts to adjacent structures because of a need for a larger radius.

The western portal is located in an area with numerous utilities including several appreciably sized water mains and storm drains including an 84-inch drainage conduit, the latter potentially only 10 feet above the tunnel crown. The tunnel profile at both construction portals is located in zones of mixed-face ground conditions. Locally these temporary portals will become permanent cut & cover and open retained cut sections. The finish grades at these latter areas are a steep 6%.

Should twin bored tunnels be utilized, the pillar between the tunnels is slightly less than half a tunnel diameter. Under this geometric scenario, there has been concern on the tunnel/pillar stability. It is currently anticipated that the tunnel would be supported by precast concrete segmental rings. Numerical analyses have been conducted on the issue. Findings indicate that the impact on the tunnel lining rings is not significant and the pillar should be stable.

Downtown Tunnel

There are a number of critical design and construction issues that have been identified and studied:

- Alignment through historical colonial Baltimore City with sensitive structures, some of which are in very fragile condition;
- Alignment with very narrow ROWs and several curves;
- Dense business and residential areas;
- Alignment through former waterfront areas and reclaimed land, current waterfront close-by;

Figure 2. Schematic plan alignment for Cooks Lane tunnel

Figure 3. Alignment passes under residence in mixed-face conditions

- Eastern portion generally consisting of permeable Cretaceous sands/gravels with some overlying compressible peat, high groundwater table with brackish groundwater quality;
- Western portion generally consisting of rock with Transition Group material creating major lengths of mixed face conditions, high groundwater table;
- Anticipated buried/abandoned waterfront structures;
- Interfacing with five cut & cover stations with very narrow widths;
- Severely limited areas for construction staging; and
- Crossing under the operating 100-year-old Howard St. CSX rail tunnel in mixed-face ground conditions.

The DTT is confined within a narrow ROW following, in some cases, colonial streets (see Figure 4). A significant portion of the eastern section traverses former waterfront areas, which have been reclaimed over several hundred years. Historic maps were studied and potential areas that once contained waterfront structures such

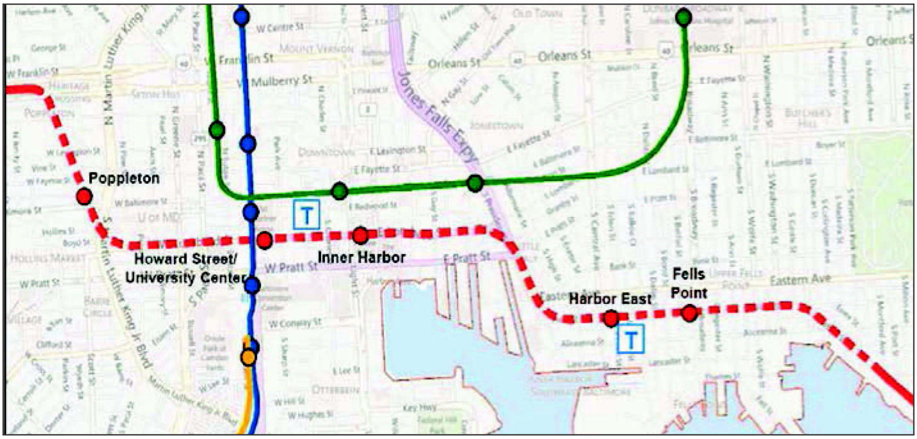

Figure 4. Schematic plan alignment for downtown tunnel

as piers, quays, etc., have been identified. A good portion of the alignment is also through the areas that were consumed by the 1904 Great Baltimore Fire, where in the aftermath, rubble was pushed into basements and pits. These locations are not always documented.

The minimum horizontal radius has been set to about 650 feet to permit both an achievable tunnel excavation radius and the desired maximum LRT vehicle speed of approximately 60 mph. The vertical alignment is constrained by a maximum operational grade of 7% in the tunnel approach sections, the presence of adjacent and overlying structures and facilities, underground conditions and anticipated tunnel construction methodology.

There are sections along the horizontal alignment where the tunnel will pass below existing buildings, structures, utilities and historic properties. This will include, for example, the Old St. Paul's Cemetery (National Historic Site), the U.S. Customs Building, the Constellation Energy Building and the Howard Street CSX freight railroad tunnel. Overlying the latter is the surface Baltimore Central Light Rail Line.

The alignment will pass beneath or directly adjacent to multiple buildings, including both older row houses and recently constructed mid-rise buildings (see Figure 5). The older row houses presumably have rubble-wall foundations constructed on non-engineered fill, much of it placed partially in former offshore areas or along the shoreline. Newer midrise buildings are located on either deep foundations or spread footings.

The current alignment utilizes twin running tunnels with a horizontal separation of 18 feet, which is controlled by ROW width of approximately 64 feet along the tunnel alignment. At the interfaces with stations, the pillar width between tunnels will need to be locally reduced to fit station configurations and ROW constraints. The station boxes are extremely confined due to adjacent structures. Accommodating the station footprints has been very challenging. Finding station construction work areas have been very difficult.

The vertical alignment is located at depths to accommodate the station configuration (three-tiered or two-tiered, based on cost evaluation studies), to avoid key underground infrastructures such as the CSX freight tunnel or the Constellation Energy Building as well as to minimize impact on utilities and adjacent structures.

Locating the tunnels at greater depths fully in rock is generally desired but difficult to accomplish due to the relatively deep and undulating top of rock surface. In contrast, a more shallow vertical alignment on the westerly and easterly sections would generally increase excavation impacts on all overlying facilities and would also locate tunnels to

Figure 5. Narrow right-of-ways with adjacent historic sensitive structures

a greater amount in undesirable high groundwater inflow mixed-soil interfaces involving the permeable Cretaceous Group, while the amount of mixed-face conditions with rock in the invert would almost remain the same. The current vertical alignment is a compromise. A special study was undertaken to estimate probable pile tip elevations of former structures using pile drivability analyses, since it is anticipated that a number of piles have been abandoned in-place during various past land reclamations. The current tunnel profile is generally below these elevations.

TUNNELING CONSIDERATIONS

General

All general tunnel excavation methodologies were studied for application to both the CLT and DTT Segments and consisted of:

- Cut & cover,
- SEM (including Roadheader), and
- Mechanized tunneling (twin TBMs or one large TBM).

Site constraints, political considerations and geotechnical conditions all played a major role in the current preferred excavation methodology of both tunnel sections.

Cooks Lane Tunnel

Due to the tunnel length and depth, the geologic and surface conditions, and proximity/access to existing residential areas along the CLT alignment, the use of cut & cover for the full length was found to be neither a cost effective nor a technically efficient solution. Furthermore, it would be extremely disruptive to the neighborhood, more expensive than mined tunnels, and would significantly impact traffic along Cooks Lane. It was found to be more appropriate to construct the tunnel by using either TBM or SEM means. However, limited use of cut & cover and retained cut sections at the extreme ends of the tunnel alignment will still be necessary to accommodate mined tunnels.

A geotechnical consideration for the CLT is that although the majority of the tunnel is in variable quality rock, there is about 1,300 feet of mixed-face ground conditions. The major component of the mixed-face conditions is the Transition Group material overlying rock. The Transition Group material is anticipated to have very short standup time (i.e., minutes) when below the groundwater table. The standup time is appreciably improved by pre-drainage or the use of ground treatment such as jet grouting (albeit difficult).

Considerations were given to providing ground treatment from the surface to improve the ground in the mixed-face ground condition sections. This would allow the

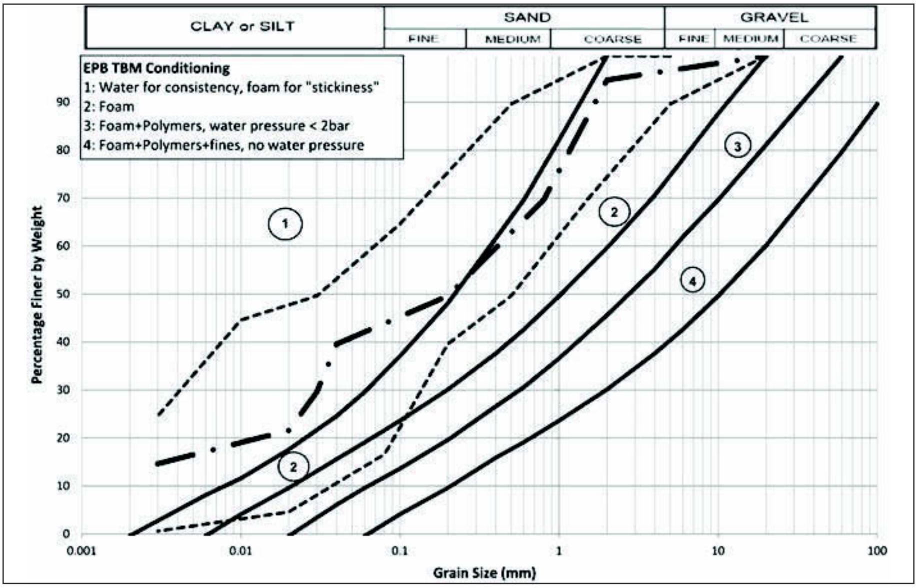

Figure 6. Transition group soil gradation versus EPB TBM application limits

use of SEM or a standard hard rock TBM. However, the surface disruptions would not be acceptable to the public. The application of ground treatment ahead of the tunnel face from within the tunnel is also doable but costly, time consuming, and risky. Preliminary cost analyses and risk assessments indicate that a pressurized-face TBM would be cost effective. Such a TBM would be a hybrid, essentially a hard rock TBM with pressurized-face capabilities. These TBM types have been successfully utilized in a number of recent tunnel projects. For the CLT, both Earth Pressure Balance (EPB) and Slurry Face (SF) pressured-face TBMs were considered. Due to the characteristics of the Transition Group, which is the material requiring face stabilization during excavation, the EPB TBM is far more economical and technically suitable for CLT.

Figure 6 superimposes the current Transition Group grain size distributions (dashed lines representing minimum, median, maximum grain sizes) on the EPB standard application limits in terms of grain size. Permeability is also a factor, but the general range of the Transition Group permeability is well within the typical limit of EPB application. Figure 6 illustrates that generally the Transition Group material can be confidently handled by a standard EPB TBM application.

Preliminary studies have been conducted regarding using either a single large TBM or a single standard subway tunnel size TBM (with twin bores). Currently, the twin TBM bores with a single TBM making two excavation passes is favored although the issue is not closed. The single large TBM bore results in more mixed-face conditions and more impacts to utilities and adjacent structures (due to a larger wider horizontal curve). Future studies will expand the comparison as well as further consideration for SEM excavation, although the impacts of SEM with blasting and the need for major ground stabilization may continue to negate its applicability. Cross passages would be mined with SEM techniques. Excavation with a roadheader would be difficult to the high strength abrasive rock.

Downtown Tunnel

Cut & cover excavation for the running tunnels has been ruled out since the alignment would need to have sharper curves (with undesirable speed restrictions on LRT vehicles) to avoid going under a number of structures including some historic (where underpinning is not desirable). In addition, the massive surface disruption would not be acceptable. SEM tunnel excavation was considered but would need major groundwater control particularly for the eastern areas whereby there are high permeability Cretaceous sands/gravels with recharge from the nearby harbor. Construction dewatering is not advisable as there are a number of adjacent areas that are contaminated and/or have near surface compressible soils sensitive to groundwater drawdown. Hence, groundwater control would need to be handled by some form of ground improvement either from within the tunnel or from the ground surface ahead of an excavation. This application would need to take place over thousands of feet of the alignment. Such surface disruption for major ground improvement would not be acceptable, thus leaving ground improvement from within the tunnel, a costly and risky undertaking. SEM tunneling would require multiple headings due to the length of tunneling; potential construction works site areas are very limited if exist at all. It was determined that some form of mechanized tunneling with a TBM is most appropriate. However, any TBM would need to have both hard rock and pressurized-face capabilities. Hence a hard rock TBM with either an EPB or SF configuration was suggested subject to detailed study.

TBM alternatives of either two subway size or a single large TBM have been studied. The current preferred TBM configuration is twin side-by-side tunnels. The large single TBM would require a deeper vertical alignment due to the need to pass under various structures. For the latter, the amount of mixed-face ground conditions increases as does excavation risks. Due to the larger turning radius, additional structures are impacted as well as the need to obtain more easements under private property.

More than one-third of the alignment contains Cretaceous deposits. The majority of the Cretaceous deposits are clean to silty sands/gravels. Long-term dewatering observations made during the construction of the Baltimore Metro Section C, which is within 1,000 feet of the DTTs, indicated that the Cretaceous deposits have a mass permeability of 1.5×10^{-2} cm/sec., typical of sands/gravels. Figure 7 illustrates the grain size distribution (dashed lines shows the minimum, median, maximum) for the clean sands/gravels. It is to be noted that these size distributions are based on standard split spoon sampling and are biased against larger size material such as large gravel and cobbles. On-going special sonic coring sampling indicates that some grain size distributions will likely shift further to the right (in Figure 7) due to inclusion of larger retained sample pieces. SF TBMs are suited for sands/gravels with relatively high permeability although EPB TBMs have handled, on a limited basis, such material with special soil conditioners. Figure 7 also includes application limits for SF TBMs. The grain size range for the Cretaceous clean sands/gravels fit in the range of "standard application + separation" limits for SF TBMs. Both the anticipated grain size distributions and mass permeability are well within the means of SF TBM applications. Currently, the SF TBM is the preferred TBM type for the DTTs.

The Transition Group is about one-fourth of the tunnel length. Figure 7 indicates that this material is suitable for an EPB TBM. A SF TBM can process this material but with additional separation effort which adds to the cost. The Transition Group breaks down into finer particles some of which have potential TBM clogging impacts. The SF TBM is particularly susceptible to clogging problems in cohesive type ground. The Transition Group's Consistency Index (CI) has been determined to assess "clogging potential" to TBM excavation and spoil handling components. Based on the categories defined by Thewes and Burger (2005), about 75% of the Transition Group has a "medium" to "high" clogging potential. Any TBM but especially a SF TBM will need to address the clogging potential. Additional issues to be addressed by a SF TBM

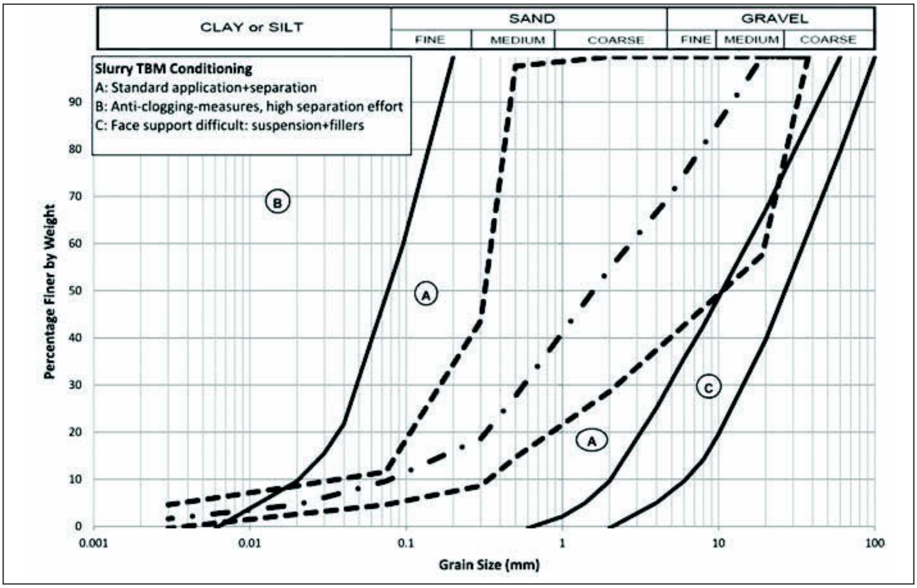

Figure 7. Cretaceous sands/gravel gradations vs. SF TBM application limits

application are excavation through ground treatment areas (with cementitious based materials) and the presence of brackish groundwater, both conditions which will diminish the performance of fresh slurry.

Cross passages are planned as SEM excavations. They have been located so as to avoid mixed-face conditions. Some of these cross passages will be in the Cretaceous Deposits where currently it is planned to do chemical grouting prior to excavation.

CONTRACT PACKAGING AND PROCUREMENT STRATEGY

At its inception, the entire Project was envisioned to be delivered by conventional DBB contracts. At about 50% of the preliminary engineering design stage, the PMC initiated a process of evaluating alternative methods of project delivery, and as a secondary focus, the contract packaging options with those alternatives. Concurrently the Maryland Department of Transportation/MTA program leadership conducted an evaluation of private finance alternatives.

For the project delivery evaluation, a team was assembled including representatives of MTA, PMC, and GEC with substantial experience in DB, as well as in DBB projects. The team analyzed the entire Project including all alignment segments and all types of contracts (trackwork, systems, finishes etc.). This section of the report includes discussion of tunnel construction and other contracts being mentioned only as they relate to the Project's tunnel segments. The team discussion was conducted mostly along DBB versus DB lines, although other delivery methods have also been considered by adding Maintenance or Operations and Maintenance. Potential contract packaging options were compared based on a number of objectives that included reducing potential for claims, providing bidding competition, reducing overall schedule duration, minimizing traffic disruption, assuring clarity of responsibility of the Contractors. Also consideration has been given to reducing risk (by transferring risk to the party best able to manage the risk through the contract), schedule saving (in reduction in overall project critical path), potential for innovations (allowing where appropriate the early involvement of the Contractor in design and facilitating the use of proprietary means and methods that could otherwise not be realized in a prescriptive design basis), cost reduction, market acceptance (in realizing three to five competitive bids for any one package reflecting competition and best value for money), and some others.

Other objectives brought out in discussion included meeting commitments and undertakings given during public consultation, optimizing opportunity for Contractor innovations to reduce costs in any DB packaging and providing maximum opportunity for involvement of local contractors and suppliers.

Single DB Contract

A single DB contract for the whole alignment was believed to be the most challenging because of the significant difficulty of drafting a conclusive and complete set of MTA's requirements given the very complex commitments provided to the public and the significant public interaction required in satisfying those commitments. The difficulty to provide a Design/Builder with a clear set of requirements in the timescale necessary to maintain the overall schedule was acknowledged, but it was also understood that any of the more comprehensive delivery approaches would require this. One concern was that a less than precise set of requirements would lead to ambiguity resulting in claims and scope changes once a contract was awarded, offsetting any financial or schedule advantage of an initial low bid.

There was also some concern that allowing a DB Contractor to choose the vertical alignment for the DTT, and thus determine the depth and configuration of stations with some assumptions as to actual ground conditions to be encountered could come back to MTA should a shallow alignment be chosen and subsequently encounter significant obstructions and mixed face conditions much worse than could be reflected in bid documents. Also, the vertical and horizontal physical and geotechnical constraints in the tunnel sections favored an approach to procurement aligned to risk reduction rather than potential cost reduction opportunity through innovations in design or through means/methods. The sensitive nature of existing building foundations would also benefit from a more prescribed vertical alignment ensuring risk of settlement was minimized.

Single DBB Contract

There were significant interface risks in the DTT segment containing utility relocation and temporary street decking, slurry wall installation and associated settlement protection, temporary bracing slurry walls once the station boxes were excavated, and excavation itself in station boxes. The coordination of temporary means and methods dependent on plant and equipment to be used by the Contractor appeared to be better managed through a DB approach. It was clearly understood that the trade-off was in the reduced expectation of Contractor innovations and creative solutions.

Hybrid DB/DBB Contract

The team recognized the considerable political pressure to visibly show a commitment in providing local contractors and material suppliers employment and opportunity through this major Project while compliance with Federal procurement prohibits formal expectation of local preference for firms. It was considered this might be best achieved through a combination of multiple DB and DBB contracts. There were also recognized limited opportunities for contractor tunneling innovations given the very restrictive scope lines in the surface section horizontal profile.

Downtown Segment

The Downtown Segment comprising twin bore tunnel and associated cross passages together with five underground stations deemed to be suitable for a single DB tunneling contract with the inclusion of associated utility relocation (or support) and temporary street decking. The single contract would also include slurry walls installation at all underground stations, excavation of station boxes, and temporary bracing of slurry walls as may be necessary to allow for staged excavation of the boxes and the station invert slabs. This approach would provide the Contractor with total control over the logistics of the construction, minimize risk from interface constraints between tunnel and station boxes, and allow greatest opportunity for the Contractor in managing resources and staging areas. Tunnel finishes, invert slab, station structure and station finishes, together with mechanical/electrical/plumbing (MEP) equipment and ventilation structures were considered to be more suited to a DBB approach based around three separate contracts thus encouraging maximum participation from local contractors.

Tunnel trackwork, installation of systems (overhead catenaries, communications, signals, power supply, and traction power distribution), and station fare collection equipment under this model will likely be part of a system-wide contract. Restoration of streets (including utilities) will be part of the particular DBB station finish contract.

The possibility of including all underground station finishes with the tunnel excavation DB package was discussed but agreed that it would create a very large single package limiting the competition. This might also generate opposition by not supporting the participation of local medium-size contractors. In addition, it would not necessarily result in any cost or schedule advantages or reduce any significant interface risk.

The option to undertake utility relocations under a contract separate from the tunnel excavation and station box construction was felt to put in some amount of additional risk. The utility relocation and/or suspension from temporary street decking is so interconnected with slurry wall construction that resulting potential schedule delays due to interfacing could become significant. As part of the DB contract for tunnels and station boxes excavation, the utility relocations could be performed during final design and TBM procurement.

Cooks Lane Segment

The Cook's Lane Segment deemed to be best delivered through a single DB contract that would include excavation of tunnels and cross passages. Tunnel finishes and MEP were believed more suited to a DBB approach. It was felt unlikely that there would be any benefit from aggregating the CLT with the DTT—such as re-use of tunnel-boring machine, etc.

Other Methods of Project Delivery

The team participants also addressed how well other procurement methods might satisfy the above-mentioned key Project objectives of a procurement strategy. The following approaches have been considered:

- Design/Bid/Build,
- Design/Build (More prescriptive),
- Design/Build (Less prescriptive),
- Design/Build/Maintain, and
- Design/Build/Operate/Maintain.

Design/Build 'more prescriptive' method provides a very limited choice to the DB Contractor in terms of specifications or standard detailing. The DB Contractor is tasked with drawing up and completing the detailed design. This approach is contrasted with DB 'less prescriptive' where the Contractor is given unlimited design responsibility and would have to work to a performance specification only. On the other hand this approach presents added risk to the owner in the areas of compliance with commitments made to the communities along the alignment.

Presently the project is at the preliminary engineering stage. As the design progresses, the above-described procurement strategies will be further advanced before the final contract arrangement is determined.

Funding

The project Finance Plan calls for a Federal contribution of up to 50% of the program cost for design, equipment purchase, and construction from the FTA New Starts fund. A Full Funding Grant Agreement between Maryland and the FTA to define the federal contribution is expected to be in place no later than June 2014. The balance of funding is to come from the Maryland State Transportation Trust Fund (TTF). Augmentation of the Maryland TTF to provide for a better balance in the transportation program between highway and transit is currently under consideration and will be on the legislative agenda for the 2013 State Legislature which runs from January to April.

Public Private Partnerships

The State of Maryland has used P3 approach for port, airport, and ancillary highway facilities infrastructure. P3s are under active consideration for several transit infrastructure projects around the country, and Denver RTD is using the process for the Eagle Commuter Rail Project in the Denver, Colorado area. Maryland's Office of the Governor has been studying policy choices to vastly expand the use of P3 for all types of infrastructure. The opportunity for use of a P3 on the Baltimore Red Line is under active consideration.

ACKNOWLEDGMENTS

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DESIGNING AT THE LIMIT: BRISBANE AIRPORT LINK SEGMENTAL LINING

Anthony Harding ■ Halcrow

Owen Francis ■ Arup

ABSTRACT

At 11.34m internal diameter the mainline TBM tunnels for the Brisbane Airport Link Project are the largest SFRC segmental lining in the world. As well as the challenge of designing the lining for ground that varies from soil to hard rock with water pressures of up to 5 bar, the design made provision for a number of unique construction challenges. These included large cavern traverses comprising invert excavation and partial ring construction, and the installation of a precast concrete road deck which required the use of a 50m long longitudinal bridge and gantry crane supported from the erector cone recesses in the segments. This paper describes how, working closely with the contractor, these challenges were overcome.

PROJECT BACKGROUND

With a combined price tag of around five billion US dollars, Brisbane's Airport Link, Northern Busway (Windsor to Kedron) and Airport Roundabout Upgrade project is Australia's largest ever road infrastructure project to date, connecting Brisbane city to the northern suburbs and airport precinct. Around 75% of the price was for the construction of the 6.7km Airport Link toll road, whose 5.7km tunnel section required a total of 15km of tunnels, including intermediate exit and entry ramps.

Conducted as a public-private partnership (PPP), BrisConnections (composed of Macquarie Group, Thiess and John Holland) were awarded the 45 year concession to own and operate the Airport Link tunnel in June 2008. A Thiess John Holland Joint Venture (TJH JV) was contracted by BrisConnections to undertake the design, construction and commissioning of the project. TJH JV engaged a Parsons Brinckerhoff Arup Joint Venture (PBA JV) to undertake the design (excluding mechanical and electrical design, which was carried out by a separate M&E fit out team). Halcrow staff were seconded into PBA's tunnel design team to provide additional breadth of expertise in steel fiber reinforced segmental lining design.

The extent and general arrangement of the mainline TBM bored tunnel works is shown in Figure 1. Key elements of the works include:

- Bored (TBM) tunnels between Lutwyche and Toombul
- Mined caverns at Kedron and Lutwyche to connect to the on and off ramps
- Mined mainline tunnels between the southern connection at Bowen Hills and Lutwyche

The twin bore tunnels for the two lane sections of main line tunnels were constructed by two 12.48m earth pressure balance TBMs through a mixture of soft ground, mixed face conditions, and rock varying from highly weathered sedimentary silts and sandstones to competent tuff. The final lining is exposed to water pressures of up to 5 bar.

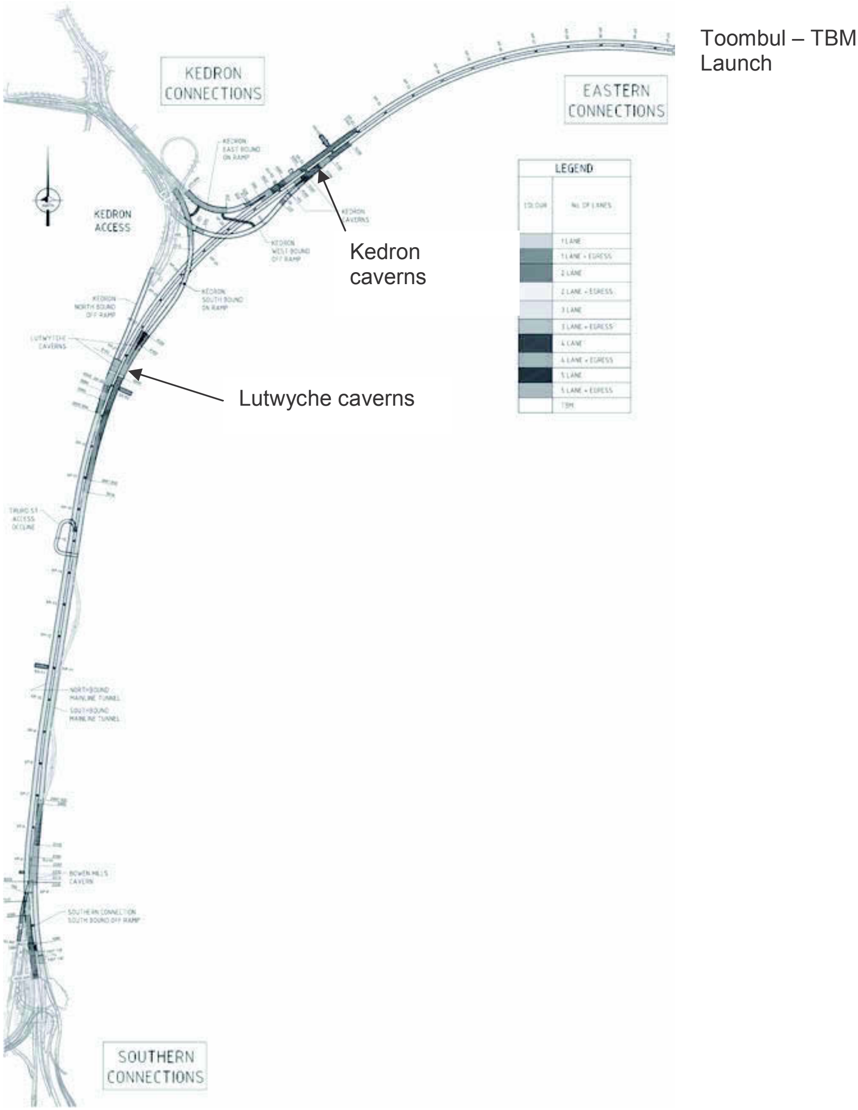

Figure 1. Extent and general arrangement of the Airport Link tunnels

The tunnel drives started westbound from from Toombul in the east, traversing the Kedron caverns and terminating just short of the Lutwyche caverns. The drive from Toombul to Kedron was approximately 1500m and the drive from Kedron to Lutwyche approximately 500m.

As is common for a PPP project, there was a strong drive not only to keep costs down but also to improve program and bring revenue in early. This drive was apparent throughout the design of the TBM tunnels, and introduced a number of key challenges for the segmental lining design. The remainder of the paper describes a number of constraints on the lining design, and the solutions adopted. Particular detail is provided on the gantry bridge and some of the complexities of the SFRC design. Finally some of the ensuing construction challenges are described, along with their solutions.

KEY CHALLENGES

The success of the TBM tunneling depended on the lining design accommodating construction activities and requirements that would facilitate the meeting of the program. Through collaboration between designer and contractor, construction challenges and design constraints were identified and solutions developed. Key aspects of this collaboration were:

- Management of both organizations promoting a 'best for project' attitude.
- Construction and design issues identified were raised as they became apparent and resolutions agreed quickly.
- Meetings every fortnight involving design and construction teams to look at all the developing options, and the status of outstanding issues.
- Co-location of construction team design managers and design team ensured that many ad-hoc conversations occurred which significantly improved overall communication between design and construction teams.

The main constraints are summarized in Table 1. As well as the requirements to design the lining in SFRC, other key challenges emerged both for the design and construction phases of the project, and these are described further in the remainder of the paper.

GANTRY BRIDGE AND SEGMENTATION

The selection of the segmentation was an early design process and as such required the development of a number of options, with the pros and cons of each being carefully considered by both design and construction teams. A number of focused meetings were held to ensure that all concerns were understood and adequately addressed.

The most significant of the constraints was the requirement to support the gantry bridge from the erector cones in the segments. To provide as much support as possible in the gantry, it was desirable to have the supports as low down as possible. Conversely the supports had to be but sufficiently above the road deck to ensure that the support never clashed with the road deck construction. This is illustrated in Figure 2.

From past experience it was thought that the limit on aspect ratio for a SFRC lining was 1:11.5, and this led to the selection of a 9+1 segmentation. 10+1 was rejected as it would lead to an undesirable increase in erection times. Having studied a number of options, an arrangement of two bolt pairs per segment and one per key was adopted. This arrangement allowed the ring to alternate between left hand support and right hand support being available, leading to one support every 4m (2 rings) on either side.

Under this arrangement it was not possible to have sufficient build positions available and avoid cruciform joints if the segments were all the same size. Therefore the segments were varied in size such that the joint positions would not coincide. The approach was to vary the position of each joint relative the midpoint between the bolt pairs on either side of the joint by multiples of 90mm on intrados as illustrated in Figure 2. By careful selection of which joints were in which relative position, cruciform joints could only occur in three of the 19 possible build positions (Figure 3).

The constraints on the lining were therefore threefold:

- Three relative (ring to ring) positions with cruciform joints
- Four absolute positions that could not be built due to no gantry support being available

Table 1. Constraints on the lining design

Element	Requirement	Solution
Gantry bridge	Program savings from constructing road deck within TBM backup using a gantry bridge	Segmentation allowed for alternating gantry bridge supports for erector cones
Cruciform joints	Contractual requirement to avoid cruciform joints	Non-standard segment sizes to minimize number of build positions with cruciform joints, hence maximizing build positions
Build positions	Maximise build positions to follow alignment	
Segment reinforcement	Program/cost savings from reduction/elimination of conventional re-bar	Steel Fibre Reinforced Concrete (SFRC) segments
Road deck	Program savings from road deck construction in parallel with tunnel excavation	Precast deck units were adopted and the segmental lining designed to accommodate the resultant concentrated point loading
Smoke duct slab	Program savings from Smoke duct construction in parallel with tunnel excavation	A hung smoke duct solution was adopted and the segment lining designed to accommodate the resultant concentrated loading
Location of cross passage openings	Ensure optimal position of steel opening sets for cross passages	Two largest segments selected to maximize opening size, positions verified as feasible.
Temporary support of cross passage openings	Temporary support of cross passage openings to impose minimal restriction space available for parallel construction activities	Opening rings designed with welded lintel and sill beams in low load sections, and fabricated steel segments with compact internal bracing frames elsewhere to minimize impact on space.
Fixing points for internal structures	Ensure that joints do not compromise fixing locations	Checked positions of joints in relation to structures, checked that positions feasible

Figure 2. Support brackets and gantry above

Joints	Could coincide with
2 and 6	2 or 6
3 and 7	3 or 7
4 and 8	4 or 8
5 and 9	5 or 9

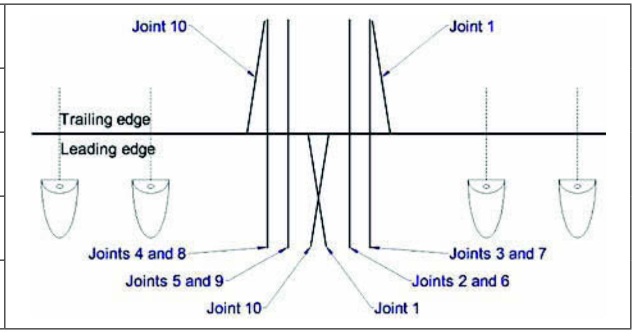

Figure 3. Illustration of where joints fall between two bolt pair positions

- Up to six positions that would provide the same gantry support as the previous ring and therefore be incompatible with the gantry bridge

The result was a ring that would provide between 8 and 12 buildable positions well distributed around the circle, thereby providing for effective application of the ring taper.

The location of the cross passage was carefully checked because steel 'opening sets' were to be employed to allow easy cross passage break-out. These opening sets comprise steel segments that are built into two or three successive rings to form a frame with easily removable opening. The arrangement was checked to ensure that sufficient opening could be provided with only two segments per ring, with adequate steel above and below to form the lintel and sill beams. By employing the two largest segments, a near-optimal position could be obtained, providing a better solution than would have been obtained with uniformly sized segments. Furthermore, gantry positions were available opposite the opening, meaning that bespoke gantry supports would be required on the opening side only (where they could be bolted to the steel segments).

STEEL FIBRE REINFORCED SEGMENTAL LINING DESIGN

The use of SFRC, rather than conventional bar reinforced concrete, is known to provide significant benefits for long term durability and maintenance of segmental tunnel linings. However, the structural capacity of SFRC is typically lower than that of conventionally reinforced concrete for segments of the same thickness.

For the SFRC segmental tunnel lining of the Airport Link Projects, the following distinctly different critical loading cases emerged:

1. Segment Handling
2. Ram thrust loading
3. Full overburden loading at the deepest section with soft ground in the crown
4. Rock wedge loading in self supporting rock with low or no water pressure

Segment Handling

Previous research (Rail Link Engineering, 1997) has suggested that the practical limit of SFRC is up to aspect ratios of 11.5:1, beyond which significant damage is expected due to handling. The largest segment had an aspect ratio of 1:11, which was very close to the limiting value. Therefore it appeared that that high levels of damage could result if handling was not carefully controlled.

Working together, the designer and contractor identified every single handling stage that the segments would be exposed to: from being lifted out of the moulds to

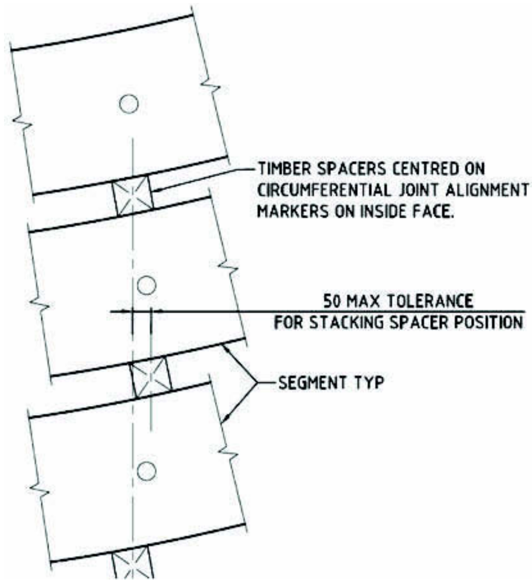

Figure 4. Offsets between timber spacers

being installed in the TBM. Some 11 steps were identified, (not including vacuum lifting of individual segments, which was ignored), and each one examined to set constraints that would limit excessive bending. The key driver of the bending was the relative positions of the timber spacers between the segment of the stack, and the stack supports in each arrangement, as illustrated in Figure 4. By coordinating the design of each piece of handling equipment excessive moments and flexural damage were almost eliminated. Furthermore, the enhanced toughness (resistance to impact) of SFRC over normal concrete also limited other types of damage, leading a negligible number of segments rejected due to damage. This suggests that thorough coordination of stacking and handling could result in increases in practical handling sizes beyond 11:1 in the future.

Ram Thrust Loading

The circumferential joints of the lining are subject to concentrated loading from the TBM ram load. The TBM ram thrust loading was developed by 19 pairs of thrust cylinders and applied to the circle joint by 19 equally spaced, 900mm long × 360mm wide ram shoes. For the 9+1 segmentation adopted this resulted in 2 shoes per segment and 1 shoe per key. The maximum installed thrust was approximately 89MN. The typical operational thrust varied up to approximately 60MN. The design analysis revealed that as the applied ram thrust approached maximum installed, splitting at the circle joint face between adjacent ram shoes rather than bursting beneath ram shoes was the limiting case for the standard SFRC (Type 1) segments. The tensile stress developed between ram shoes is shown in Figure 5. A sensitivity analysis was then carried out to determine the limiting ram thrust to control splitting. In the most onerous case, the required thrust limitation still permitted 87% of the maximum installed thrust to be applied.

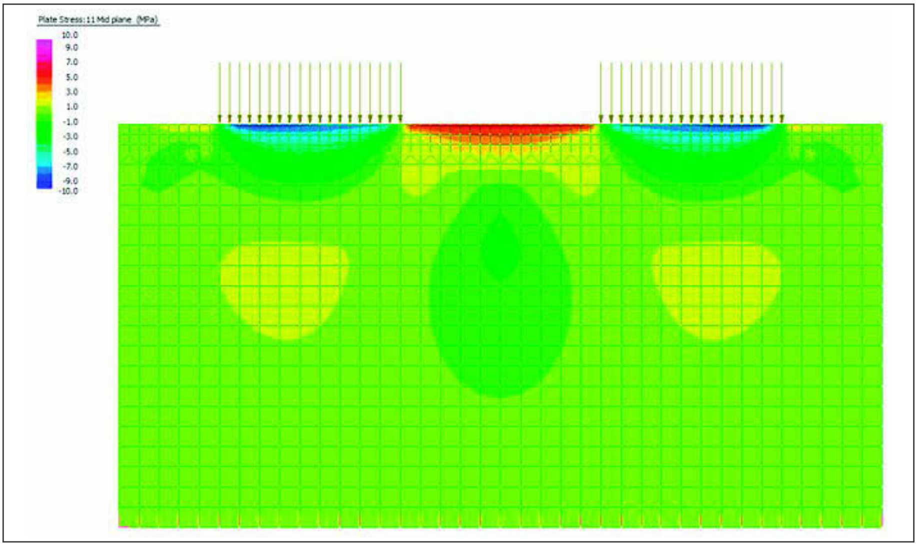

Figure 5. Simplified analysis showing peak tensile (splitting) stress between shoes

Full Overburden

Once the final ground investigation was complete it became apparent that mixed face conditions, in which the overlying alluvial soil extended to just below tunnel crown, would occur over a significant length of the alignment after launch. Despite the alignment adopting the maximum allowable grade over the initial 200m of drive in an effort to reduce the extent of mixed face conditions, conditions of negligible and very low rock cover continued to exist for some considerable distance.

As a consequence, the linings were subject to full overburden loading conditions for tunnel depths of up to 2 diameters. Although large bending moments were induced, these tended to occur in conjunction with large axial thrusts such that the resultant moment-thrust combination was not critical. However, for the largest axial thrusts, the resultant localized stress induced at the radial joint was in excess of the bursting capacity of the standard SFRC segments.

The highest hoop thrusts in the lining occurred over an approximately 700m long section of tunnel subject to full overburden loading approaching 2 tunnel diameters, as previously described. The issue of bursting at radial joints is a complex one (Francis and Mangione 2012), and therefore needed careful consideration. Therefore extensive analysis of the joint behavior in this highly loaded length of tunnel was undertaken using a combination of closed form solutions, finite element analysis, and back analysis of testing of the CTRL ring testing. The analysis concluded that the bursting stress at the radial joints could not be accommodated by SFRC alone and localized bar reinforcement was designed to provide the required bursting capacity. The resultant hybrid segments contained the same dosage of steel fibers as Type 1 segment together with localized bar reinforcement at the radial joint.

For the remainder of the tunnel extent, the design analysis revealed that bursting stress induced at the radial joints could be accommodated by standard SFRC segments.

Rock Wedge Loading

For self-supporting rock conditions the localized wedges and blocks which could be released as a result of the tunnel excavation were estimated on the basis of the joint set information, excavated tunnel diameter and tunnel orientation.

The critical cases were found to occur immediately adjacent to the drained Kedron and Lutwyche caverns where water pressure was very low. In these circumstances the bending moments induced by the maximum rock wedge loading occurred without the beneficial effect of significant hoop thrust. These sections required careful consideration to demonstrate that the moment-thrust capacity of the SFRC lining was sufficient.

CONSTRUCTION CHALLENGES

Mix Design

Based on experience from past projects the recommendation from the designers was to start at least six months before production, which is 2–3 times the time typically allowed for conventionally reinforced concrete. However, this was borne out as the initial mix, which was simply a conventional mix to which fibers had been added, failed to meet the relatively onerous requirement for an equivalent flexural strength at 3mm of 3.4MPa. The requirement for an adequate early stripping strength (12 MPa), combined with a compressive strength requirement of 55MPa, lead to actual strengths in the region of 70–80 MPa, resulting in brittle failure of the flexural test beams and decreased residual flexural strengths.

Considerable refinement was required to arrive at a mix that provided the required stripping strength while still providing sufficient ductility to meet the flexural strength requirement. This was achieved by:

- Increasing the proportion of fly ash in the binder from 20% to nearly 29%
- Controlling the compressive strength to keep the average closer to 65MPa
- Adjusting the proportions of sand and aggregate in the mix to improve pull-out performance of the fibers

The requirement to restrict the compressive strength was managed by a carefully controlled target compressive strength, which resulted in a small number of segments that were below the required compressive strength. However, it was relatively straightforward to demonstrate that these segments would have adequate strength for the low-loaded sections of tunnel, and so the low strength segments were simply restricted for use in those locations.

Radial Joint Reinforcement

The provision of the reinforcement at the joints required careful detailing to ensure that the reinforcement remained in place without being too costly to build. Rather than provide a light cage to hold the joint reinforcement in place, a 'spiral' arrangement was developed by TJH and the reinforcement supplier that ensured a minimum of steel was used. This is illustrated in Figure 6.

Gaps in the spiral were provided for the bolt pockets, and the weight of the reinforcement was relied upon to keep the reinforcement correctly positioned. There were concerns that the vibration would move the reinforcement, but coring of trial segments demonstrated that this was not a problem.

Cavern Traverse

The TBMs were required to traverse the length of the Kedron Caverns caverns, approximately 402m and 192m for the two drives respectively, before being re-launched at the western end. A number of options for the traverse were considered including: sliding on a cradle and rail system as well as disassembly and re-launch of the TBMs. However, the solution which was ultimately adopted is show in Figure 5 and comprised additional invert excavation (which was carried out by the TBM) followed by erection of partial rings in the invert to provide sufficient thrust reaction for forward advance. This solution offered a significant program saving and also allowed the precast road deck and sub-floor utilities design for the mainline tunnel profile to be continued largely unchanged through the cavern.

The partial excavation solution also presented many challenges, chief of which was the requirement to maintain the operation of the gantry bridge, which was supported by up-stands of the segmental lining, as shown in Figure 7(b). Without support these upstands of the partially complete ring did not have sufficient resistance to the loads from the gantry bridge.

Following a comprehensive discussion of the options, including the design and construction constraints, temporary structural steel props at the back of the up-stand segments were developed. By using two props instead of one, the required resistance to the loads from the gantry could be provided, while the resulting props were sufficiently small to handle and install in the relatively restricted space between the TBM and the cavern wall.

Figure 6. Radial joint reinforcing steel

(A) Partial invert excavation	(B) Upstand segment support

Figure 7. Cavern traverse

CONCLUSIONS

Extending the reach of a new technology can be difficult, and requires commitment throughout the project team. However, where such commitment exists, and where risks are effectively identified and managed, the boundaries of technology can be effectively extended, even in the midst of the cost and schedule pressures that are typical of a design and build PPP project.

This has been demonstrated on the TBM tunnels of the recently completed Brisbane Airport Link project, where the majority of the segmental lining was pure steel fiber reinforced concrete (SFRC)—the largest such lining in the world. In order to manage the design risks, analysis focused on understanding the problem first, so the risk could be understood and quantified. In design, careful analysis was undertaken to understand the structural behavior and also to determine the limits of that understanding. Strong communication between the construction and design teams ensured that what was designed met the manufacturing and construction requirements, while design requirements were effectively communicated through to the construction team to ensure that they were met.

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LINER SEGMENT DESIGN OF THE LARGEST TBM TUNNEL IN THE WORLD—ALASKAN WAY TUNNEL IN SEATTLE

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ABSTRACT

Alaskan Way (SR 99) Tunnel in Seattle will be the world's largest TBM bored tunnel. The 17.1-m-diameter (56 ft), 2,835-m-long (9,300 ft), bored tunnel under downtown Seattle will reach depths of up to 66 m (215 ft). Along its alignment, the tunnel traverses through variable glacially overconsolidated soil deposits with high groundwater pressures of up to 5.2 bars. The tunnel will convey traffic underneath downtown Seattle and replace the 1950s SR 99 Alaskan Way Viaduct that runs along Seattle's waterfront. The double-deck structure had been deteriorating and was further damaged in the 2001 Nisqually earthquake. This paper discusses the geo-structural analytical approach and parametric evaluations conducted for the precast concrete segmental liner design. Geotechnical evaluations included numerical modeling of ground relaxation, existing and potential future building loads, and varying geologic conditions. Also discussed will be the implementation of the results of the geotechnical continuum analyses, performed for the varying soil conditions, on the structural design of the liner segments to meet strict service and strength requirements. The paper also describes the analytical procedure and design approach implemented for the seismic design of the tunnel.

SR99 TUNNEL PROJECT

The Alaskan Way (SR 99) Tunnel is being constructed as part of the Washington State Department of Transportation's (WSDOT) Alaskan Way Viaduct Replacement Program. The Alaskan Way Viaduct is an aging double-deck highway structure in Seattle, Washington that was built in the 1950s. The Viaduct has been deteriorating due to the age of the structure, as well as damage resulting from the 2001 Nisqually earthquake. The SR 99 Tunnel consists of three segments: the South Approach, Bored Tunnel, and North Approach. The South and North Approaches include cut-and-cover tunnels and U-sections. This paper focuses on liner design for the bored tunnel segment.

The new SR 99 Bored Tunnel alignment is shown on Figure 1 in plan view and on Figure 2 in profile view (note tunnel stationing is provided in feet). The bored tunnel begins south of downtown Seattle near Elliott Bay, extends north along the existing Alaskan Way Viaduct, then crosses under the Viaduct, traverses under downtown Seattle, and emerges north of downtown Seattle just east of the Space Needle. Development along the alignment consists of on-grade and elevated roadways, buildings ranging from single-story to high-rise developments, railroad and sewer tunnels, and public and private utilities. The bored tunnel will be approximately 2,835 m (9,300 ft) long. The tunnel will be excavated by an Earth Pressure Balance (EPB) Tunnel Boring Machine (TBM) with a 17.5-m-diameter (57.5 ft) cutterhead, and the tunnel liner will have an outer diameter of

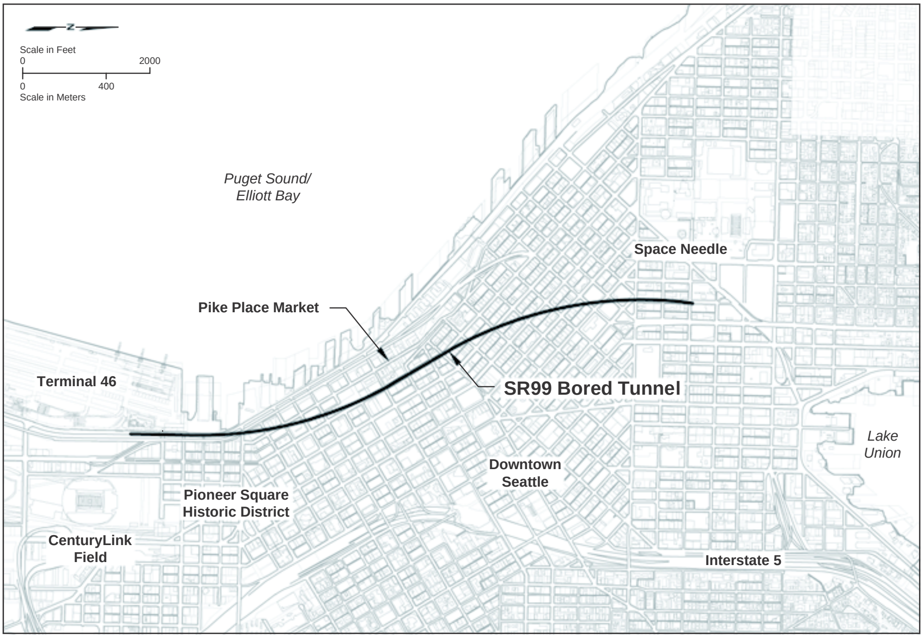

Figure 1. Plan view of SR 99 bored tunnel alignment

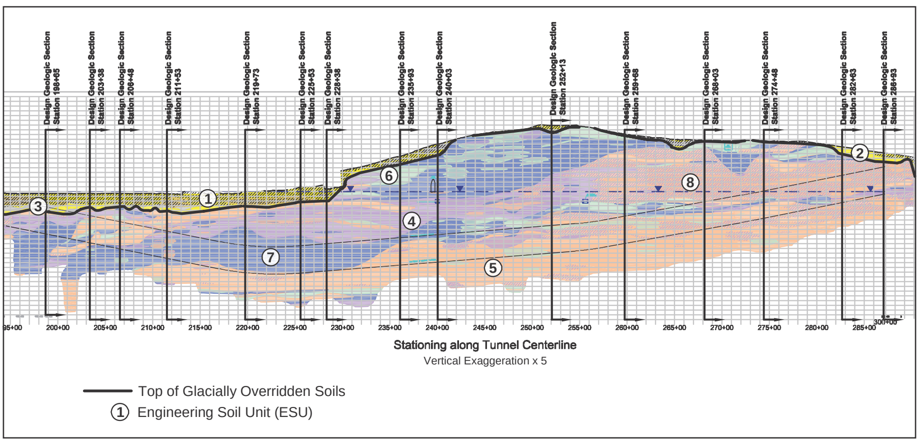

Figure 2. Subsurface profile and design geologic sections along SR 99 bored tunnel alignment

17.1 m (56 ft). At its lowest point, the tunnel crown is at elevation –29 m (–95 ft), and the tunnel crown is 66 m (215 ft) deep at its greatest depth.

This paper discusses the geo-structural analytical approach and parametric evaluations conducted for the precast concrete segmental liner design. Geotechnical evaluations included numerical modeling of ground relaxation, existing and potential future building loads, and varying geologic conditions. Implementation of the results of the geotechnical modeling on the structural design of the liner segments to meet strict service and strength requirements is also discussed. Additionally, this paper presents an efficient approach for conducting non-linear dynamic time history analyses for the seismic design and discusses the technique to demonstrate that the tunnel satisfies the stringent seismic performance objectives for two levels of design earthquakes with 108-year and 2,500-year return periods.

GEOLOGIC/SUBSURFACE CONDITIONS

Geology

Seattle is located adjacent to Puget Sound in the Puget Lowland between the Olympic Mountains to the west and the Cascade Range to the east. The Puget Lowland has been subject to several glacial advances, resulting in a complex stratigraphy of glacial and non-glacial soil deposits (Galster and Laprade 1991). All but the most recent recessional glacial deposits and non-glacial deposits have been overridden and overconsolidated by glacial ice. The north-south trending, elongate hills and valleys left behind by the last glacial advance and retreat have been partially eroded and filled naturally, and several natural lakes developed as the glacier retreated. Additionally, substantial regrading occurred in the late 19th and early 20th centuries, resulting in cutting of hills and filling of valleys and tidelands.

While the excavations and structures for the approaches at the north and south ends of the bored tunnel must contend with significant depths of recent, normally consolidated soils and fill, the bored tunnel generally lies within glacially overridden soil deposits. These deposits are often highly variable within relatively short distances due to the variability in erosion and deposition during the multiple glacial events and interglacial periods.

The soil deposits were grouped into Engineering Soil Units (ESUs) based on their geotechnical characteristics. The ESUs along the SR 99 Bored Tunnel alignment are

Table 1. Description of engineering soil units

Stress History	ESU	Description
Recent Deposits	1	Highly heterogeneous mixture of sand, silt and clay with peat and wood debris. Variable characteristics that can change drastically within a short distance depending on its specific content.
	2	Loose to dense silt and sand with gravel. Includes normally consolidated alluvium, beach deposits, reworked glacial deposits, and recessional ice-contact deposits.
	3	Soft to very stiff low plasticity clay and silt with fine sand interbeds. Includes normally consolidated estuarine deposits and recessional lacustrine deposits.
Glacially Overridden/Overconsolidated	4	Very dense or hard cohesive mixture of gravel, sand, silt and clay. Includes glacially-overridden till, glaciomarine and till-like diamict deposits.
	5	Dense to very dense silty sand to sandy gravel. Includes glacially overridden glaciofluvial (outwash) deposits and non-glacial fluvial deposits.
	6	Very dense silt, silty fine sand, and fine sandy silt. Includes glacially overridden lacustrine deposits.
	7	Hard, interbedded, low to high plasticity silt and clay. Includes glacially overridden glaciomarine, glaciolacustrine, and non-glacial mud-flow deposits. Localized, trace to abundant zones of sheared and fractured/jointed slickensided soil attributed to glacial ice loading, stress relief upon unloading, and/or desiccation (Galster and Laprade 1991).
	8	Dense to very dense, unsorted mixture of gravel, sand and silt. Can have a similar appearance to ESU 4, but can vary over short distances and gradations include clean or relatively clean sand. Can be adjacent to and transition into ESU 4 and ESU 5.

Table 2. Geotechnical properties of engineering soil units

Stress History	ESU	Unit Weight, kN/m³ (pcf)	At-rest Lateral Earth Pressure Coefficient	Effective Friction Angle, degrees	Effective Cohesion, kPa (psf)	Maximum Shear Modulus, Pa (psf)
Recent Deposits	1	18.1 (115)	0.45	34	0	5.7×10^7 (1.2×10^6)
	2	19.6 (125)	0.40	36	0	1.1×10^8 (2.3×10^6)
	3	17.6–18.1 (112–115)	0.5–0.6	25–32	0	3.6×10^7–6.7×10^7 (7.5×10^5–1.4×10^6)
Glacially Overridden/ Overconsolidated	4	22.8 (145)	0.6	40	240 (5,000)	1.2×10^9 (2.5×10^7)
	5	20.4 (130)	0.8	39	0	8.1×10^8 (1.7×10^7)
	6	19.6 (125)	0.8	39	0	4.1×10^8 (8.6×10^6)
	7–Intact	18.9 (120)	1.4	25	57 (1,200)	4.1×10^8 (8.6×10^6)
	7–Residual	18.9 (120)	1.4	15	0	4.1×10^8 (8.6×10^6)
	8	22.8 (145)	1.0	0	0	1.4×10^9 (2.9×10^7)

briefly described in Table 1, and the design geotechnical properties for the ESUs are presented in Table 2. A subsurface profile showing the distribution of ESUs along the bored tunnel alignment is provided on Figure 2.

HYDROGEOLOGY

There are two general aquifers along the bored tunnel alignment. The upper aquifer is unconfined and present throughout the recent, normally consolidated deposits and often occurs as perched groundwater on underlying glacially overridden soils. The lower aquifer is typically confined at the elevation of the bored tunnel, regionally recharged, and present in the glacially overridden deposits. The bored tunnel alignment is almost entirely within the lower aquifer. Based on seasonal variation of groundwater levels measured near the bored tunnel elevation, an upper bound groundwater elevation of 6 m (20 ft) was used for liner design, resulting in groundwater pressures as high as 5.2 bars at the deepest location of the tunnel invert.

Regional Seismicity

The seismicity of western Washington is dominated by the Cascadia Subduction Zone in which the offshore Juan de Fuca plate is subducting beneath the continental North

American plate. Three main types of earthquakes are typically associated with subduction zone environments—crustal, intraplate, and interplate earthquakes. Seismic records for the Puget Sound area indicate a distinct shallow zone of crustal seismicity (the Seattle Fault Zone) that may have surficial expressions and can extend to depths of up to 25 to 30 km (16 to 19 mi). The northern-most splay of the Seattle Fault Zone is approximately 2.4 km (1.5 mi) south of the south end the bored tunnel (U.S. Geological Survey 2006). A deeper zone is associated with the subducting Juan de Fuca plate and produces intraplate earthquakes at depths of 40 to 70 km (25 to 43 mi) beneath the Puget Sound region (e.g., the 1949, Magnitude (M) = 7.1; 1965, M = 6.5; and 2001, M = 6.8 earthquakes) and interplate earthquakes at shallow depths near the Washington coast (e.g., the 1700, M ~ 9.0 earthquake).

DESIGN CRITERIA

Precast segmented lining was chosen for the bored tunnel. The lining segments will be installed using an EPB TBM. The Design Life of the bored tunnel is required to be 100 years. The bored tunnel has an interior diameter of 15.8 m (52 ft) that provides a 9.8-m (32-ft) curb-to-curb roadway width and 4.7-m (15.5-ft) vertical clearance within the bored tunnel. The bored tunnel accommodates two 3.4-m-wide (11 ft) travel lanes and 2.4-m-wide (8 ft) west and 0.6-m-wide (2 ft) east shoulders in each direction, providing a uniform shoulder travel way and accommodating larger vehicles to transit and move goods and services through the tunnel. The tunnel clearance envelope features a consistent vertical and horizontal cross section from the cut-and-cover sections through the bored tunnel.

The precast segmented lining is made of universal rings that are 0.6-m-thick (2 ft) and approximately 2.0-m-wide (6.5 ft), consisting of seven typical segments, two counter segments, and one key segment. The universal ring will be placed to ensure that there are no continuous joints between ring segments. This is an advantage when considering water tightness and structural strength. In addition, it will allow the TBM to negotiate sharp vertical curves if needed to respond to an unexpected change of face conditions.

Contract Requirements

The SR 99 Bored Tunnel Project contract identified existing buildings and other structures that were required to be analyzed in the liner design. For consideration of potential future development, the contract also required evaluation of a 335 kPa (7,000 psf) building surcharge applied at the height and width limits of WSDOT's right of way above the tunnel, which is to 16.5 m (54 ft) above the crown and 25.6 m (84 ft) wide.

Dual levels of design earthquakes, Expected and Rare Earthquakes, were considered for the design of the tunnel liner. The Expected Earthquake has a 108-year return period and is associated with an Operational Performance Objective, while the Rare Earthquake has a 2,500-year return period and is associated with a Life Safety Performance Objective. Under the Expected Earthquake, minimal damage to the liner segments, joints and water tightness is anticipated because the lining is designed to respond in an elastic manner. Concrete compression strain is limited to 0.003, and tensile strain in reinforcing steel is limited to 0.002. Under the Rare Earthquake, the objective is to prevent collapse of the tunnel liner. Inelastic deformations are allowed under the Rare Earthquake but are limited to the acceptable levels. Concrete strain is allowed to exceed 0.003 but limited to 0.005 provided that the strain is predominantly due to flexure. The tensile strains in a mild reinforcing steel is limited to 0.06 for reinforcing bars up to US #10 size and 0.045 for US #11 size and larger.

DESIGN METHODOLOGY

Geo-Structural Analysis

The general geo-structural design used a two-step approach:

1. Geotechnical numerical models were used to determine static, unfactored soil and groundwater loads; seismic ground deformations; and soil springs for soil-structure interaction used in the structural design of the tunnel liner.
2. Structural numerical models were used to evaluate various limit states of the tunnel liner and interior structure using the soil and groundwater loads, seismic deformations, and soil springs from the geotechnical modeling.

The purposes of the static geotechnical numerical models were to assess the impacts of geologic variability, tunnel excavation and relaxation, and combinations of structure, building, and future loads on the soil loads transmitted to the liner. After reviewing these evaluations, select sections were further analyzed to evaluate seismic ground deformation using geotechnical models and various limit states using structural models.

Because the contract provided the minimum liner design criteria (i.e., 0.6 m (2 ft) thick, 1% steel reinforcement), the liner design analysis focused on evaluating the minimum criteria using conservative, simplifying assumptions; performing sensitivity analyses; and concentrating geo-structural analysis effort on critical design sections to determine if a more robust liner was necessary.

DESIGN ANALYSIS

Static Analysis

Design geologic sections were selected to assess the geologic variability along the tunnel alignment, as well as topographic/geometric variability and building and structure locations. Figure 3 shows the 15 design geologic sections, which are also shown on the subsurface profile provided on Figure 2.

Based on review of the tunnel alignment and existing structures and buildings, including considerations for significant basement excavations and unbalanced loading/unloading conditions on the liner, buildings were identified for liner design evaluations in addition to those required by the contract. Also, tunnel deformation mitigation and buoyancy resistance measures at the south end of the alignment (South End Settlement Mitigation Plan, SESMP), which include jet grouting, drilled shafts, and an uplift slab that will influence loads on the liner, were evaluated. Figure 3 shows all the existing buildings and structures evaluated for the liner design (12 structures and 74 buildings). Building and structure foundations vary from spread footings and mat foundations to deep shafts and piles, ranging from 2.4 to 19 m (8 to 63 ft) long and as close as 4.9 m (16 ft) above the tunnel crown. Buildings along the tunnel alignment range from 4.0 to 166 m (13 to 546 ft) tall with basement excavations ranging from approximately 0 to 27 m (0 to 87 ft) deep.

Geotechnical Screening Analysis

Because of the large number of structures and buildings to evaluate, an initial assessment of induced stresses in an elastic medium was performed as a screening analysis. This elastic evaluation estimated the induced stresses at the tunnel crown for groups of structures and buildings, due to both structural loading and excavation unloading. Based on these estimated induced stresses, upper- and lower-bound influence cases were identified at the design geologic sections.

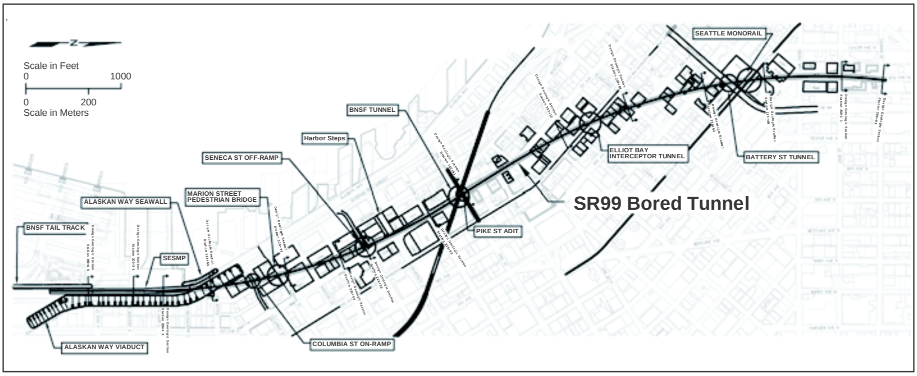

Figure 3. Buildings and structures that were evaluated and locations of design geologic sections

Geotechnical Modeling

The upper- and lower-bound influence cases from the screening analysis were evaluated using geotechnical numerical models. Static soil loads and springs, as well as preliminary unfactored structural liner reactions, were determined from the geotechnical models. Based on the results of these geotechnical models, representative design sections and loading cases were selected for detailed structural modeling.

Simplifying assumptions and sensitivity analyses. Several conservative, simplifying assumptions were made to increase modeling efficiency and check the minimum design criteria. If these assumptions resulted in issues for the minimum liner design, additional effort was focused on refining the evaluations at these critical design sections.

Soil modulus. For static tunnel liner modeling, the maximum shear moduli (see Table 2) were reduced based on the anticipated level of strain and published modulus reduction curves. Limited modeling performed using a non-linear soil modulus, indicated that using representative, constant soil moduli yielded similar or more conservative results. Thus, using a representative, constant soil modulus was deemed an acceptable simplification for static liner design.

Foundations. Shallow foundations were modeled as pressures at model boundaries (e.g., at the bottom of basement excavations). For deep foundations, a sensitivity analysis was performed, modeling the deep foundations using forces applied within the model grid versus modeling structural elements interfaced to the soil mesh. While using structural elements can produce more accurate load distributions, it requires more modeling effort and assumptions. The sensitivity analysis showed that for a structure on piles, the differences in soil loads on the liner between the two methods were negligible. Thus, the simplified force method was used for the liner design.

Grout. Sensitivity analyses performed using estimated grout properties resulted in significantly reduced soil loads on the liner compared to modeling the grout with properties similar to the adjacent soil. The grout filling the gap between the outside diameter of the liner and the excavation was conservatively modeled based on the strength and stiffness of the adjacent soil.

Modeling in 2-D. The majority of the geotechnical numerical modeling for the liner design was performed in 2-D (FLAC, Itasca 2011). While a 2-D geotechnical model cannot explicitly account for the various contributions to volume loss and ground relaxation due to tunnel construction, the bulk of the modeling was performed in 2-D to efficiently evaluate the design cases using simplifying, conservative assumptions regarding ground relaxation as described below in "Excavation relaxation." Limited 3-D modeling was performed at critical design sections and compared favorably with the final 2-D model results.

Excavation relaxation. The soil loads on the liner are dependent on the volume loss and the resulting soil relaxation/arching that occurs due to tunnel excavation and construction. The excavation relaxation was modeled in 2-D by reducing *in situ* stresses at the perimeter of the tunnel excavation and monitoring the volume loss at the tunnel level (i.e., percentage volume/foot of contraction relative to the excavation volume/foot).

The final volume loss at the tunnel level was conservatively assumed to be equal to the empirically estimated volume loss at the ground surface. In shallow sections and where normally consolidated soil constitutes a significant thickness of the overburden, the final volume loss at the tunnel level and at the ground surface will likely be similar. However, in deeper sections and where overconsolidated soil constitutes a significant thickness of the overburden, the final volume loss at the tunnel level will be greater than at the ground surface as a result of bulking/dilation and arching. Since soil loads on the liner will decrease due to arching as volume loss increases, assuming a lower volume loss was conservative.

For deep sections, preliminary models assuming the final volume loss at the tunnel level was the same as the volume loss empirically estimated at the ground surface, which was as low as 0.2%, yielded an equivalent overburden soil load as high as 4.3 tunnel diameters. This high of a soil load appeared to be overly conservative when compared to closed-form solutions and typical equivalent overburden loads for tunnels (i.e., less than 2.5 tunnel diameters). For select deep sections, 3-D modeling of the full tunnel excavation and construction procedures was performed to provide a better estimate of the final volume loss at the tunnel level.

The design values of final volume loss at the tunnel level used in the 2-D geotechnical models generally ranged from approximately 0.5% in deeper sections to 1% in shallower sections.

Soil loads. The estimated soil loads ranged from an equivalent overburden of approximately 0.2 tunnel diameters in very shallow sections to 2.0 tunnel diameters in deep sections. Groundwater pressures at the tunnel invert varied from 0.1 to 5.2 bars. The 335 kPa (7,000 psf) future building surcharge case often resulted in higher soil loads on the liner than the existing buildings and structures cases, and the increase in loads was particularly significant for shallower sections (see Figure 4).

Sensitivity analyses were performed to evaluate the potential effects of encountering highly sheared/slickensided soils in ESU 7 and varying lateral earth pressures of the glacially overridden soil, but these conservative checks were not critical for the liner design relative to the envelope of loading conditions along the alignment.

Soil springs. The soil springs generated for use in the structural engineering models were also dependent on volume loss because a certain amount of volume loss and soil relaxation/arching occurs before installing the liner. This was accounted for by first relaxing the tunnel excavation consistent with target volume loss at the tunnel, then generating the soil springs by increasing/decreasing the model forces at the perimeter of the excavation and monitoring displacements. As previously discussed, the geotechnical models use a linear modulus; however, the soil springs generated are non-linear due to plasticity effects. Figure 5 shows an example of radial soil springs demonstrating the non-linear nature due to plasticity, as well as the non-symmetric soil spring response for displacement toward, versus away from, the center of the tunnel (note that positive deformation is toward center of tunnel in Figure 5).

Structural Modeling

For structural analysis, the ring was modeled with a series of beam elements whose stiffness is reduced with the Muir-Wood formula to take into account the effects of the

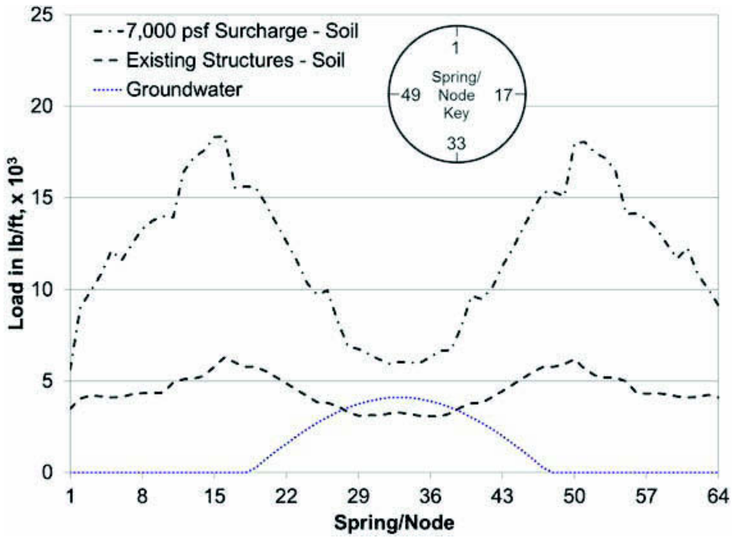

Figure 4. Example soil and groundwater loads on liner for shallow section

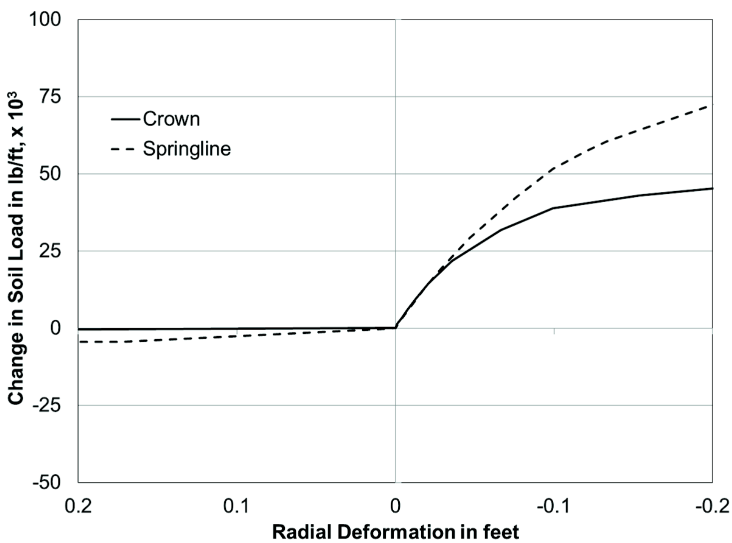

Figure 5. Example of radial soil springs

joints. In the model, the ring elements of the tunnel were connected to the surrounding soil through non-linear springs in both the radial and tangential directions at each of the nodes. The model used radial springs that support only compression and tangential springs that are symmetric in the two directions.

The tunnel liner was analyzed and designed for two conditions. The first condition included only the tunnel ring, representing the scenario at the end of the tunneling operations. The second condition included the completed tunnel during in-service

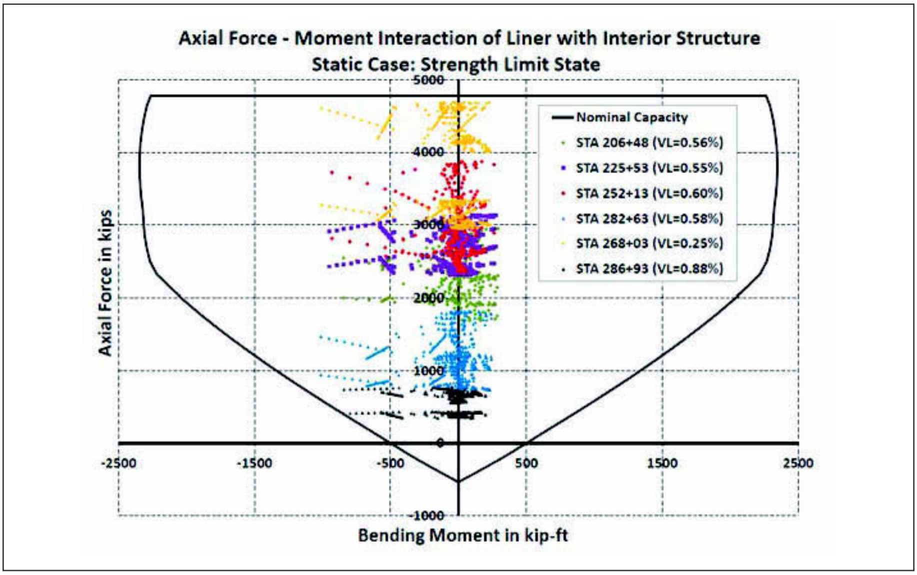

Figure 6. Interaction diagram with load demands at strength limit states

condition, including the tunnel lining, the interior structures, the systems and any associated loads.

The strength design of the liner is in accordance with AASHTO Load and Resistance Factor Design (LRFD) method, which takes into account the statistical variability of member strength and of the magnitude of the applied loads. The load factors in AASHTO have been modified according to the FHWA Manual. From a structural design point of view, the precast liner segments primarily behave as compression and bending members. Since the liner is thin relative to its diameter, compression is evidently more dominant for the liner design considering load combinations in the strength limit states as indicated by the load demands for some of the design sections, which are approaching the upper limit of the capacity interaction diagram (Figure 6). The solid line Figure 6 represents the capacity of the liner.

Seismic Analysis and Design

Analysis of the bored tunnel included loading from seismic deformations and ground accelerations considering three primary modes of deformation during seismic ground movement: (1) ovaling, (2) axial, and (3) curvature deformations.

A two-step analysis procedure similar to that used in the static analysis was adopted to analyze seismic ovaling. First, deformations of the soil surrounding the liner due to the seismic wave propagating from bed rock through soil media, without the liner, were computed with a continuum model. Second, the ground deformations were imposed on the liner through supporting elements (non-linear springs) using beam-on-spring models by performing non-linear dynamic time history analysis. The liner was analyzed for three Expected Earthquake events, and three Rare Earthquake events. The results for the Expected and Rare Earthquakes events were then enveloped, respectively.

The results from the time history analysis show that the maximum ovaling is about 3.8 cm (1.5 in.) or 0.2% of the ring diameter for the Rare Earthquakes. It is observed that maximum ovaling is generally in a diagonal direction, which is consist with the open round cavity deformation caused by a free-field ground shear distortion.

The liner segment gaskets were also evaluated for water tightness under this maximum ovaling. Forces in the liner were also reported to analyze the adequacy of the reinforcement in the liner.

In addition to the 2-D models for studying ovaling effect, a 3-D global spine model of the entire tunnel was created to determine forces and deformations along the longitudinal axis of the liner as well as the seismic movements at each end of the tunnel. Displacement time series in three principal directions were applied to the tunnel structure considering seismic site response, stiffness of the liner and the surrounding soil, and stiffness of adjacent cut-and-cover structure to capture the longitudinal response of the tunnel. The results from the global 3-D seismic analysis controlled the design of shear bicones and circumferential joint bolts to meet the seismic shear demand. Table 3 shows the calculated joint opening and closing from the global spine model at the south end of the tunnel. The values formed the basis for the flexible joint design. Note that there is a significant difference between the movements from the Rare Earthquake and those of the Expected Earthquake. The difference can be attributed to slippage between the liner and the surrounding soil that takes place during a Rare Earthquake.

Table 3. Tunnel movement at south end

Joint Gap	Rare Earthquake, mm (in.)	Expected Earthquake, mm (in.)
Opening	167 (6.6)	3 (0.1)
Closing	218 (8.6)	5 (0.2)

To predict the local behavior of the radial and circumferential joints, a 3-D finite element model of four rings was created. The maximum seismic ovaling deformation from the 2-D model and maximum curvature from the 3-D spine model were applied to the finite element model and the openings of the circumferential and radial joints were then determined. The maximum joint opening was found to be about 3 mm (0.11 in.). Gasket size was then determined for a combined value of opening from static and seismic loads as well as construction tolerances, in addition to the consideration of gasket offset and water pressure.

CONCLUSIONS

SR 99 Bored Tunnel is a significant undertaking due to its size, geologic location, and the site conditions under which the tunnel is built. The following main conclusions can be drawn from the liner design of the SR 99 Bored Tunnel:

1. Simplifying assumptions and sensitivity analyses regarding geotechnical properties and modeling methodology increased the efficiency of geotechnical design analyses to meet contract requirements, given the minimum liner design criteria.
2. Both soil loads on the liner and soil springs generated for soil-structure interaction analyses are affected by volume loss and the resulting soil relaxation that occurs during tunnel construction.
3. Dynamic time history analysis is an invaluable tool to quantify liner seismic deformations and forces, especially seismic movements at joints.
4. Seismic loads and deformations do not govern the size and reinforcement of the liner, except for the size of the shear cones at the circumferential joints and the size of the gasket.
5. The two-step approach is an efficient and sufficiently accurate method for liner design.

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OPENING SUPPORTS TO SEGMENTAL LININGS—A NOVEL SHOTCRETE SUPPORT SOLUTION

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Malcolm Chappell ▪ Halcrow

Matt Burdick ▪ Traylor Frontier-Kemper JV

Michael Krulc ▪ Traylor Frontier-Kemper JV

ABSTRACT

Substantial schedule advantages can be made on projects if cross passage construction can occur at the same time as TBM operation with little or no impact on TBM logistics. However, this approach can prove difficult to achieve with many of the more cost-effective means of supporting the segmental lining during cross passage break-out. This paper discusses the constraints and shortcomings of conventional approaches, and describes how a cost effective solution involving a shotcrete shell and opening constructed inside the segmental was designed and constructed to deliver cost and time savings on Seattle University Link contract U220.

INTRODUCTION

In a world of technology & electronic communication it is easy to forget what engineers do best with a problem: sitting around a table, exchanging views and ideas with sketches, and respecting everyone's view. This was the approach adopted in this project to arrive at a solution for the cross passage opening support.

Schedule and space constraints usually impede concurrent cross passage excavation and TBM operation, and where the space is constrained, opening support solutions that maximize available cross section are of great benefit. While there are a number of solutions that minimize space, some of these solutions can require expensive steelwork and extensive drilling and fixing. However, following a detailed round the table discussion of the options, and constraints it was identified that a shotcrete shell would not only limit obstruction of the space during installation, but also permit easy installation, particularly around services, and avoid costly steelwork and drilling and fixing.

BACKGROUND

University Link Extension is a 5.07 km (3.15 mile) extension of the existing Central Puget Sound Regional Transit Authority (Sound Transit) light rail system that runs in twin-bored tunnels from downtown Seattle north to the University of Washington, with stations at Capitol Hill and on the University of Washington campus near Husky Stadium. University Link will serve the three largest urban centers in the State of Washington—downtown Seattle, Capitol Hill and the University District. By 2030, the University Link line alone is projected to add 70,000 boardings a day to the light rail system.

Contract U220 comprises site preparation, slurry wall construction and partial excavation of the University of Washington Station box and excavating the 3.48 km (2.16 miles) of twin-bored 6.55 m (21'-6") excavated diameter tunnels from the

University of Washington Station to Capitol Hill Station as well as construction of multiple cross passages between the twin-bored tunnels, permanent invert and walkway concrete, wet standpipe and permanent electrical installation in the running tunnels. The running tunnel sections were excavated with earth pressure balance TBMs and had the final lining constructed during excavation using bolted and gasketed precast concrete segments.

On March 25, 2009 a joint venture of Traylor Bros. Inc., and Frontier-Kemper Constructors (TFK JV), both of which are headquartered in Evansville, IN, submitted the lower of two bids received at $309,175,274. This was significantly lower than the $354,000,000 bid price offered by the second bidder and the published Engineer's Estimate ($395,000,000).

Construction management was provided to Sound Transit by a joint venture of CH2M Hill and Jacobs Engineers. Engineering design was provided by a joint venture of Jacobs Associates, HNTB and Earth Tech, including crosspassage design, initial lining design, and a potential method for providing temporary support to the TBM tunnel segmental lining around openings cut for access into the crosspassages. The offered support option relied upon a series of rolled steel beams and braces (commonly referred to as Hamster Cages) that are erected inside the tunnel and rigidly blocked against the intrados of the segmental lining. TFK hired Halcrow Inc, who also designed the segmental lining for TFK, to provide design services for the temporary propping system.

Schedule

The Contract allotted a total of 41 months from Notice to Proceed to Substantial Completion for the U220 project. Allowing 15 months for site preparation, station box slurry wall construction, station box excavation and invert concrete construction, and a further 13 months for TBM mining and all cross passage and tunnel finishing work. With 16 crosspassages to complete, this would not be possible unless crosspassage work started before completion of TBM mining. Thus, TFK decided that crosspassage excavation and TBM mining would have to occur simultaneously in the same tunnel to achieve the required substantial completion deadline. Additionally, two cross passages would have to be excavated simultaneously while several others would be undergoing waterproofing installation and final lining construction.

This was a new approach for the TFK team and the problems that needed to be overcome to make it happen were numerous and challenging. However, it was very clear that the driving tenet during cross passage planning would be "Do not stop the TBM," which created many of the challenges that had to be overcome.

ALIGNMENT AND GEOLOGY

The tunnel route comprises approximately 3350 m (11,000 ft) of twin bore tunnel. The depth at the University of Washington Station is approximately 23 m (77 ft), and approximately 14 m (45 ft) at the Capitol Hill Station. The shallowest point in the tunnel has only 4 m (13 ft) of cover below the Lake Washington Ship Canal (Montlake Cut), and a deepest point of approximately 95 m (310 ft).

The soils encountered on the tunnel route comprise glacial and non-glacial deposits, the majority of which have been overconsolidated by glaciation. The deposits consist of clays, silts, sands and gravels in varying proportions. Some of the layers form aquacludes, leading to a somewhat complex groundwater profile. Water pressures on the lining vary from a minimum of 1 bar at crown, to a maximum of 6.5 bar at invert.

GENERAL LAYOUT

The TBM tunnel lining is a conventionally reinforced precast concrete lining, 5.74 m (226 in) internal diameter and 267 mm (10.5 in) thick, with a ring length of 1.52 m (60 in) comprising 5 segments plus a key, arranged in a trapezoid/parallelogram arrangement. Segments are bolted with spear bolts on the radial joints and push-fit dowels on the circumferential joints. Separate up and down rings are provided to assist in the negotiation of vertical and horizontal curves while maintaining the counter key segment below axis as much as possible.

There are 16 crosspassages along the route at approximately 230 m (750 ft) centers. Typical cross passages have a maximum width of excavation of around 3.75 m (12 ft 4 in), while two larger passages, for a sump and interconnection of Traction Electric conduit go up to 5.1 m (16 ft 8 in). All of the crosspassages require an opening two rings wide, within which a permanent opening support is cast. The opening requires temporary support from break-out until the permanent concrete has achieved its specified strength. The contract specifies that the temporary opening support to be the contractor's design.

A number of temporary services are installed along the length of the TBM tunnel as shown in the typical tunnel cross section below in Figure 1 and need to remain in almost continuous operation during cross passage excavation, except for short (less than one hour) interruptions in preparation for temporary support installation.

The design of the opening support also had to allow for the installation of pre support at each cross passage, consisting of a series of spiles over the top of the cross passage excavation profile.

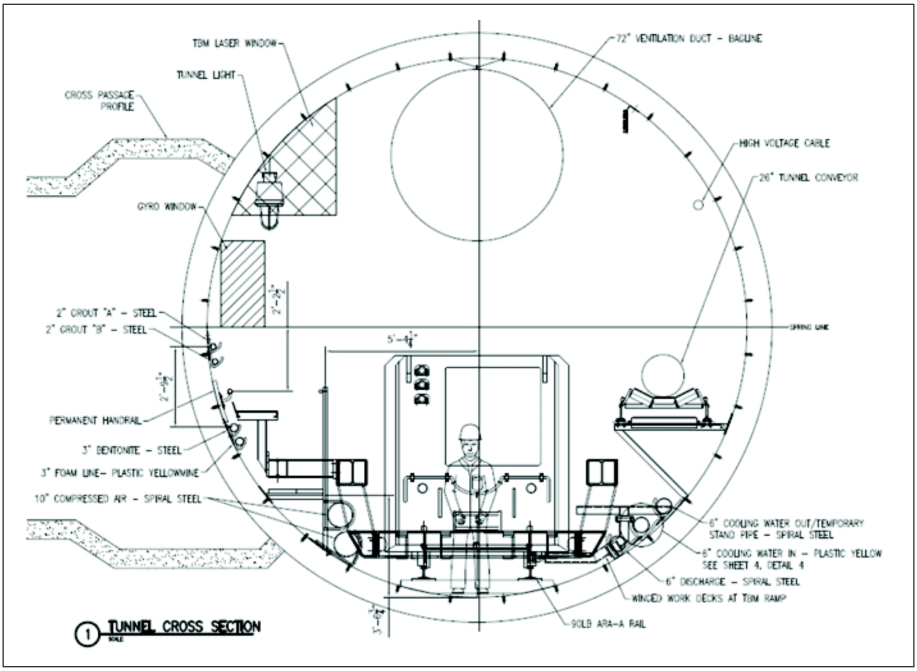

Figure 1. Typical tunnel cross section

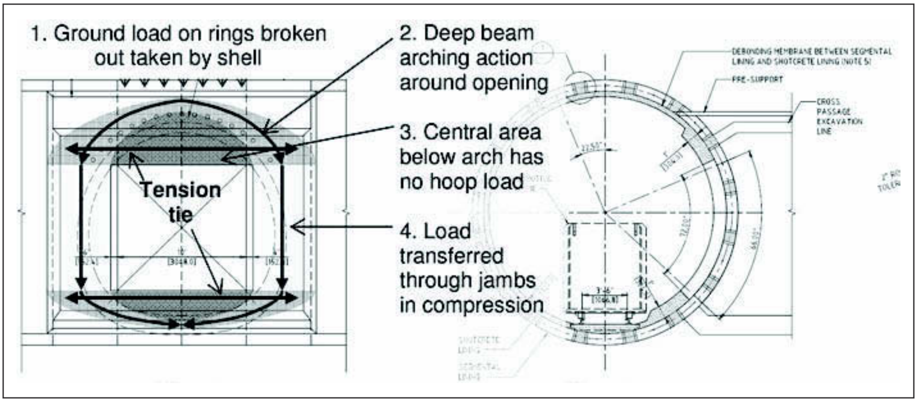

Figure 2. Shotcrete support option with indicative structural behavior

OPTIONS AND DECISIONS

Temporary propping design had to accommodate continuous supply of the TBM with segments, utility pipe and rail sections and removal of excavated material via belt conveyor, by ensuring that the temporary support system did not conflict with the previously installed utility systems. The hamster cages were eliminated from consideration almost immediately as their use presented numerous conflicts with the existing utilities, conveyor structure and the requirement to maintain continuous train traffic in the tunnel. After eliminating the hamster cage option, Halcrow started looking at the steel door frame in detail. Early evolutions of the door frame design were very heavy and required a high quantity of large diameter anchors to be drilled into the segmental lining around the opening. While technically feasible, the cost to fabricate ten complete cross passages worth of support was prohibitive as were the costs associated with the installation and removal of the large diameter anchors. It was at this stage that the designers and contractors sat down together to try to identify a better solution. Following extensive discussion of the constraints of the problem and possible ways around them, the discussion yielded the third, and ultimately the chosen temporary support system, which relied on a reinforced shotcrete shell placed inside the segmental lining around the cross passage opening. A similar system had been successfully implemented on a project in the UK several years before and appeared to be appropriate for our situation.

Shotcrete Shell Option

The shotcrete shell concept is similar to the shotcrete opening support that would be provided for a cross passage opening in the SCL lining for a conventionally mined tunnel, except that it supports the two rings of the broken out lining. The concept is illustrated in Figure 2.

This solution offers a number of significant benefits over the jamb and lintel solution:

- The shotcrete is designed only for temporary loads and is therefore much thinner and more flexible than the segmental lining.
- The concrete is predominately in compression, so reinforcement levels are low (except around the opening).
- Service relocation limited to blocking out existing lines by 6 inches.
- No relocation of the conveyor was required, and could be protected from the installation with a plastic sheet.

- The relatively low cost of the propping system enabled TFK to have propping in place at every cross passage in both tunnels at one time.

DESIGN

The design of a novel solution such as this needs careful consideration to ensure that the structural behavior is understood and correctly accounted for in the design. The first step is to understand how the segmental lining is loaded by the ground. Prior to cross passage breakout the shell only supports self weight. No deformation of the permanent lining is expected in the short term (prior to breakout), so no load will be exerted on the shell. Some shrinkage may lead to a small gap between lining and shell.

Once the rings are broken out, hoop force in those rings is removed and all ground load on those rings will be transmitted directly through the segments onto the 127 mm (5 in) shell inside. The shell then acts as a hoop, transferring hoop around the ring, with load transfer around the opening being provided in a 305 mm (12 in) thickened section.

Loading

The magnitude of the loading might, at first glance, appear straightforward. This kind of opening is often designed on the basis of the pressures required to ensure a stable tunnel, such as those recommended by Terzaghi, which are easy to calculate and design for. However, these pressures were primarily derived from field observations of conventional tunneling using steel sets, which allow significant relaxation of the ground—and hence reduction in load—prior to installation of the lining. Modern TBMs are specifically designed to limit this movement in order to control settlement, so there is potential for higher loads to be present in the lining. The authors are aware of instances of the grout load pressures essentially becoming 'locked in' to the lining. Furthermore, the presence of cohesive ground also increases the risk of loads significantly above those that would be predicted by Terzaghi in the medium term, which presents further risk if the cross passage excavation occurs significantly behind the TBM.

To address these risks, the design assessed how much movement of the lining would be required to alleviate the load to manageable levels. Using an axisymmetrical finite element model of the tunnel, it was shown that the load would reduce to manageable levels with relatively small inward displacements of the order expected with the shell (ignoring the beneficial effects of shrinkage), but that further movement would provide much less further alleviation of load.

Design

With reference to Figure 2, it can be seen that the design can be broken into a number of component parts that each lend themselves to simple analysis by hand calculation, as presented in Table 1.

Due to the rather novel nature of the design, additional analysis was undertaken using a 3D plate model to verify that all behavior of the system was adequately understood. The finite element model was generally within 10% of the hand calculations (see Table 2), which was considered to be very good agreement. In particular, the deep beam behavior of the lintel and sill area was clearly visible in the model, including the slab action of the area with no hoop load. Nevertheless, the modeling did pick up a number of minor unforeseen effects as follows:

- Torsion in the jambs. This arises because the inside of the jambs is not supported by the soil. This torsion was within capacity of the section and was verified by hand calculation.
- Longitudinal bending moments between the loaded and unloaded sections of the thin section. These arose in the model because the loaded sections

Table 1. Methods of analysis

Component	Method of Analysis	Reinforcement
Thin shotcrete ring	Closed form analysis for tunnels in soft ground, horizontal load 50% of vertical	#5 bars at 203 mm (8 inches) spacing (1 layer, centrally placed)
The lintel and sill deep beam	Strut and tie model (as per deep beam theory), plus moments and shears from the section above the lintel with no hoop load	5 #10 bars each face main steel, chained #4 ties at 127 mm (5 inch) spacing
Section above lintel with no hoop load	Designed as a slab with the thickened section of lintel acting as an edge beam	
Jambs	Curved columns with uniform horizontal UDL as per normal jambs	6 #4 bars each face main steel, #4 ties at 254 mm (10 inch) spacing

Table 2. Differences between hand calculations and FE model

Value	Hand Calculation	Model	Difference
Maximum moment in shell	31 kNm/m	34 kNm/m	10%
Lintel moment	181 kNm/m	190 kNm/m	5%
Lintel tension	1982 kN/m	1947 kN/m	2%
Jamb compression	1982 kN/m	1892 kN/m	5%

were in compression and moved in, while the unloaded sections inside the intact rings did not move in. There was some doubt as to whether these would occur in reality due to the small relative displacements and effects of cracking. Nevertheless they were checked and found to be within capacity.

The conclusion of the modeling was that the hand calculations would result in a design that is robust without being conservative. It also identified the need to undertake a torsion check on the jambs when the jambs are only partially supported by the ground behind. However, it is the authors' view that finite element modeling would demonstrate lower levels of torsion, and is therefore recommended if hand calculations were to indicate that additional torsional reinforcement was required.

CONSTRUCTION

Pre-Work

Installation methodology evolved as TFK's crew became more familiar with the process and were able to refine it. Propping installation boiled down to three basic steps:

1. Segment preparation
2. Reinforcing steel and formwork installation
3. Shotcrete placement

Perhaps the biggest concern about the shotcrete propping system was the impact that a shotcrete shell might have on the finished surface of the TBM tunnel lining; but the shotcrete also had to adhere to the lining during spraying while still being easy to remove. TFK performed a series of tests on a small mockup section of tunnel lining to confirm the suitability of a number of different methods to cover the segment joints and bolt pockets including backer rod, custom cut foam pieces, various tapes and thin plywood sections, and multiple bond breaker products and installation procedures. Shotcrete was applied over the mockup and allowed to cure prior to removal. While

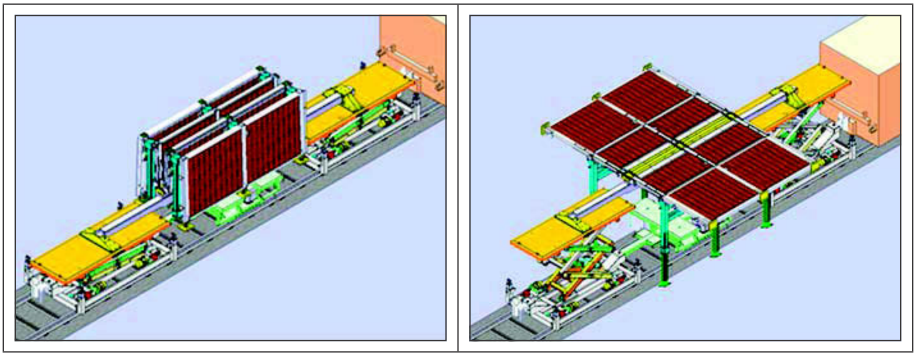

Figure 3. Scissor car with work deck in transport position and with work deck deployed

actual shotcrete removal from the mockup proved to be significantly more difficult than anticipated, it did result in a clean final lining surface. The solution settled upon relied on 3 mm (⅛") plywood sections nailed over the bolt pockets and a double layer of bondbreaker applied over the full area of the propping and a single layer applied one ring either side of the propping to ease removal of overspray. It was determined that excessive adhesion to the radial and longitudinal joints was not a significant concern.

Equipment

In order to make use of the unique propping solution and progress cross passage excavation concurrent with TBM mining, a custom folding work deck and a scissor-car-based work deck delivery system were devised and fabricated. The work decks and scissor car were designed by Kelley Engineered Equipment (KEE) of Omaha, NE. KEE also fabricated the scissor car while Traylor Bros., Inc. equipment shop in Evansville, IN fabricated the work decks. The decks are illustrated in Figure 3.

Work Sequence

TBM mining was conducted 24 hours per day, five days per week with maintenance performed on the weekends, while cross passage works were 24 hours per day, six days per week. The initial plan was to commence cross passage excavation from the Southbound tunnel after the trailing gear of the Northbound TBM (which was trailing Southbound by one month) had cleared the third cross passage along the alignment,; roughly four months after the start of TBM mining. In reality, it took much longer to get ready for cross passage excavation than anticipated and thus true excavation did not begin until seven months after the start of TBM mining.

Throughout the propping installation process, the basic reinforcing steel detail remained unchanged—the radial and longitudinal steel in the thinner shell section was shipped loose and installed piece by piece while the reinforcement for the thickened section around the cross passage opening was separated into four cages that were pre-assembled off site and set in place. A series of hangers and slab bolsters were utilized to provide appropriate clear cover against the segmental lining and adequate support to hang the cages. The completed reinforcement, platform, and services are illustrated in Figure 4.

In order to provide shotcrete at relatively short notice and avoid obstructing the tunnel, the dry mix method was required. Dry mix shotcrete was delivered to the job site in one cubic yard super sacks and transported to the cross passage work via flat car and locomotive, before being placed by hand to the required profile as shown in Figure 5.

Coordination with Cross Passage Pre-Support Requirements (Including Dewatering)

The flexibility of the opening support being shotcrete was realized in the following:

- Pre-support spiles installed through the segmental lining over the crown of the cross passage, which were installed prior to the installation of propping to avoid conflicts between the propping reinforcing steel and the spiles. This is illustrated in Figure 6.
- Dewatering wells which were routed through the propping reinforcing cages and shotcrete without causing interference.
- Post-installed dewatering wells, which required core drilling a six inch diameter holes through the propping sill beam and reinforcing cage.

Figure 4. Completed reinforcing and formwork

Figure 5. Placing dry mix shotcrete in crown

Figure 6. CP 19 pre support spiles above lintel cage

Movements and Monitoring Results

Overall the shotcrete propping performed admirably and did not exhibit large movements or deformations. In a limited number of instances total movement approached the trigger levels identified by Halcrow in the design documents but never reached a level of real concern.

Method of Demolition

The project schedule required that temporary propping be in place in tunnels and at all sixteen cross passages until very late in the job, requiring an efficient demolition methodology be developed and implemented. This exercise was made more difficult due to the unknowns in how the shotcrete would behave during the demolition process. Despite this complexity, an effective methodology and sequence were arrived upon during the second demolition attempt. The primary demolition tool is a Gradall XL 4300 armed with a 2,500 ft-lb hydraulic demolition hammer. A second excavator (Cat 304), with a hydraulic hammer, bucket and thumb was also utilized to assist with the breaking operations and perform debris load out, with an excavator situated on either side of the cross passage and appropriate protective devices in place around the utilities.

The demolition procedure begins by breaking out a four foot wide swath of the shell section along the full 20 foot length of the crown. This creates relief and allows the remaining demolition to occur rapidly. The excavators next break the bond between the shell and segmental lining shell section on the non-cross passage side of the tunnel and then pull the shell section off the wall in several large pieces, as illustrated in Figure 7. The large pieces are processed, sorted and loaded out before additional demolition occurs. Next, a relief cut is made across the columns on the cross passage side roughly two feet below the lintel cage, after which the large machine breaks the lintel cage free from the segmental lining and it falls off in one large section, as illustrated in Figure 8. The two columns are broken down traditionally to the top of the sill beam, and the sill beam and invert portion of the propping are removed in a follow-on operation when there is good access for rapid completion of the invert concrete across the propping width.

A follow-on finishing operation is required to remove overspray, plywood covers and miscellaneous anchors and repair any surface damage inflicted during the demolition process. This work is performed by hand and is accessed off scissor lifts working on the tunnel invert.

Figure 7. Lintel beam removal

Figure 8. Conveyor side shell removal

CONCLUSION

The opening in the shotcrete shell behaves as a deep beam above and below the opening and, while a number of effects need to be considered in design, hand calculations will usually suffice. Finite element modeling is only likely to offer savings in reinforcement where torsion in the jambs is an issue.

Use of the shotcrete shell approach to temporarily support around cross passage openings permitted TFK to start cross passage work earlier and concurrently execute the work on more fronts than would have been possible with a more traditional method of temporary support. Without these advantages the project could not have been completed within the tight schedule demands outlined in the contract.

Finally the project highlights the inescapable value of focused face-to-face discussion without which it is doubtful if the solution described would have come to light.

SEGMENTAL LINING DESIGN FOR LARGE-DIAMETER ROAD TUNNELS

Harry Asche ▪ Aurecon

Tom Ireland ▪ Aurecon

ABSTRACT

The use of segmental linings for tunneling works is increasingly being preferred to other forms of tunneling, and the diameter of these tunnels is also increasing due to advances in TBM capability. With mechanization, tunneling costs are reducing, resulting in larger segmentally lined tunnels, now in particular for roads. Road tunnels generally have a much lower cover to diameter ratio than metro tunnels.

Many of the current design approaches have been developed for smaller diameter service or metro tunnels, and these methods are being scaled up without, in all cases, the consideration of what design approaches are scalable such as the aspects of segmental lining detailing that require special consideration at larger diameters. In the English-speaking world of tunnel design, the most commonly used design approach is due to Muir Wood (1975) and Curtis (1976). Curtis's approach assumes the combination of uniform and biaxial stress state, but ignores the gradient of a gravitational stress field: the stresses, displacements, thrusts and moments are identical at crown and invert. This simplifying assumption can introduce significant errors for large road tunnels at low cover. Hartmann (1970 and 1985) published a solution which accounts for the gravitational stress field, and this paper demonstrates this effect on lining actions and compares them to FE solutions.

The authors have been involved design reviews of the 3 Brisbane Road tunnels, and in the verification of the 14m OD 3-lane road tunnel in Auckland NZ. These designs are used to demonstrate some issues with the adoption of traditional analysis methods, and some design recommendations are made.

INTRODUCTION

The use of segmental linings for tunneling works is increasingly being preferred to other forms of tunneling, and the diameter of these tunnels is also increasing due to advances in TBM capability. With mechanization, tunneling costs are reducing, resulting in larger segmentally lined tunnels, now in particular for roads. Road tunnels generally have a much lower cover to diameter ratio than metro tunnels.

Many of the current design approaches have been developed for smaller diameter service or metro tunnels, and these methods are being scaled up without, in all cases, the consideration of what design approaches are scalable such as the aspects of segmental lining detailing that require special consideration at larger diameters. In the English-speaking world of tunnel design, the most commonly used design approach is due to Muir Wood (1975) and Curtis (1976). Curtis's approach assumes the combination of uniform and biaxial stress state, but ignores the gradient of a gravitational stress field: the stresses, displacements, thrusts and moments are identical at crown and invert. This simplifying assumption can introduce significant errors for large road tunnels at low cover. Hartmann (1970 and 1972) published a solution which accounts for

Table 1. Symbols used and case study values

Quantity	Symbol	Unit	Case 1	Case 2
Depth to tunnel axis	z	m	16m	44m
Radius of tunnel excavation	R_2	m	7m	7m
Thickness of lining	H	m	0.45m	0.45m
Second moment inertia of lining	I	m^3	$0.00759m^3$	$0.00759m^3$
Young's modulus of lining	E_l	MPa	16700MPa	16700MPa
Poisson's ratio of lining	ν_l	-	0	0
Young's modulus of ground	E_g	MPa	70MPa	130MPa
Poisson's ratio of ground	ν_g	-	0.3	0.3
Ratio of horizontal to vertical insitu stress	K_o	-	0.35	0.7
Unit weight	γ	MPa/m^3	$0.0185MPa/m^3$	$0.0185MPa/m^3$

the gravitational stress field, and this paper demonstrates this effect on lining actions and compares them to FE solutions.

A case study from the Waterview Connection road tunnel in Auckland is used to demonstrate some of the limitations with the commonly used methods.

BACKGROUND

Theories of Ground Load on Circular Tunnels

The general solution for two dimensional problems in polar coordinates was first given by Michell (1899) and this solution is presented and discussed in detail by Timoshenko and Goodier (1951).

To model linings, a thin ring is often assumed. Flügge (1973) discusses circular elastic thin rings and gives a set of solutions commonly used.

In the particular application to tunnels, Duddeck and Erdmann (1982) present a survey of the various approaches and publications. In the English-speaking world of tunnel design, the most commonly used is due to Curtis (1976), which Duddeck and Edrmann point out is the same as achieved independently in the previous decade by Norwegian and German authors.

Curtis's approach assumes the combination of uniform and biaxial stress state, but ignores the gradient of a gravitational stress field: the stresses, displacements, thrusts and moments are identical at crown and invert.

Hartmann (1970 and 1985) published a solution which accounts for the gravitational stress field, by including $\cos(\theta)/\sin(\theta)$ and $\cos(3\theta)/\sin(3\theta)$ terms. This solution (like Curtis) assumes an infinite elastic material. The complete effect of a free surface at the ground level is not accounted for in Hartmann. Mindlin (1940) and Verruijt and Booker (1996) account for this effect but only for the case of unlined tunnels.

This paper presents a description of the Hartmann derivation, and in doing so, corrects an error in the Hartmann solution.

HARTMANN DERIVATION

The section describes the basis for the Hartmann derivation with the symbols used defined in Table 1, along with the parameters used in the case study presented in Figures 1 to 8.

Curtis and similar authors assume an infinite cylinder of ground with uniform and biaxial stresses applied, and this stress field is expressed in polar coordinates as follows:

$$\sigma_r(r,\theta) = \frac{1}{2}\gamma\ z(1 + K_0) + \frac{1}{2}\ \gamma\ z(1 - K_0)\cos(2\theta)$$

$$\sigma_\theta(r,\theta) = \frac{1}{2}\ \gamma\ z(1 + K_0) - \frac{1}{2}\ \gamma\ z(1 - K_0)\cos(2\theta)$$

$$\tau_{r\theta}(r,\theta) = -\frac{1}{2}\ \gamma\ z(1 - K_0)\sin(2\theta)$$

Hartmann assumes that the stress field includes a gravitational gradient:

$$\sigma_r(r,\theta) = \frac{1}{2}\gamma z(1 + K_0) - \frac{1}{4}\gamma r(3 + K_0)\cos(\theta) + \frac{1}{2}\gamma z(1 - K_0)\cos(2\theta) - \frac{1}{4}\gamma r(1 - K_0)\cos(3\theta)$$

$$\sigma_\theta(r,\theta) = \frac{1}{2}\gamma z(1 + K_0) - \frac{1}{4}\gamma r(1 + 3K_0)\cos(\theta) - \frac{1}{2}\gamma z(1 - K_0)\cos(2\theta) + \frac{1}{4}\gamma r(1 - K_0)\cos(3\theta)$$

$$\tau_{r\theta}(r,\theta) = -\frac{1}{4}\gamma r(1 - K_0)\sin(\theta) + \frac{1}{2}\gamma z(1 - K_0)\sin(2\theta) - \frac{1}{4}\gamma r(1 - K_0)\sin(3\theta)$$

The solution for the case posed by Hartmann is given in brief below. An error in the deflection calculation is corrected. Each of the stress field components can be treated separately, that is, the uniform case, the case of cos(θ)/sin(θ), of cos(2θ)/sin(2θ), and that of cos(3θ)/sin(3θ).

The loads to be shared between the ground and the lining are radial and tangential vectors except that the uniform case is only radial.

$$p_u = \frac{1}{2}\gamma z(1 + K_0)$$

$$p_1 = \begin{bmatrix} -\frac{1}{4}\gamma R_2(3 + K_0) \\ -\frac{1}{4}\gamma R_2(1 - K_0) \end{bmatrix}$$

$$p_2 = \begin{bmatrix} \frac{1}{2}\gamma z(1 - K_0) \\ \frac{1}{2}\gamma z(1 - K_0) \end{bmatrix}$$

$$p_3 = \begin{bmatrix} -\frac{1}{4}\gamma R_2(1 - K_0) \\ -\frac{1}{4}\gamma R_2(1 - K_0) \end{bmatrix}$$

The ground and the lining have stiffness matrices relating the radial and tangential tractions to corresponding radial and tangential deflections as shown in Table 2. The uniform case has radial tractions and deflections only. The case of cos(θ)/sin(θ) is special and is treated differently. The tractions on the lining must be self-equilibrating which means that the radial and tangential tractions must be equal and opposite. This gives rise to a peculiar set of forces on the lining which generate tangential deflections only and no bending.

The tractions on the lining are as follows:

$$S_{n.lu} = p_u \frac{X}{X + U}$$

Table 2. Stiffness derivation

Stiffness	Ground	Lining
Uniform	$X = R_2 \dfrac{1+\nu_g}{E_g}$	$U = R_2^2 \dfrac{1-\nu_l^2}{E_l H}$
cos(θ)/sin(θ)	$\mathbf{Y} = \dfrac{R_2}{8} \dfrac{1+\nu_g}{E_g(1-\nu_g)} \begin{bmatrix} 1-2\nu_g & -(3-2\nu_g) \\ -(3-2\nu_g) & 1-2\nu_g \end{bmatrix}$	$V = R_2^2 \dfrac{1-\nu_l^2}{E_l H}$
cos(2θ)/sin(2θ)	$\mathbf{A} = \dfrac{R_2}{3} \dfrac{1+\nu_g}{E_g} \begin{bmatrix} 5-6\nu_g & 4-6\nu_g \\ 4-6\nu_g & 5-6\nu_g \end{bmatrix}$	$\mathbf{C} = \dfrac{R_2^4}{36} \dfrac{1-\nu_l^2}{E_l I} \begin{bmatrix} 4 & 2 \\ 2 & 1 \end{bmatrix} + \dfrac{R_2^2}{6} \dfrac{1-\nu_l^2}{E_l H} \begin{bmatrix} 0 & 0 \\ 1 & 2 \end{bmatrix}$
cos(3θ)/sin(3θ)	$\mathbf{B} = \dfrac{R_2}{8} \dfrac{1+\nu_g}{E_g} \begin{bmatrix} 7-8\nu_g & 5-8\nu_g \\ 5-8\nu_g & 7-8\nu_g \end{bmatrix}$	$\mathbf{D} = \dfrac{R_2^4}{576} \dfrac{1-\nu_l^2}{E_l I} \begin{bmatrix} 9 & 3 \\ 3 & 1 \end{bmatrix} + \dfrac{R_2^2}{24} \dfrac{1-\nu_l^2}{E_l H} \begin{bmatrix} 0 & 0 \\ 1 & 3 \end{bmatrix}$

$$S_{t.l1} = -S_{n.l1} = \frac{1}{4}\gamma R_2 \frac{(1-K_0)(Y_{2,2}-Y_{1,2})+(3+K_0)(Y_{2,1}-Y_{1,1})}{V+(Y_{2,2}-Y_{1,2})-(Y_{2,1}-Y_{1,1})}$$

$$\begin{bmatrix} S_{n.l2} \\ S_{t.l2} \end{bmatrix} = [[\mathbf{A}+\mathbf{C}]^{-1} \cdot \mathbf{A}]\boldsymbol{p}_2$$

$$\begin{bmatrix} S_{n.l3} \\ S_{t.l3} \end{bmatrix} = [[\mathbf{B}+\mathbf{D}]^{-1} \cdot \mathbf{B}]\boldsymbol{p}_3$$

The deflections are calculated as follows, noting that the case of cos(θ)/sin(θ) includes an infinite deflection, resolved by measuring with respect to a far point at R_3:

$$u_{lu} = U \cdot S_{lu}$$

$$\begin{bmatrix} u_{l1} \\ v_{l1} \end{bmatrix} = \begin{bmatrix} \delta_{wb} \\ \delta_{wb} + V \cdot S_{t.l1} \end{bmatrix}$$

$$\delta_{wb} = (Y_1+K)S_{n.g1} + (Y_2+K)S_{t.g1}$$

$$K = -\frac{R_2}{4} \frac{1+\nu_g}{E_g} \frac{3-4\nu_g}{1-\nu_g} \ln\left(\frac{R_2}{R_3}\right)$$

$$\begin{bmatrix} u_{l2} \\ v_{l2} \end{bmatrix} = \mathbf{C} \begin{bmatrix} S_{n.l2} \\ S_{t.l2} \end{bmatrix}$$

$$\begin{bmatrix} u_{l3} \\ v_{l3} \end{bmatrix} = \mathbf{D} \begin{bmatrix} S_{n.l3} \\ S_{t.l3} \end{bmatrix}$$

The actions are calculated as follows:

$$N(\theta) = S_{n.lu}R_2 - S_{t.l1}R_2\cos(\theta) - (S_{n.l2}+2S_{t.l2})\frac{R_2}{3}\cos(2\theta) - (S_{n.l3}+3S_{t.l3})\frac{R_2}{8}\cos(3\theta)$$

$$M(\theta) = M_u + \left(S_{n.l2} + \frac{S_{t.l2}}{2}\right)\frac{R_2^2}{3}\sin(2\theta) + \left(S_{n.l3} + \frac{S_{t.l3}}{3}\right)\frac{R_2^2}{8}\sin(3\theta)$$

where:

$$M_u = \frac{E_l I}{1 - \nu_l^2} \frac{U \cdot S_{n.lu}}{R_2^2}$$

For comparison with biaxial solutions such as Curtis (1976), set the terms of cos(θ) and cos(3θ)/sin(3θ) to zero. Note also that Curtis (1976) does not include the term for M_u in the calculation for moment.

CURRENT DESIGN PRACTICE

Overview

Current design practice often involves analysis of the ring using a range of different methodologies including closed form solutions, FE/FD analysis to determine the ground loading applied to the ring and finally beam spring models utilising software such as Strand7 to analyse various other special load cases.

A typical design approach is summarized in Table 3. A tick indicates a method that is often used by design practitioners. No tick does not indicate that the method cannot be used for the relevant load case, but that to do so is more complicated.

Limitations of Closed Form Solutions

The closed form solutions are often applied with the Muir Wood (1975) empirical ring stiffness equation used to calculate the moments. As acknowledged by Muir Wood (2000) in his later work, this approach does not allow for the stiffening effect of axial load and can result in some very large moments where axial force is low. This problem is avoided by explicitly calculating the moments resulting from joint rotation. A methodology for this was outlined in Ireland and Asche (2011).

The other limitation to Curtis's approach is assuming the combination of uniform and biaxial stress state, but ignoring the gradient of a gravitational stress field: the stresses, displacements, thrusts and moments are identical at crown and invert. This is particularly relevant to large diameter tunnels at low cover.

Limitations of Finite Element Analysis

The development of widely available and user friendly finite element and finite difference codes (e.g., Phases, Plaxis, FLAC) has allowed a generation of tunnel designers to dispense with closed form solutions, but the fundamental questions remain. Assuming an elastic ground or even an elasto-plastic ground gives very small loads to a tunnel lining installed some distance from the face. In soft ground, we know that significant pressures slowly build up. Most users of these programs apply a face release factor which gives a significant load, even though there is no rational guidance for selecting this number in the model. The model is predicting that the stress on the lining is occurring due to the redistribution of insitu stresses, something which must occur fairly quickly, yet measurements of lining load show a more gradual increase in load. If the insitu horizontal stress is greater than the vertical, the model will predict that the lining will squash from the sides inwards, although most measurements of linings show the lining squatting.

This code is most typically used with lining elements to model the segmentally lined tunnel. The problem remains of what stiffness to use for this lining for the soil/structure interaction calculation. With the absence of many other alternatives, designers most often use the Muir Wood (1975) empirical ring stiffness formula which has been demonstrated to be not applicable to many design cases, and usually underestimates the

Table 3. Segmental lining load cases and analysis used

Load Case	Closed Form Solutions	FE/FD	Beam Spring Model
Ground Loading—share of excavation unloading	✓	✓	
Ground Loading—3D effects		✓(3D)	
Ground Loading—rock blocks			✓
Hydrostatic pressure	✓	✓	✓
Lining Self-weight		✓	✓
Surcharge Loading	✓	✓	✓
Grout Pressure–primary	✓	✓	✓
Grout Pressure–secondary			✓
Construction loads—TBM Gantry wheel			✓
Internal fit out—Jet fans, duct hangers			✓
Seismic loading			✓

moments induced in the lining. The other approach observed is to use the full section stiffness which results in an overestimate of moments. However, there are two fundamental problems in this approach which are not well known. The first of these problems is that the beam element, subject to displacement due to ground release on to the lining, can underestimate the true moments by up to 33% in the biaxial case. The FLAC Verification Manual (Itasca 2000) demonstrates the issue by showing a lined tunnel subject to biaxial stress. While the deflections and thrusts converge to the theoretical solution, the moments do not. For cases where the ground is flexible and the lining is stiff, this effect can be negligible. However, cases where the ground stiffness and load is high exhibit this error. A second error arises from the fact that a ring of any significant thickness experiences non-uniform stress across its section when a uniform radial load is applied. This effect is the origin of the M_u term in the equations above. Beam elements cannot account for this, yet the error in calculated extreme fibre stresses can be significant. This is quantified in the case study section below.

Limitations of Beam Spring Models

As observed from Table 3, the beam spring model can be used to model most load cases, and is also the most suitable method for determining lining forces from the application of point loads. However typically Strand 7 is used with Duddeck ground reaction springs and the beam stiffness most often used is the Muir Wood formula. This approach typically overestimates the moments in the lining, and for low cover tunnels, can result in unexpected temporary load cases governing the reinforcement requirements, such as secondary grouting, TBM gantry loading or Jet fan loading. This issue is demonstrated in the case study below.

CASE STUDY OF A SHALLOW AND LARGE-DIAMETER TUNNEL

The authors have been involved design reviews of the 3 Brisbane Road tunnels, and in the verification of the 14.53m OD 3-lane road tunnel in Auckland NZ.

This case study of the Waterview Connection Tunnel in Auckland is used to demonstrate the issues with ignoring the gravitational field stress for large diameter low cover tunnels.

The segmental lining analyzed in this case example is a 9 segment + key ring of 13.1m internal diameter with a segment thickness of 450mm as shown in Figure 1.

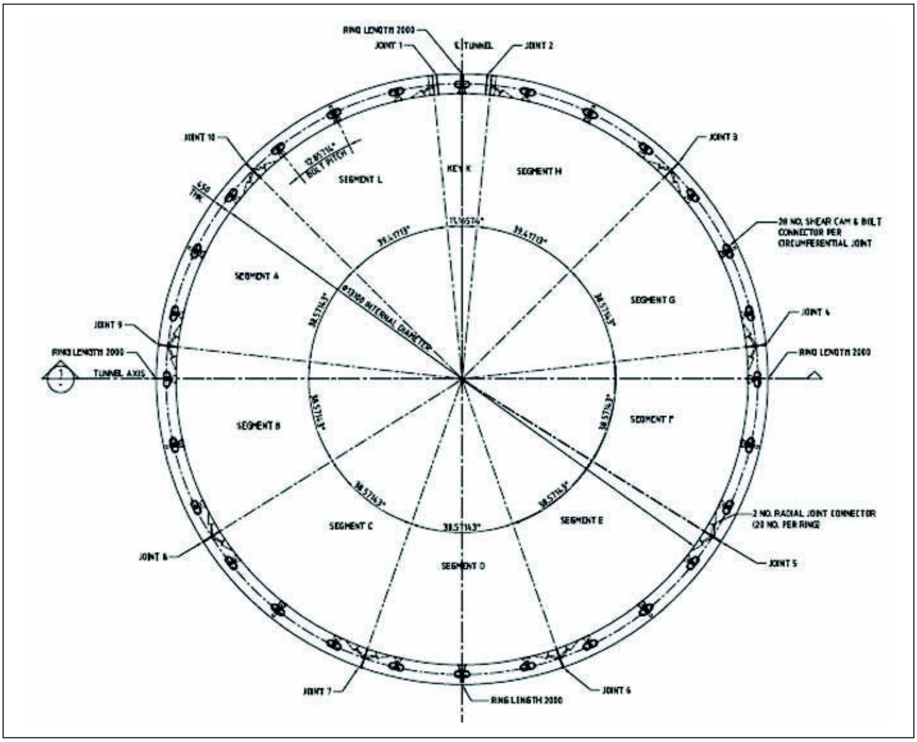

Figure 1. Segmental lining example—Waterview Connection, Auckland

Case 1 has a shallow cover of 9m which is equivalent to 0.64 tunnel diameters. Case 1 has a mixed face of alluvial clay in the crown and top half of the face, with Residual clay and Extremely Weathered Sandstone layer, and the Extremely Weak East Coast Bays Sandstone making up the remainder of the face.

Case 2 is the deep cover example where the cover is 37m which is 2.5 tunnel diameters. The material is contained entirely within the soft rock of the East Coast Bays Formation.

Comparison Between Hartmann and Curtis

As discussed above, the most commonly used closed form solution is a biaxial stress field such as that of Curtis (1976). However, particularly for large diameter tunnels, this approach misses the significant difference between crown and invert. Figures 2, 3, 4 and 5 show comparisons between the two closed form solutions.

It can be seen that the greatest difference between the solutions is in the thrust calculation. The difference is most significant for the shallow tunnel situation (Case 1).

Comparison Between Closed Form Solution and FLAC

This section now compares the closed form solution derived by Hartmann that includes the gravitational stress field with the Finite Difference code FLAC.

Figures 6 and 7 show the results for bending moment for Cases 1 and 2, using the equations for the Hartmann solution derived above and comparing these with FLAC.

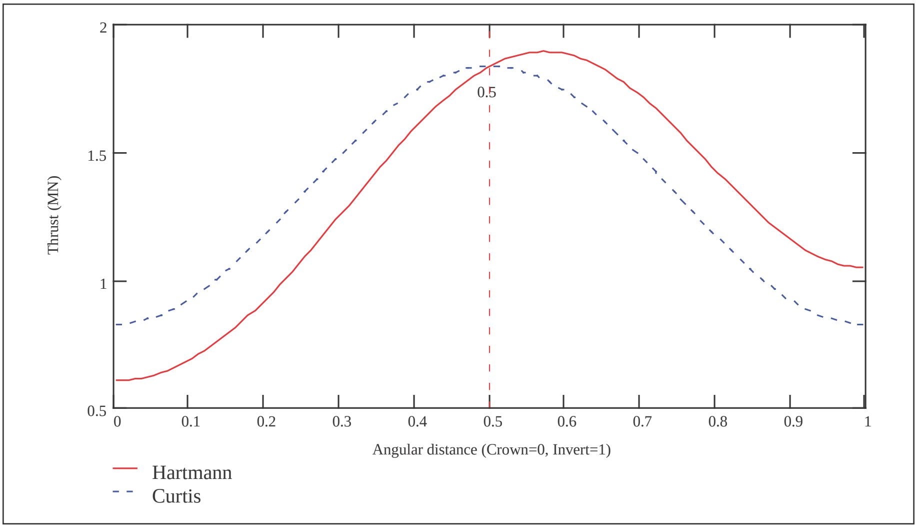

Figure 2. Curtis versus Hartmann solutions for Thrust–Case 1

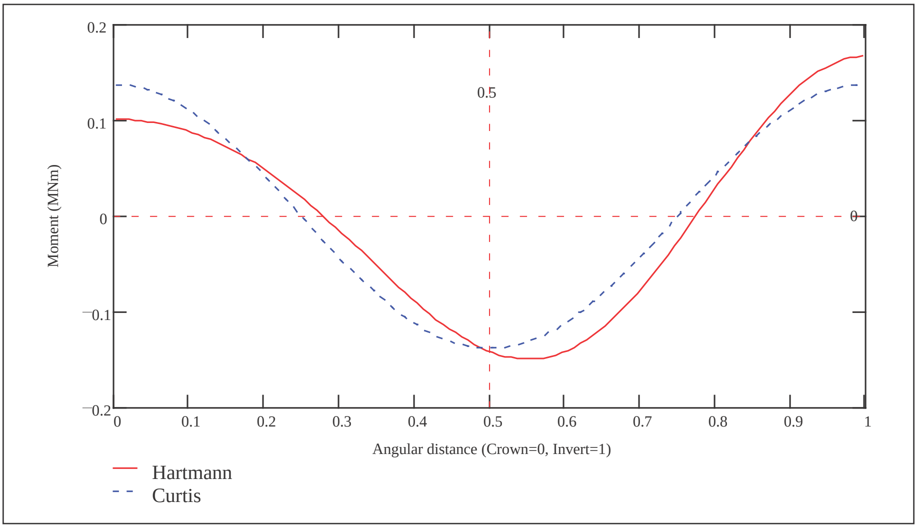

Figure 3. Curtis versus Hartmann solutions for Moment–Case 1

The results of comparison between deflection, tractions and thrusts show that with a fine and large enough mesh, FLAC converges to the closed form solution. However, the moment results converge to a different value. It can be shown that this is a direct result of the beam element formulation. Phases (Rocscience) shows exactly the same behaviour. In Figure 6, the results are similar because both effects are slight. Case 2 shows both effects, being the underestimate of moment as well as the effect of including M_u.

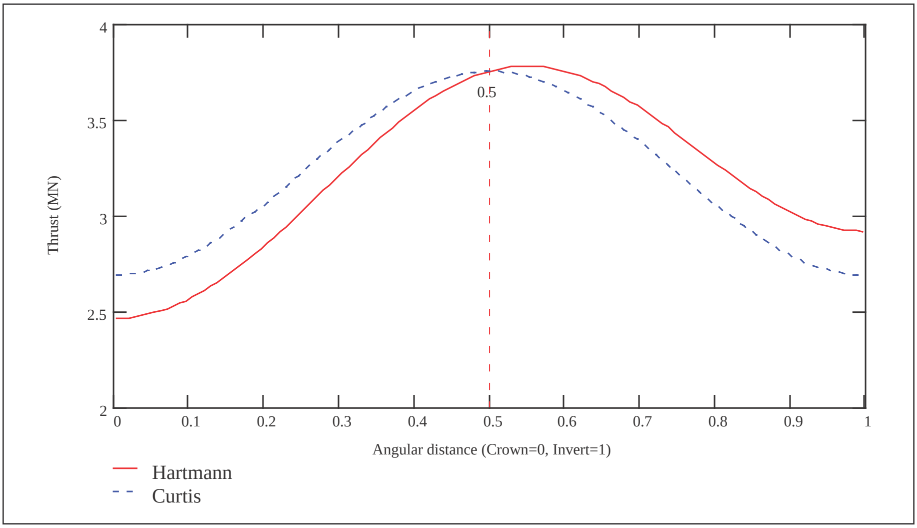

Figure 4. Curtis versus Hartmann solutions for Thrust–Case 2

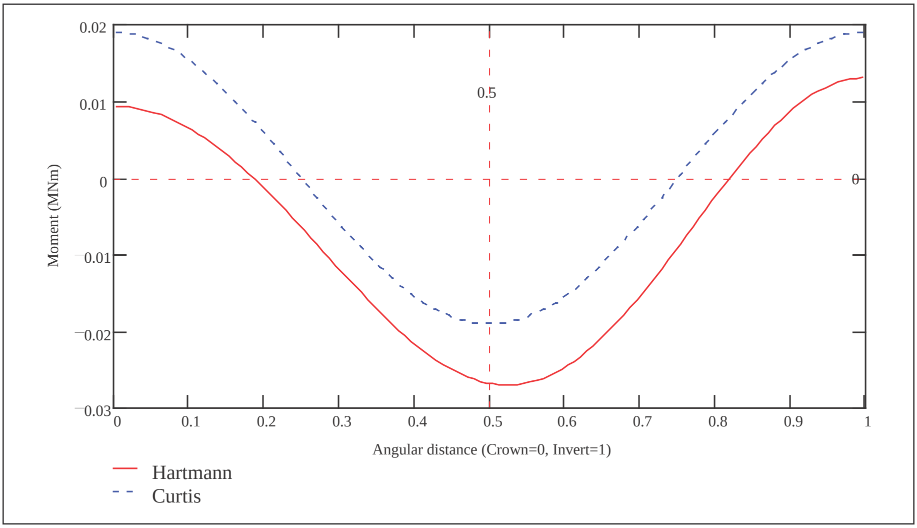

Figure 5. Curtis versus Hartmann solutions for Moment–Case 2

Comparison Between Closed Form Solution and Beam Spring Model

This issue is demonstrated by the Moment Interaction diagrams in Figure 8 which shows both the output from a Curtis (1976) closed form analysis, and the same analysis using Strand7 with the beam stiffness from Muir Wood (1975).

The difference in moment calculated is due to the use of the empirical segmental lining stiffness, which is over estimating the moments at low covers. The Stand7 results are also showing a higher axial thrust, as the analysis takes into account the gravitation stress from self-weight, which is ignored by the Curtis solution.

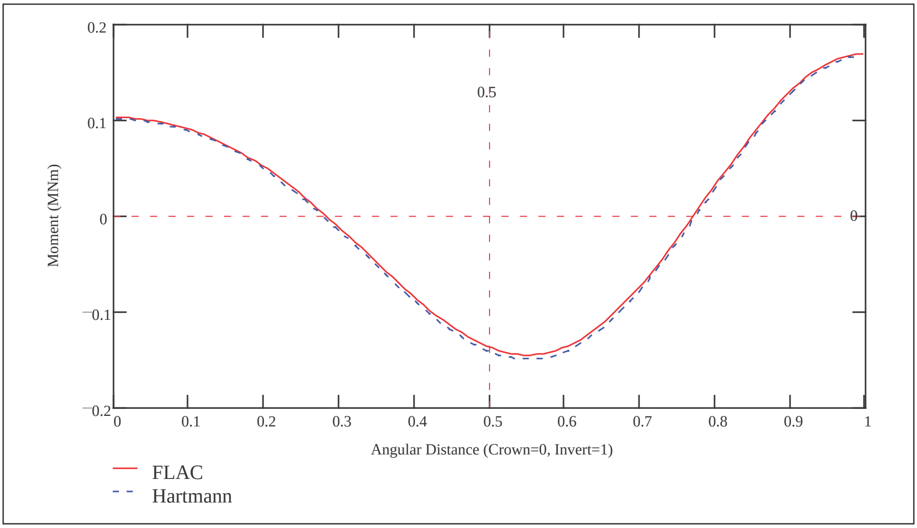

Figure 6. Case 1 shallow tunnel, weak ground

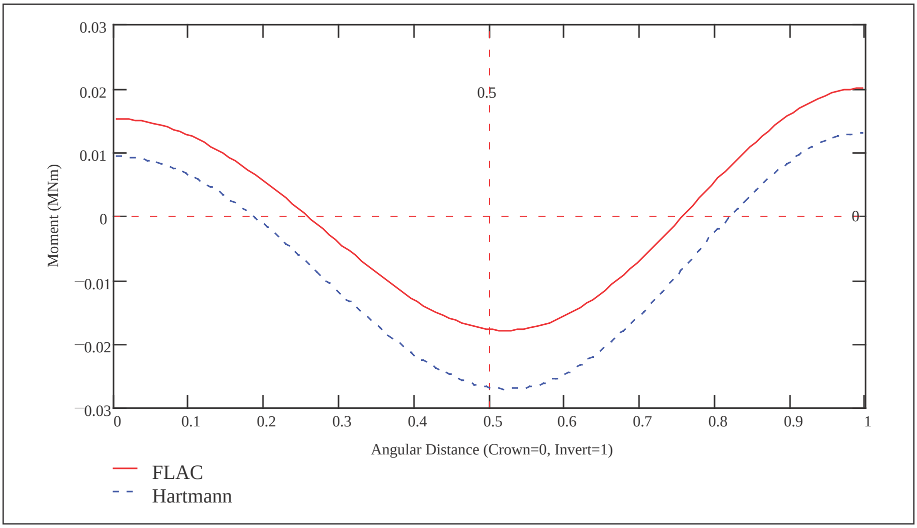

Figure 7. Case 2 deep tunnel, stronger ground

RECOMMENDATIONS

This paper demonstrates a few significant issues in the analysis of circular tunnels. Our recommendations are as follows:

1. The use of biaxial closed form solutions (such as Curtis (1976)) can be quite misleading for large diameter shallow tunnels. We recommend that the Hartmann solution be used instead.
2. The use of FE/FD codes with beam elements can involve some errors, particularly where the ground is stiff, and the ground loads are strong relative to the lining. We recommend that some checks be carried out between the codes

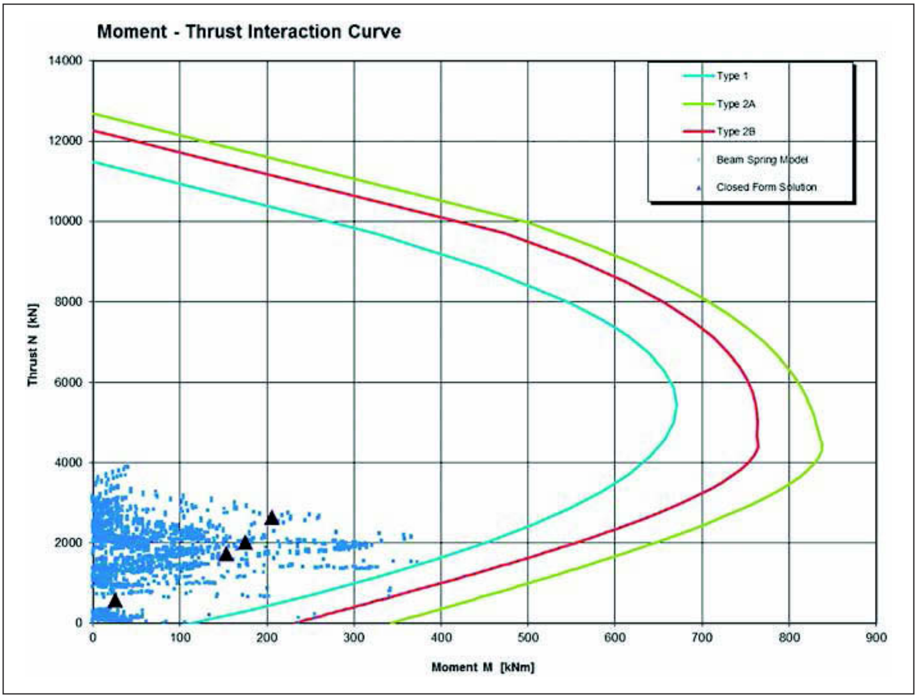

Figure 8. Curtis versus Beam Spring model—Case 1

and the Hartmann solution for the particular geometry, depth and ground/lining stiffnesses. If there are significant differences, then for the more complex case which the FE/FD code is to analyze then a correction factor can be used.

3. Where beam spring models are used, it is suggested that the beam stiffness applied to the model is calculated explicitly from the joint rotation as described in Ireland and Asche (2011). The method involves using the Hartmann solution to calculate the ovalization of the ring due to soil/structure interaction. The joint rotation and resulting moment can then be calculated, and Morgan's equation used to back-calculate the ring stiffness. This provides an equivalent ring stiffness at each section that takes into account the ring stiffening effect of axial thrust. This ring stiffness is then used in the beam spring model for the load cases that include point loads.

The main recommendation is that the discipline of cross checking between closed form solutions and FE/FD results is important, and not to believe the results from complex analysis software that are inconsistent with proven closed form solutions.

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PART 16

Pressure Face TBM Case Histories—I

Chairs

Ed Whitman
Michels Tunneling

Glen Frank
JayDee Contractors, Inc.

CHALLENGES OF EPBM EXCAVATIONS IN PRAGUE

Karel Rossler ▪ Metrostav a.s.

David Cyron ▪ Metrostav a.s.

ABSTRACT

Two EPBMs (Earth Pressure Balance Machines) completed the 4.8 km twin tunnels section of the Prague Metro V.A. subway extension in advance of the schedule in November 2012. It was the first time experience with the modern EPBM tunneling technology in Czech Republic, yet it was adopted very quickly. The EPBM excavations were challenged by interruptions caused by underground stations, in which the EPBMs had to walk and restart, by impact of thrust cylinders on segmental lining, by reduced two-component grouting performance, by strict surface settlement control, or by muck stickiness. The EPBM challenges were met thanks to the innovative approaches that adapted the technology to given project conditions.

PROJECT

The construction of Metro V.A. started in April 2010. It will add to operation 4 new stations and 6.1 km of tunnels to the existing Prague subway network by the end of 2014. Prague subway, whose origin dates back to 1966 has 59 km of operated alignment tracks, which are mostly underground, and 57 mostly mined underground stations. The Metro V.A. extension included 4.8 km of twin tunnels bored by two Earth Pressure Balance Machines (EPBMs), and 1.3 km section including mined double track tunnel, Figure 1.

Metro V.A. was the first phase of the two phase project to build the subway connection to the Vaclav Havel International Airport. The funds of EUR 750 million for the five year project of the first phase were provided by the owner Dopravni Podnik Praha a.s. (DPP), out of which 40% could be provided by the European Union. The project was supervised by the Inzenyring Dopravnich Staveb a.s. (IDS), and designed by the Metroprojekt Praha a.s.

The construction works including the rails installation have been won for the price of EUR 545 million by the Joint Venture of the Metrostav a.s. and the Hochtief CZ a.s. companies, which divided the amount of the works in the ratio of 60%, and 40%, respectively. The Metrostav Company was responsible for building 4.8 km of twin tunnels, and two underground stations Petriny, and Veleslavin (Veleslavin was built by the daughter Subterra Company a.s.).

Schedule

The five year project schedule dictated that the construction works be completed in four years by the end of 2013 to allow time for the technology installation, and the test operation. To comply with the four year construction schedule the construction of the stations, and the tunnels had to be planned to run in parallel, and independently as much as possible. That could seem a difficult task, since the 4.8 km alignment of the EPBM twin tunnels was passing through three underground stations. The stations excavations would obstruct the EPBM excavations, and vice versa the stations final linings would not be finished before the EPBM operations cleared the stations. The avoid

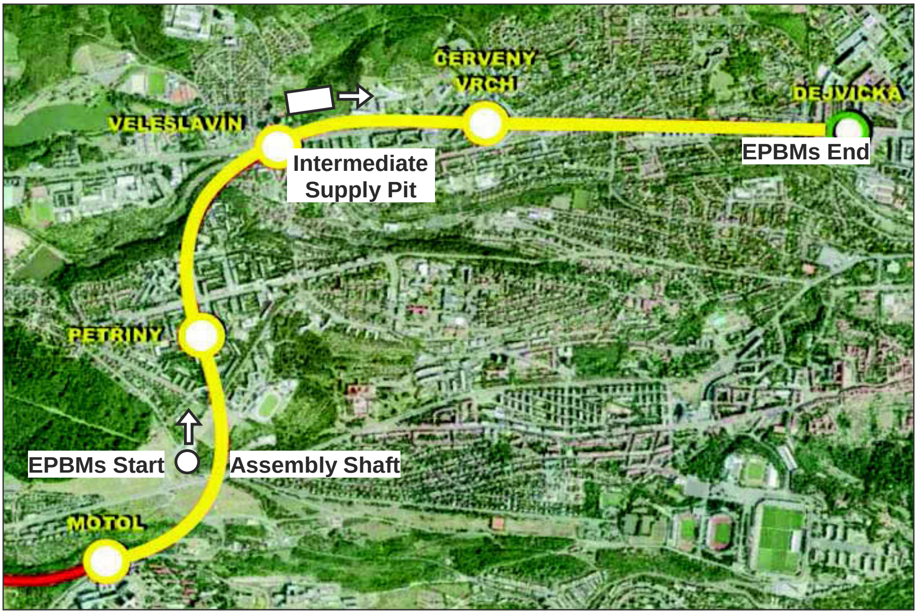

Figure 1. Metro V.A. will add to operation 4 new stations, and 6.1 km of tunnels

potential construction site interferences, separate access pits, shafts and adits had to be built for the stations constructions, plus an intermediate access shaft was placed in the middle of the 4.8 km EPBM tunnels for relocating the supply and logistic of EPBM operations to allow an early start for the final lining works of the two underground stations Petriny and Veleslavin.

Thereby, the EPBM excavations got on a critical path, which commanded the twin tubes to be completed in a year and a half from the date the machines were assembled at the launch shaft (mid of 2011) until they broke through in the Dejvicka disassembly chamber (end of 2012).

Alignment

The Metro V.A. extension will add an additional capacity of 45 000 passengers per day to the existing Prague subway ridership of 1.3 million of passengers per day. To comply with the planned capacity, the 100 m station platforms accommodated 5 wagon trains, and the alignment was designed for the train speed of 80 km/h with minimum headway of 90 seconds. The alignment minimum horizontal curve was 640 m, and the vertical gradient 3.95%.

Three underground stations along the twin tubes alignment were designed with center platforms 10–12 m wide, while the Motol station, as part of the double track tunnel alignment, included two side platforms 2×5 m wide. For better passenger circulation the stations were kept as shallow as possible. Considering the minimum rock cover the resulting stations depths ranged from 20 to 40 m. Ventilation shafts, rooms, and technological objects were built at each station, and the distance between the twin tunnels crosspassages was about 200 m.

Geology

Major portion of the EPBM tunnels alignment was situated in the clayshale bedrock, Figure 2, with the bedrock cover of 20–30 m. At the launch shaft, about 0.7 km of the

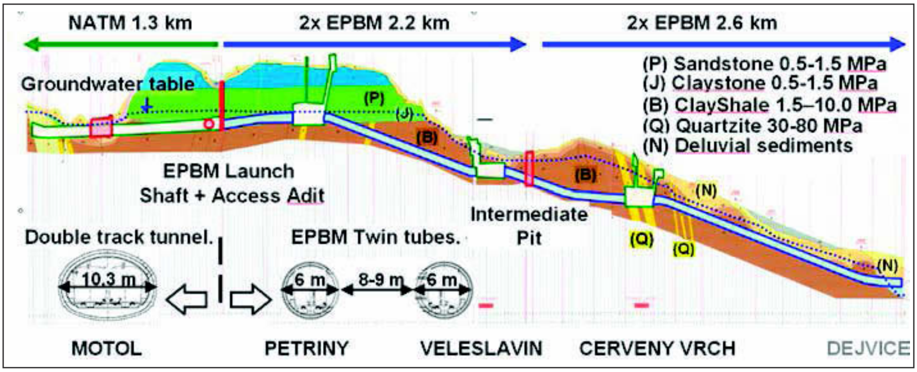

Figure 2. Major portion of the alignment was situated in the clayshale bedrock

alignment was intersected by the waterbearing sandstone sealed by the clayshale layer at the bottom. The last, one third of alignment was shallow with overburden 12–16 m, and intersected by deluvial sediments, and by weathered clayshale bedrock horizon classified usually as clayey sand or clayey gravel.

The bedrock layers were characterized by uniaxial compressive strengths of intact rock samples in the range of 0.5–1.5 MPa. The deformation modulus of the weak and soft rock mass including three sets of discontinuities was estimated between 45 GPa and 300 GPa, yet, the bedrock provided vertically stable excavation face, which allowed the major portion of the alignment to be excavated in the EPBM open mode.

The maximum groundwater table elevation above the tunnels crown was 28 m. Clayshales and claystones were typically impervious, however in weathered zones, shears, faults, or especially when intersected by blocky quartzite intrusions the ground water inflows could reach 1–5 l/sec. High groundwater inflows were experienced in the waterbearing sandstone, where the inflows locally ranged 5–10 l/min.

EPBM TECHNOLOGY

The two EPBMs S609, S610 were procured from the Herrenknecht AG. Company. The machines technical parameters, Tab. 1, respected given geological conditions, and several technological features were added to the EPBM design: wear resistant material of chromium carbide for the cutterhead and the screw conveyor protection in the highly abrasive sandstones with high quartzite content of more than 85% and Cerchar abrasiveness of 4.5 that were encountered in the first 500 m of excavation; the cutterhead opening ratio was increased from 23% to 30% to minimize the potential of cutterhead clogging by clayey muck.

Motivated by the strict settlement criteria Metrostav requested a feature that would minimize the convergence of the unsupported tunnel walls due to the overcut along the shield. Herrenknecht engineers recommended a bentonite pressure system that injected bentonite into the gap around the shield and the face. The system, was successfully used during the planned long standstills for conserving the shield and limiting the ground loads on the machine, however, the system was not used during the excavation cycles due to required large volume of activated bentonite.

The two shields, the front shield and the tailskin, were connected with the short auxiliary cylinders, whose function was to drag the tailskin passively behind the shield. The inclination of the tailskin was thus controlled by the position of the front shield and by the segmental rings.

The cutterhead was not axially displaceable neither articulated. The overcut was controlled by the gauge cutters shift. To prevent self undercutting which would result in machine diving, the EPBMs were equipped with the shift plate installed under the front shield, whose thickness was equal to the overcut of the gauge cutters (see also Table 1).

Table 1. EPBM parameters

Sealing operation pressure	3,5 bar
Cutting diameter	6 060–6 100 mm
Thrust cylinders quantity	32,0 unit
Thrust cylinders groups	5,0 unit
Thrust cylinders stroke	2 200 mm
Installed nominal thrust	38 926 kN
Main drive power	1 200 kW
Nominal torque	4 400 kNm
Thrust cylinder speed	80,0 mm/min

Logistics

To meet the planned schedule of less than 18 months of the EPBM excavation, the logistics were designed as a unique combination of belt conveyors for muck transport, rubber wheeled Multi Service Vehicles (MSV) for segments supply, and a two-component grouting system for pumping the grout from the surface directly to the EPBM.

The twin tubes excavation had two access sites: the launch shaft, and the intermediate pit, which divided the 4.8 km alignment into 2.2 km and the 2.6 km sections. The project schedule defined that the EPBMs had to achieve an average performance of 12–16 m/day for the entire 4.8 km alignment. The logistics were designed to allow the simultaneous EPBMs operation achieving maximum speeds of 30m/day, which made the requirement of average speed of 12 m/day no difficult task. In reality, the best advance rates of 630 m/mo, and 36 m/day were achieved.

The use of conveyor belts increased the work safety and continuity of the transport operations especially in the circular launch shaft of 22 m in diameter, where crane handling of the muck trains of the two simultaneous EPBM operations would be slow and dangerous. With the conveyor belts transporting the muck directly to the surface, only one tower crane was required for loading the segments on MSVs, in addition, the segment transport gained independence from the muck transport, and two MSVs were used per tunnel. At the launch shaft, the joint conveyor belt (800 mm wide) for both EPBMs transported the muck through the tunnel adit to the surface deposit. At the intermediate shaft, the two EPBM conveyor belts (650 mm) transported the muck from the pit bottom to the surface in an inclination of 17 deg.

MSV vehicles were also an important addition to the work safety thanks to their short breaking distance and good maneuverability of independent two axle steering, which was important in tight working spaces of the underground. The MSVs had output of 147 kW, capacity of 17.5 t, and maximum speed of 16 km/h.

Segmental Lining

The segments were prefabricated by the Doprastav a.s. Company in their Slovakian plant, which in the cold production rate was able to output nine rings per day. Nine stationary formwork sets equipped with external vibrators were served by a crane transporting concrete in containers. The concrete hardened for sixteen hours in the ambient temperature of the production hall without any additional heating except for winter time when the hall was heated up to sixteen degrees Celsius. Reinforcement cages were welded from single steel bars.

The 5+1 ring configuration of the universal rings with 15 mm taper on both sides (total 30 mm) accommodated conveniently the minimum horizontal curvature of 640 m with a reserve for a 300 m recovery radius. The inside diameter was 5.3 m, the thickness 250 mm, and the width 1.5 m. The amount of steel reinforcement of 105 kg/m^3 represented the reinforcement ratio of 0.5% including both compression and tension bars.

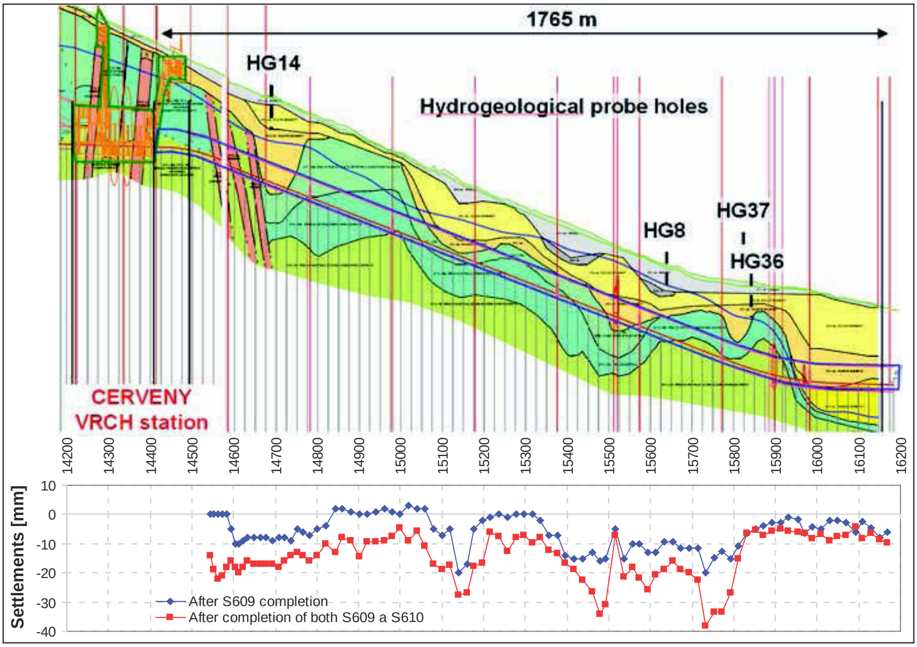

Figure 3. Measured settlements under Evropska Street

The contact at the circumferential joint between the rings was cushioned by the hard board plates, and fastened by 16 screw connection in correspondence to 16 thrust rams, which resulted in 16 available positions of the key segment in the ring assembly. The longitudinal joints of the key segment included guiding rods, and the segments were waterproofed with the neoprene gaskets.

The ram plates contact areas were optimized for the thrust pressures not to exceed 23 MPa under the maximum EPBM ram force of 2432 kN (hydraulic pressure of 320 bar in the cylinders). Attention was paid to the potential of the concrete bursting induced by tensional stresses generated under the thrust plates of the EPBM rams. The impact of the ram loading was modeled in a three dimensional numerical model including an explicit model of the steel reinforcement cage, and was confirmed by an experimental laboratory testing. Both models numerical and experimental showed more than sufficient segments concrete resistance to bursting.

MITIGATION OF SURFACE SETTLEMENT

The project prescribed a strict surface settlement limit of 10 mm above the twin tunnels. In two thirds of the 4.8 km alignment under 20–30 m of rock cover, the EPBMs complied with the settlement limit conveniently while excavating in the open mode (without active face support), however the last 1800 m of the alignment under Evropska Street, where the tunnels were shallow with 12–16 m of overburden, 6 m of ground water table above the tunnel crown, and the clayshale bedrock was alternating with deluvial and alluvial sediments in the tunnel profile, the excavations had to be performed in the closed mode to protect the face stability, and minimize ground deformations.

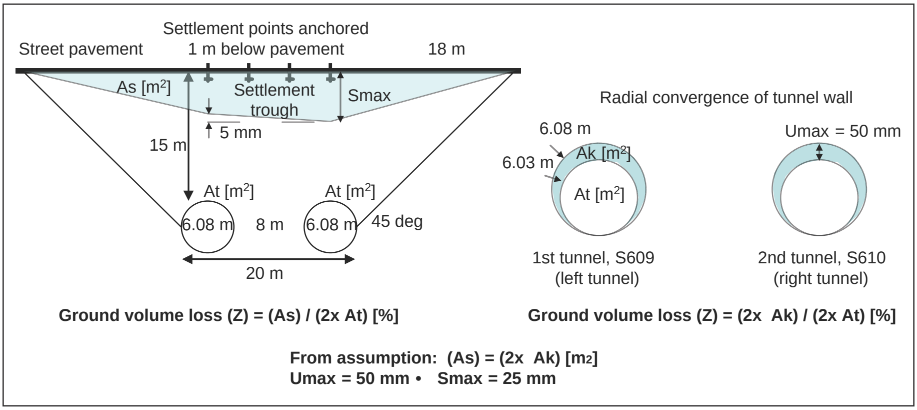

Figure 4. Transverse settlement trough and ground volume loss calculations

Table 2. Settlement trough ground losses (Z) with corresponding measured settlements (Smax), and estimated tunnel convergence (Umax)

Maximum Settlement After Both EPBMs Passed Under the Given Section	Ground Loss of Settlement Trough of Both EPBMs	Tunnel Convergence in Each of the Tunnels
Smax [mm]	Z [%]	U max [mm]
8 mm	0.50%	15 mm
10 mm	0.65%	20 mm
20 mm	1.27%	40 mm
25 mm	1.65%	50 mm
40 mm	2.55%	80 mm

Measured Settlements

The measured settlements plotted in longitudinal profile, Figure 3, revealed that the criterion of 10 mm was exceeded mainly in the depressions with deluvial sediments, where the settlements were 25 mm, 30 mm, and 40 mm.

The transverse settlement troughs were established by the four surveyed settlement points above the tunnels, and by the 45 degree angle taken from the tunnels sides, Figure 4.

The ground volume loss (Z) was calculated from the settlement troughs for a range of measured settlements (Smax). The tunnel convergences (Umax) were calculated from the assumption that the ground loss of the settlement trough was equal to the ground loss in the tunnels, Table 2.

For given settlement though geometry, the magnitude of surface settlement was estimated to be about half of tunnel deformation, which meant that the maximum tunnel deformation of 50 mm due to the overcut above the shield would lead to settlement of 25 mm. Comparison to the maximum measured settlement of 40 mm indicated that there were other settlement sources.

Sources of Settlements

To identify the settlement sources it was advantageous to divide the tunnel into three sections along the excavation profile: (i) front of the face, (ii) above the shield, and (iii) along the lining because the three sections required different measures to limit the tunnel deformation.

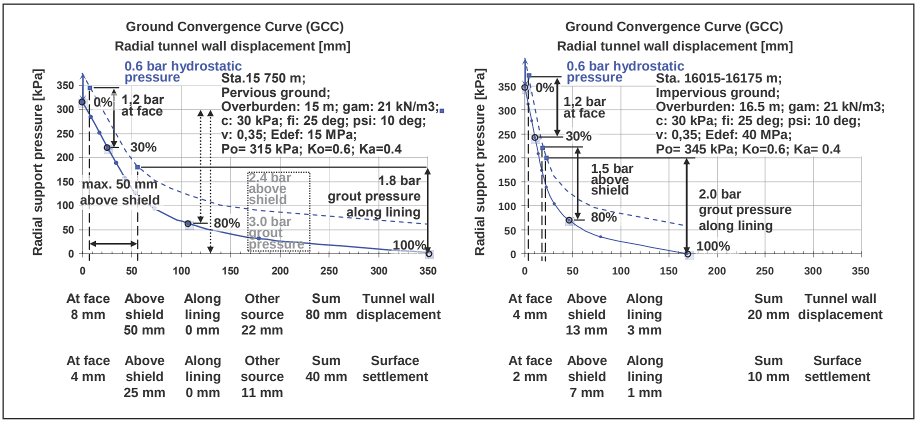

Figure 5. Ground Convergence Curve (GCC) concept investigated deformation sources along the tunnel, and identified support pressures

The surface settlement points could not identify the contribution of the three sections to the settlements because their zone of influence overlapped at the surface as the deformations progressed from the tunnel depth. Therefore, the ground deformation sources were investigated with the help of Ground Convergence Curve (GCC) scheme, Figure 5.

The text book solutions of axi-symetric or three-dimensional models of unsupported tunnel showed that in linear elasticity, the ground deformation started 2 diameters ahead of the face, and at the face reached 30% of the final tunnel deformation, the deformation of the unsupported tunnel walls was about 80% in a distance of 1 diameter behind the face, and the final 100% deformation was 2 diameters behind the face. In those linear elastic models the increment of the ground deformation was proportional to the increment of ground stress release. The same approach was also used in the GCC although it adopted non linear elastoplastic stress-strain law. The ground stress releases of 30%, 80%, 100% corresponded to locations at the face, the shield end, and the lined tunnel, respectively.

The EPBM closed mode made the tunnel undrained, therefore the convergence curve, which was a drained solution, was modified by adding the ground water pressure along the vertical axis. Being a solution for deep tunnels considering ground arching, the GCC was still an appropriate tool for the tunnel sections with overburden slightly more than 2 D.

For the stationing 15750 m, and 16015–16175 with 40 mm, and 10 mm settlements, respectively, the GCC concept concluded:

- The support pressures had to be applied in all three sections along the tunnel because the deformation of each tunnel section separately, when left unsupported, was a sufficient source to exceed the criterion of 10 mm. The support in each section was equally important.
- The support pressures, which would comply with the settlement criterion, were: (i) 1.2–1.5 bar at the face, (ii) 2.4 bar at the shield, (iii) 3.0 bar grout pressure.

Shield Overcut

The recorded data of both EPBMs showed that in spite of the pressure fluctuations, the operators were capable to keep the face support pressures above 1.2 bar, and the

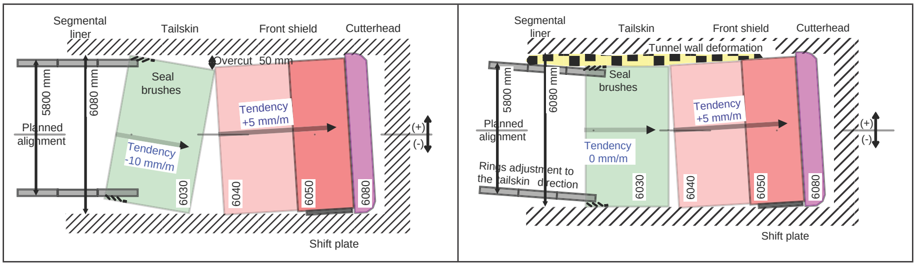

Figure 6. Due to the shield geometry the overcut over the tailskin was 50 mm

grout pressures at 3.0 bar. That meant that no significant deformations were generated along the tunnel except for the middle portion, above the shield, where no support pressure was applied since the foam, which would seep there from the face, was not considered a reliable support medium, especially in pervious ground.

The length of the front shield and the tailskin including the cutterhead was 8.84 m (more than one excavation diameter). Due to the shield geometry the overcut over the tailskin was 50 mm, Figure 6, which was considered to be a significant source of settlement of up to 25 mm.

The tunnel deformation and the ground load that was developed over the shield was demonstrated by the EPBM records of vertical tendency expressed as vertical deviation over the shield length in mm/m.

The tendency of the front shield was maintained at ±2 mm/m during excavation. The increase in tendency to +5 mm/m (upward inclination) was an indication that the ground load was imposed on the shield. In response to the front shield inclination, the tailskin vertical tendency would also increase up to –10 mm/m since it was connected to the front shield, and at the end supported by the segmental rings. Later the tailskin tendency would go to zero, as the ground load pushed the tailskin to the tunnel bottom, Figure 6.

Groundwater Fluctuations

At the section where settlements reached 40, the overcut above the shield contributed by 25 mm to the settlement, therefore there had to be another ground settlement source.

The foam injected during each excavation round, and the compressed air, which was temporarily used to compensate the face pressure fluctuations during standstills, were penetrating the pervious weathered clayshale, and pushed the groundwater away from the tunnel, which in turn led to high fluctuations of hydrostatic pressure (of more than 6 m above the original level), and created a geyser mixed with air bubbles in the hydro-geological probe hole HG 36 when the EPBM was still 20 m away from the probe, Figure 7.

Pushing the groundwater away from the tunnel could lead to local consolidation settlements, and the flow of the water could wash out the fines from the ground, which was considered the second significant source of the surface settlements.

Support Pressures

For the closed mode excavation along Evropska Street the following support pressures were prescribed:

- Face support pressure: 2.0–2.4 bar.
- Grout injection pressure: 2.5–3.0 bar (typically 0.5 bar higher than the face).

Figure 7. Hydrostatic pressure fluctuations created a geyser in hydro-geological probe hole

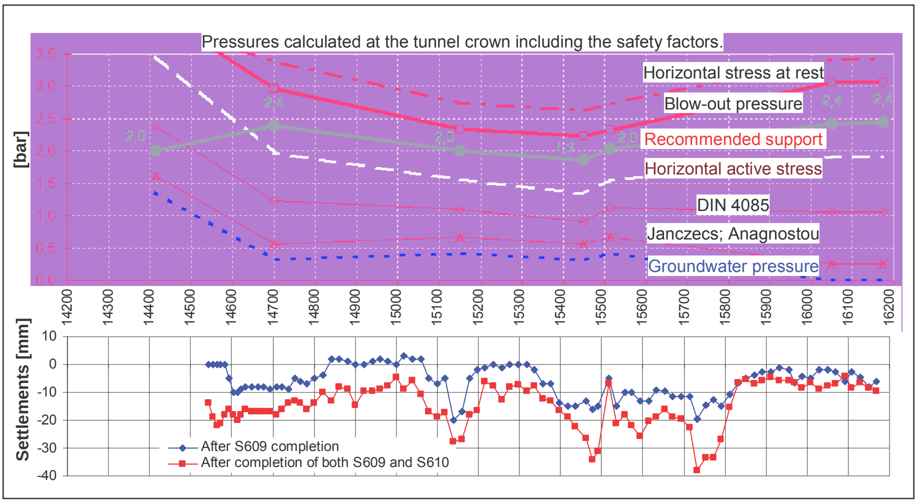

Figure 8. The face support pressures resulted from stability analyses

The face support pressures of 2.0–2.4 bar resulted from stability analyses, Figure 8, which included safety factors, and were selected close to the horizontal at rest ground stress to mitigate elastic deformation due to the low ground modulus (Edef). The face pressures magnitudes including the safety factors well compensated for the pressure fluctuations that took place during excavations, and also complied with the required grout and shield support pressures magnitudes.

The two belt scales recording the muck weight of each advance round could not be used effectively to detect the excessive ground deformation ahead of the face because the 30% variations in the ground unit weight (26 kN/m^3 for clayshale, 20–18 kN·m^3 for deluvial sediments) were too large. As a result, the 0.7% weight increase caused by a significant ground deformation ahead of the face of 30 mm would be unnoticed. Therefore the most reliable control of the face deformation was the control of the face pressures.

EPBM IMPACT ON SEGMENTS

The first segmental rings installed immediately after launching the EPBMs contained cracks, which would not be unusual for any TBM launch, however the cracks were appearing regularly also in the rings, which were properly embedded in the grout fill.

Two types of the cracks were distinguished: (a) Cluster cracks—a group of up to four cracks, which were running across the full 1.5 m segment width, and were persistently appearing at one location of the ring for a distance of tens of tunnel meters, (b) Random single cracks—a single crack in the middle of the segment, Figure 9.

The crack types had in common several features that could point out their origin: the hairline thickness of less than 1 mm; none or minimum groundwater leak; the inception of the cracks within the tailskin, while the ring was still being pushed by the thrust rams; the thrust ram forces operating at less than 100 bar (less than one third of the maximum design pressure); the cracks orientation in parallel with the tunnel longitudinal axis.

The investigation revealed that the source of the cluster cracks was the inward bending of the ring in transverse section, which was induced by the contact pressure of the tailskin, Figure 10. When bended the cracks opened on the inside surface of the segment, and then after leaving the tailskin the cracks closed. For the tailskin envelope to push on the ring, the 30 mm tolerance gap between the tailskin and the ring was exceeded, which was achieved by a combination of several factors: deformation of the tailskin; inclination of the tailskin (tailskin drift); ring deformation (elliptical ovalization); diversion of the ring from the excavated alignment (wrong selection of the ring orientation). However, none of those factors alone could lead to such a tight contact with the tailskin to crack the segments, but their combination could, as it was observed several times during excavation, when cracked rings stretched over a 70 m distance.

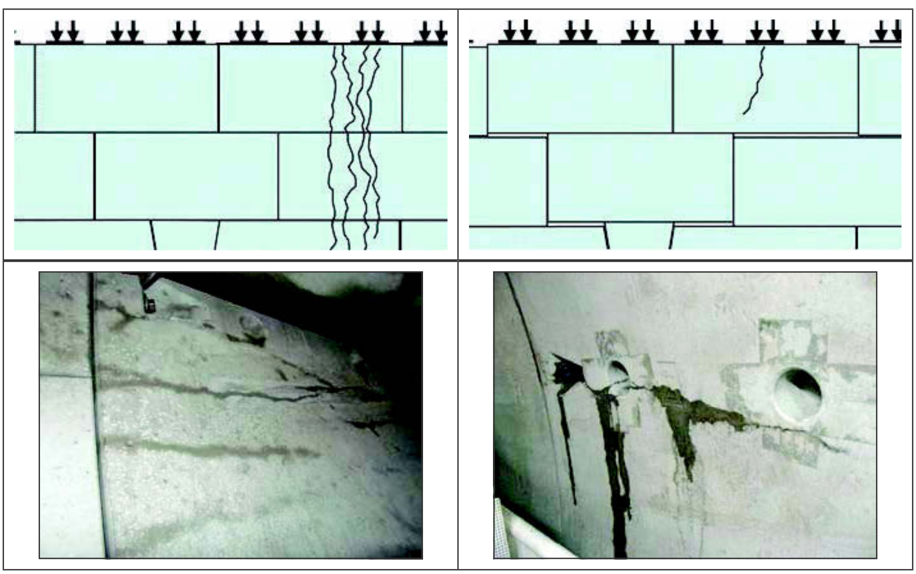

Figure 9. Cluster cracks and random single cracks

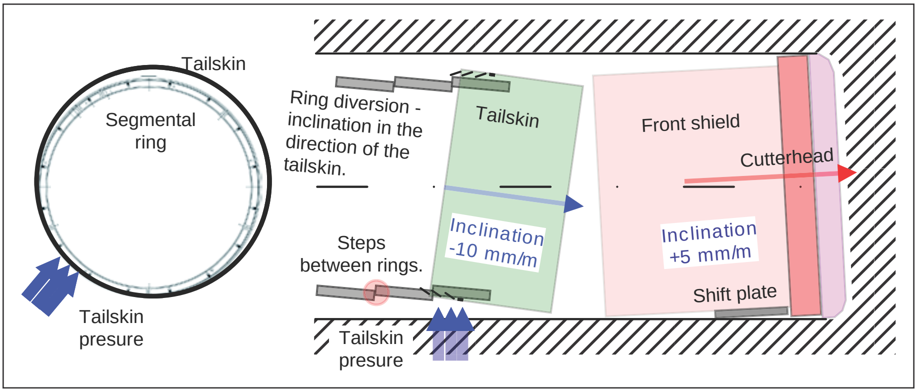

Figure 10. Source of the cluster cracks was the contact pressure of the tailskin

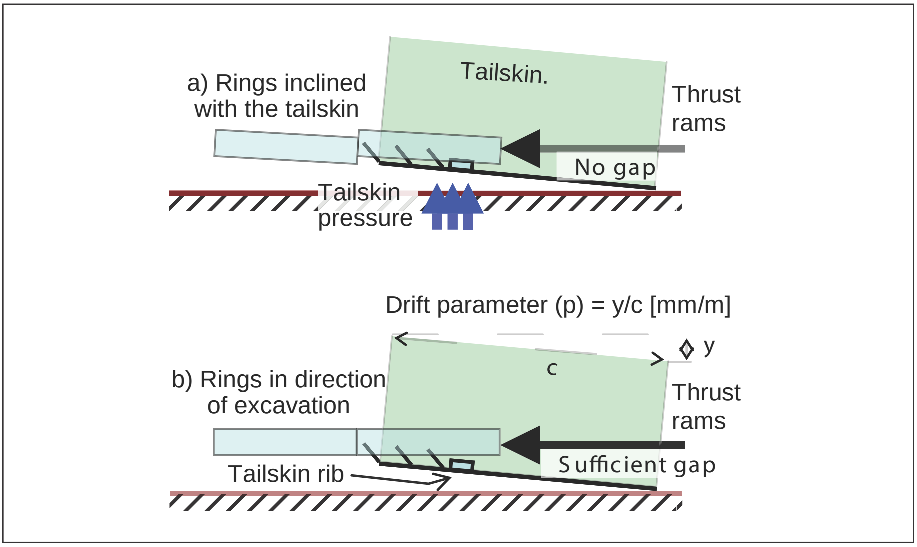

Figure 11. Correction of the rings inclination by the drift parameter

The solution was sought in eliminating the segment diversion, which would also reduce the impact of tailskin inclination. The VMT GmbH Company facilitated an input of a "drift" parameter into the segment selection procedure, which determined the key stone orientation for the next ring installation. The drift parameter corrected the ring position by making it parallel with the excavated alignment rather than with the inclined tailskin, Figure 11.

The single cracks were typically originated from the niche of the screw connector in the middle of the segment, and extended diagonally or parallel but seldom reaching across the full 1.5 m width of the segment.

The source of the random cracks was the longitudinal bending of the segments, which resulted from an uneven circumferential joint plane support of the previous ring caused by non-uniform compression of the contact hard board plates, Figure 12.

Although each ring assembly was sufficiently precise with well aligned segment edges, the differential loading of the five ram groups led to the non-uniform compression of the hard board plates during the excavation cycle. Segments that were not

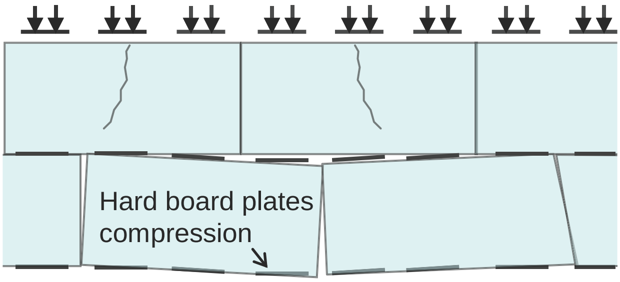

Figure 12. Source of the random cracks was uneven ring support caused by the non-uniform compression of the contact hard board plates

a) Steel cradle in a curve prepared for the shield in the intermediate shaft.

b) The same cradle to support the gantries

Figure 13. The steel cradle to support both the 6 m diameter shield and the wheeled gantries

supported along the entire length became typically bended by the grip of the three thrust plates acting on one segment, which consequentially cracked the segment usually in the middle, and the crack was running open through the entire segment thickness of 250 mm.

The unevenness of the rings was minimized by replacing the compressible 3 mm hard-board plates by thinner 1 mm hardened PVC plates.

WALKS AND STARTS IN UNDERGROUND STATIONS

The EPBM walks and the re-starts in the underground stations had a great impact on the overall EPBM advance. Three underground stations, one intermediate open pit, and one underground cavern for ventilation room represented five interruptions of the EPBM excavations.

Steel prefabricated cradle supports, and bracing frames for EPBM restarts in tight underground spaces, which were reusable, economic, flexible to modifications, and quick to assemble were engineered, to make the excavation interruptions as short as possible.

The prefabricated and unitized steel supports were used to support both the 6 m diameter shield, and the wheeled gantries. After the EPBM passed through the station the supports were dismantled, and were re-used in the next station. The steel cradle

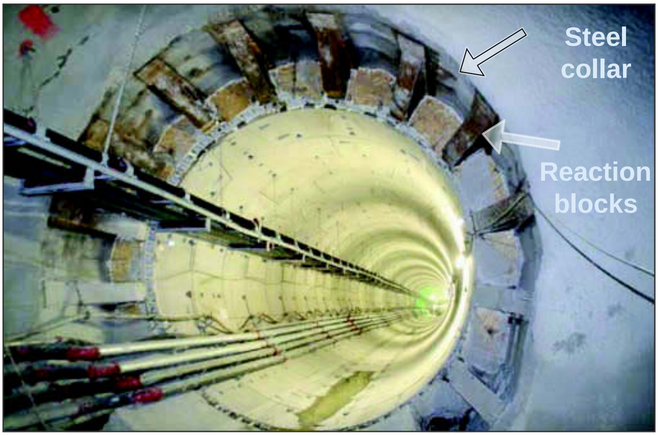

Figure 14. Bracing for EPBM start in underground station consisted of a steel collar and reaction blocks

was quick to assemble and disassemble, and easy to adjust to required directions and shapes, Figure 13.

For the initial EPBM start from the launch shaft a massive beam frame was procured from the Herrenknecht Company. The same frame was used again in the intermediate pit for the re-start of both EPBMs, however the frame would not fit in the underground station NATM drifts, and could not be assembled behind the shield since most of the space was occupied by the EPBM.

A bracing, whose major portion was installed before the shield arrival, and which was capable of taking the load from the EPBM by the time the shield was ready to bore, saved time. The bracing system consisted of a steel collar concreted and bolted into the primary lining at the neck of the 10 m launching chamber, Figure 14. The collar was installed in advance of the shield arrival, which would leave sufficient time for the collar concrete to harden. Then behind the shield the steel reaction blocks were promptly welded onto the steel collar. The bracing system reduced the EPBM start time, however, provided much less reaction force than the beam frame, due to the low tensile strength of the lining and the rock. Low start force could be a major drawback if the force was required to prevent the machine from diving. Against diving the EPBMs were equipped with the shift plate installed under the front shield.

TWO COMPONENT GROUTING

The expected minimum maintenance was the reason to adopt the two-component grouting system for the EPBM excavations, which relied on the system's ability to control the timing of grout hardening. Thanks to the composition of the grout, Component A, the grout remained liquid for minimum of 72 hours, but after mixing it at the right time and at the right place with accelerator, Component B, it would set rapidly in its destination place, which was the space between the lining and the ground. Component A, composed of 300 kg of cement, 35 kg of betonite, 8 kg of plasticizer, and 811 kg of water in 1 m^3, was mixed with the Component B, which was 7% of sodium silicate.

The prolonged liquidity allowed the grout to be transported in the pipes from the surface batch plant directly to the EPBM tank, which minimized grout handling operations. Component B (accelerator) was transported in containers by MSV and transferred to the TBM tank by pumping.

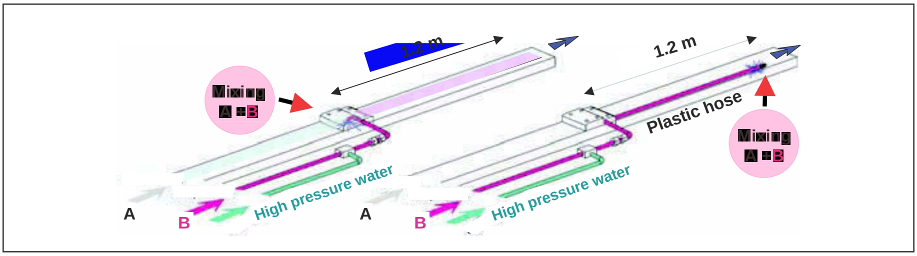

Figure 15. A flexible hose was attached to the sodium silicate line to extend the mixing point of the two components to the tailskin end

The initial experience with the grout was however not as expected, the EPBM tailskin grout lines clogged and because of that the excavations suffered from delays caused by frequent cleaning of the lines. At the beginning of excavations, the clogging could have been explained by the learning curve, and by the stop and go excavation progress due to sticky muck problems, which led to low grout flows, and to high potential of settling. Nonetheless, the problems with grout clogging were still persistent also after the machines achieved continuous excavation advance rates of 30–40 mm/min.

On average, the cleaning required minimum of 4 hours of tough hammering and drilling through hard grout in a frequency of every 2 days. The hard grout was deposited mainly in the 1.2 m long pipe, which transported the two mixed components from the mixing chamber to the tailskin end.

The situation improved significantly after a synthetic flexible hose was attached to the sodium silicate line, and extended the mixing point from the midst of the tailskin to the tailskin end, and out of the tailskin pipe, Figure 15. By mixing the components outside the tailskin the potential for grout hardening inside the tailskin pipes was minimized. Nevertheless periodic maintenance was still required although the cleaning required much less effort and less time.

STICKY MUCK

The soft clayshale was a rock favorable for cutting, with low abrasion, and good penetrations under a low cutterhead thrust. However, the clay fines, which originated from the fragmentation of the clayshale, in the presence of water, turned the muck into a sticky material.

In the planning stages the rock was expected to disintegrate into a cohesionless muck containing some fines and larger amount of hard rock chips produced by the disk cutters, and indeed, during the initial EPBM drive of the first 75 meters an attempt was made to produce a "dry" cohesionless muck without foams and without additional water. However that was possible only in the rock with none or minimum ground water seepage. The effort to keep the muck dry failed with the varying groundwater ingress. The crews had to fight either the stiff clayey chunks overloading the screw conveyor or the muck slurry, which poured from the belt on the tunnel floor after the chamber was filled with groundwater during a standstill. The excavation was slow when the sticky muck formed a stiff cake at the cutterhead center, which prevented cutters penetration into the rock. A full stop to excavation took place when the muck finally plugged all the cutterhead openings, and no muck could be transported from the face.

After 75 m of excavation it became clear that excavation in the "dry mode" was not possible in the given conditions, and that turning the clayshale rock into a plastic mud by adding several cubic meters of water and foam was unavoidable, but also the best solution, Figure 16.

Figure 16. Turning the clayshale rock into a plastic mud by adding several cubic meters of water and foam was unavoidable but also the best solution

The reason for the muck stickiness and the cutterhead clogging was the clay fines content of more than 30%, which was produced in the softer than expected rock, while the high water consumption (10–16 m^3 per 1.5 m of advance) was due to the clayshale natural water content of 5–7%.

CONCLUSIONS

The EPBM tunneling success was not measured in ideal conditions of long uninterrupted drives, or homogeneous geology, where EPBMs worked the best, and reached their full production capacity. On the contrary, the adverse conditions of drives frequently interrupted by underground stations, in non-homogeneous geology, in which muck consistency varied from liquid to sticky, stiff lumps, in which the support pressures fluctuated, and which was highly deformable and led to surface settlements, put the operators and the technicians skills to test.

The strict 10 mm settlement criterion imposed on the surface above the both EPBM tubes proved to be tough due to the deformation of unsupported tunnel walls above the shield, and the ground water fluctuations caused by foam/air penetrating the ground. Nevertheless, the increased settlements were local, and did not have any negative impact.

The PLC (Programmable Logic Controller), and computer system continuously recorded data from the sensors located in all vital mechanical devices, which allowed data analyses to find the problem sources and to optimize the EPBMs drives. Thus, the EPBM mechanized concept allowed to learn about tunneling through observing the machine behavior.

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THE NEW KAISER WILHELM TUNNEL IN GERMANY'S PICTURESQUE MOSEL VALLEY: DUAL-MODE TBM TUNNELLING UNDER THE CITY OF COCHEM

Marco Reith ▪ Alpine BeMo Tunnelling GmbH

ABSTRACT

The Deutsche Bahn AG built a new tunnel parallel to the existing 150-year-old Kaiser Wilhelm Tunnel in the Mosel Valley. Excavation of the 4,200-m-long new tunnel was performed by TBM in open and closed mode with frequent mode changes. Finalization of the mechanized drive for the New Kaiser Wilhelm Tunnel was an important step towards installing the safety standards currently required for railway tunnels in Germany. This paper deals with the difficult EPB mode when tunneling under Cochem's upper town. The demanding geological conditions and very short distances between tunnel roof and building foundations posed the big challenges on this project. A special concept—differing from the contract—was thus jointly developed by the contractor and the client to master this very difficult part of excavation.

PROJECT DESCRIPTION

The double-track Kaiser Wilhelm Tunnel, originally opened in 1879, is located on the Coblenz-Perl Mosel rail line between the picturesque Ediger-Eller and Cochem (Figures 1, 2, 3 and 4). This railway connection is an important component of the Trans-European Network (TEN) for conventional rail traffic. Its structural status and the inadequacies of its safety standards with regard to fire and disaster protection made it

Figure 1. View of the portal of the New Kaiser Wilhelm Tunnel in Cochem (Source: ABT, 2012b)

Figure 2. View of the TBM's starting position at the Ediger-Eller portal (Source: ABT, 2010)

Figure 3. Site installation at the Ediger-Eller portal (Source: ABT, 2011)

necessary to build a second tunnel. On completion the combination of a new tunnel and a refurbished old tunnel would ensure a tunnel system that meets all the requirements of a modern traffic network.

The second tunnel tube of the Kaiser Wilhelm Tunnel has a length of approx. 4,200m and the overburden above the tunnel crown ranges between 3m and 230m. The axis of the second tube runs 25m east of the existing tube. The typical cross-section of the new tunnel is shown in Figure 5.

Refurbishment of the tunnel is being conducted in two phases. In the first phase the Old Kaiser Wilhelm Tunnel is given a second parallel single-track tube (New Kaiser Wilhelm Tunnel). In the second phase the original tunnel is being reinstated and modified as a single-track tunnel.

Figure 4. Schematic view of the existing tunnel (first tube) and the new tunnel (second tube) (Source: ABT, 2012b based on Google Maps)

When completed, each tube will be operated as a single-track tunnel. The two tubes will be connected by eight cross-passages at regular intervals of approx. 500m and will thus comply with the latest safety standards (TSI-SRT, 2008/2012), and guidelines issued by the German Federal Railway Authority (EBA, 1997/2001).

The double-track, twin-tube tunnel is planned to go into operation in December 2015. Mechanized driving for the 4,242-m-long New Kaiser Wilhelm Tunnel was successfully concluded with the breakthrough on November 7, 2011 (Figures 6 and 7).

The technically most difficult part of the mechanized drive, namely in terms of tunneling, was the closed-mode EPB undertunneling of Cochem's upper town (Figure 8). In some places the tunnel roof came as close as 3.2m to the foundation of buildings above it and settlement had to be kept to a minimum. This was the first time that this tunneling method was applied anywhere in the world to tunnel under a residential area with so little overburden. The task was made all the more difficult by the fact that there

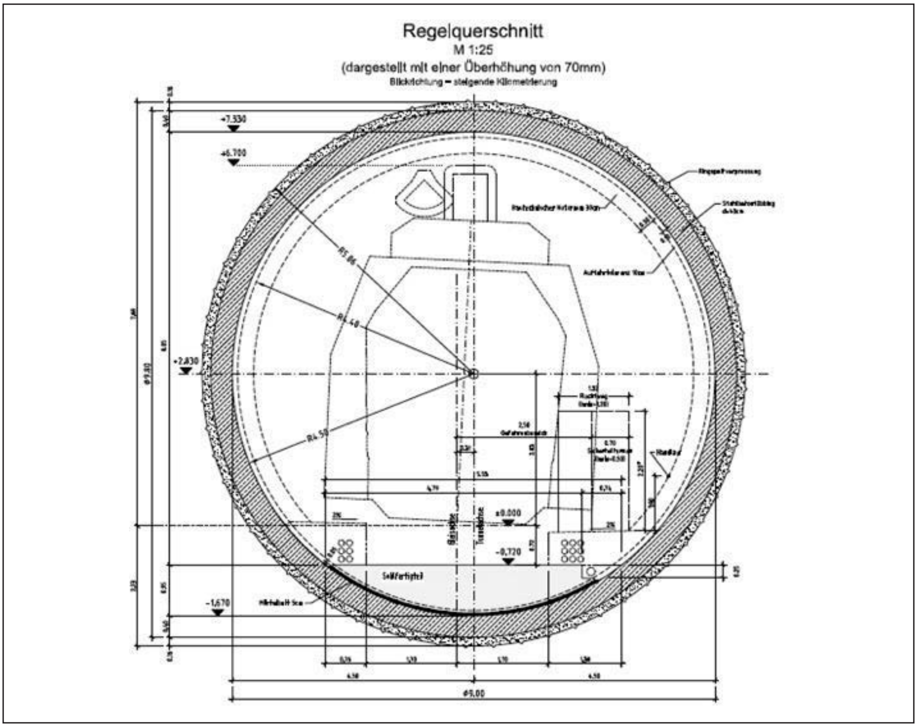

Figure 5. Typical cross-section of the new tunnel (second tube) (Source: ABT, 2012a)

Figure 6. TBM breakthrough at the Cochem portal (Source: ABT, 2011)

were mixed face conditions in this area: on the one hand, the prevailing hard rock face in the invert and, on the other hand, soft ground at the top of the tunnel face. The structural state of the affected buildings was reviewed from the pre-construction building protection program and assessed as critical for undertunneling.

The shield machine was devised to cope with the relatively stable solid rock zones encountered along the bulk of the route, which were to be tackled in open mode. However it was also possible to apply active face support to overcome fault zones and the soft ground under Cochem’s upper town. Consequently, the machine was equipped

Figure 7. View of the New Kaiser Wilhelm Tunnel at Cochem portal (Source: ABT, 2011)

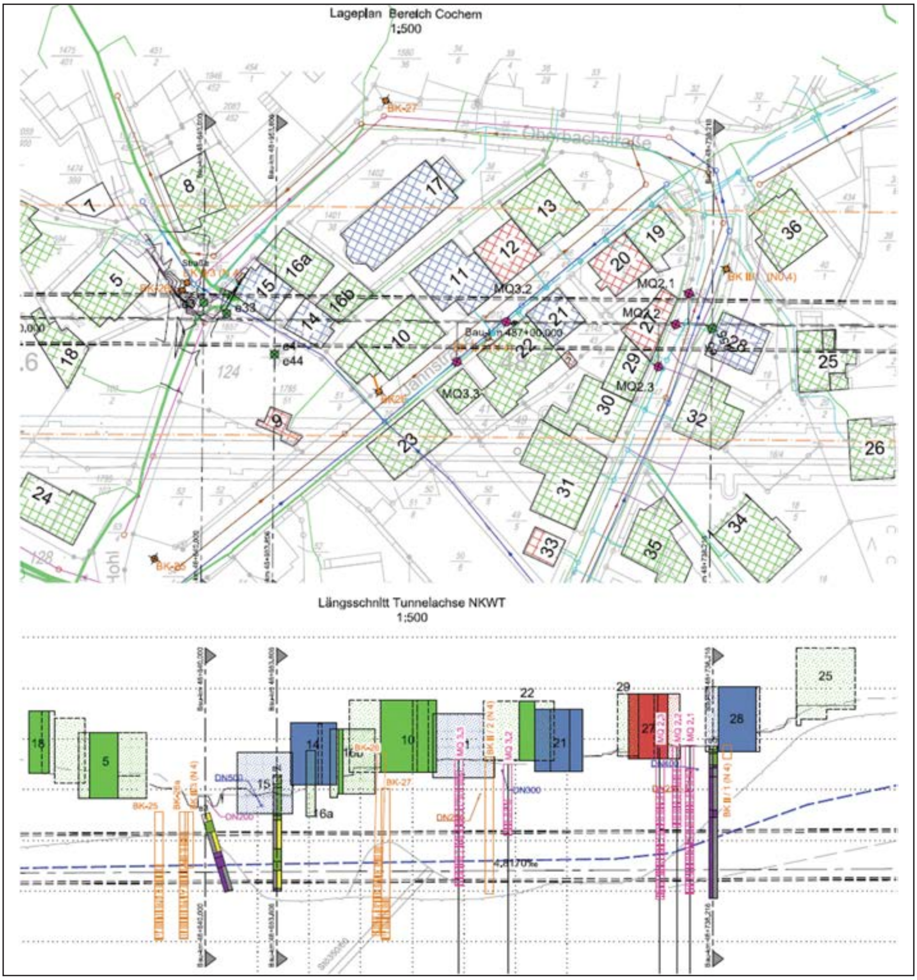

Figure 8. Surface situation and geological longitudinal section for Cochem's upper town (Source: ABT, 2012a)

with a screw conveyor that could be converted at any time from open mode to pressurized closed mode and vice versa. Operational advantages called for an EPB shield machine instead of a shield machine with fluid-supported face.

As described, by Handke et al. (2011), changing from one operating mode to the other was possible in a rather short time and without major modifications. The authors also give general project descriptions of the tunnel and the soil conditions, as well as the mechanical engineering approach, the findings obtained when excavating in solid rock and in the transition areas between stable solid rock and instable, highly fissured rock.

This paper deals exclusively with the findings obtained during the preparation and execution stages for tunneling under Cochem's upper town.

FORECAST FOR GEOLOGY UNDER COCHEM'S UPPER TOWN

The roughly 450-m-long area under Cochem's upper town is characterized by layers of soft ground consisting of quaternary slope loam or slope debris of varying thickness. These soils are highly susceptible to settlement and adopt flow characteristics when affected by water. The layers are embedded in an extensive depression enclosed by sections of solid rock.

According to the geotechnical prognosis, driving under Cochem's upper town would dip from the solid rock into the soft ground layers, until the tunnel cross-section was completely surrounded by the soft ground layers. The mixed-face conditions made steering the shield machine and ensuring face stability all the more demanding. The solid rock is characterized by relatively stable conditions. In the transition zone the solid rock gradually changes to fissured rock. Fissure systems and thick beds as well as accumulations of clay or silt on bedding planes lead to slicken-side surfaces that favor the detachment of blocks during driving.

PLANNED CONCEPT FOR DRIVING UNDER COCHEM'S UPPER TOWN

The drive under Cochem's upper town was to be executed in closed mode to ensure face stability and as little settlement as possible. To meet these demands the machine was designed as follows:

- cutting wheel largely closed
- cutting wheel can be displaced to permit retrieval of the cutting wheel.
- cutting wheel with rim
- integrated mass balance systems for extraction (belt weighing system) and annular gap grouting
- conicity and overcut can be limited
- essential machine components are checked for proper functioning, backed up by optical and acoustic alarms to ensure that errors are remedied as quickly as possible
- all data from the machine drive are collected electronically and transmitted online
- buildings are monitored continuously and data transmitted online with predefined trigger levels for contingency arrangements

If necessary, it was also planned to partially support the roof zone by means of a grout curtain installed from the machine. In this way support measures could be installed directly from inside the tunnel. Grouting from the surface was difficult because of the limited accessibility.

EXECUTION CONCEPT

Using the available geological information a multi-stage program was designed well before starting to drive under Cochem's upper town. The goal was to ensure regular construction procedures as well as safe and steady driving.

The program's main elements were to:

- assess the existing geological information and if necessary stipulate additional exploration
- analyze the buildings' ability to cope with settlement
- define machine and method precautions to ensure problem-free driving
- define necessary monitoring activities
- implement a geotechnical measurement concept to be performed on the surface by permanently surveying the individual buildings, including real time transmission to the TBM control panel
- conduct pretests for conditioning of excavated material to reduce clogging and stickiness
- conduct in-situ tests of the ground's groutability
- perform additional examination of the building geometry of the critical buildings

Assessment of Geological Information

Analysis of the available geological information showed the necessity to locate the rock horizon more precisely after performing additional exploratory drilling. The goal was not only to obtain further information on the elevation of the rock horizon, but also to have a better knowledge of the thickness of the rock layer in the tunnel cross-section. Moreover, the geotechnical investigation performed parallel to excavation in agreement with the client served to correct the soil parameters forecast (φ, c, E module, grain-size distribution curves, porosity), the stratification and the soil structure in relation to the tunnel's cross-section.

The six additional exploratory drillings indicated that mixed-face conditions prevailed in the cross-section over approx. 230m of the planned tunnel. These ranged from solid rock (clay slate, quartzitic fine sandstone) to soft ground (slope loam, silt, slope debris). In the remaining area to be undertunneled the tunnel cross-section was completely located in solid rock (Figure 8).

As far as grain-size distribution was concerned in comparison with the prognosis, the slope loam emerged as a sandy silt containing stones, whereas in the case of the slope debris the silt merely filled the porous areas between the stony and gravelly grain fractions. Given a silt proportion of ≥10%, these soils are highly susceptible to movement and moisten quickly, e.g., when driven over several times. Moreover, from a silt proportion of ≥20% groundwater or water inflow would make the soil flow-susceptible.

This information was used to assess the settlement compatibility of the critical buildings with shallow overburden and showed that additional support would be needed to drive under the most endangered buildings safely. This concerned a total of some 70m of the 450-m-long section to be excavated under Cochem's upper town. Test grouting to improve the soil revealed that the silt-dominated layers were difficult to grout, as had been forecast, but that the ground was nevertheless able to take a large amount of grouting material thanks to its heterogeneity and pore volume.

Improving the Soil

Following an extensive risk evaluation of the buildings with the goal of minimizing deformation resulting from excavation, it was decided to improve the ground over the critical

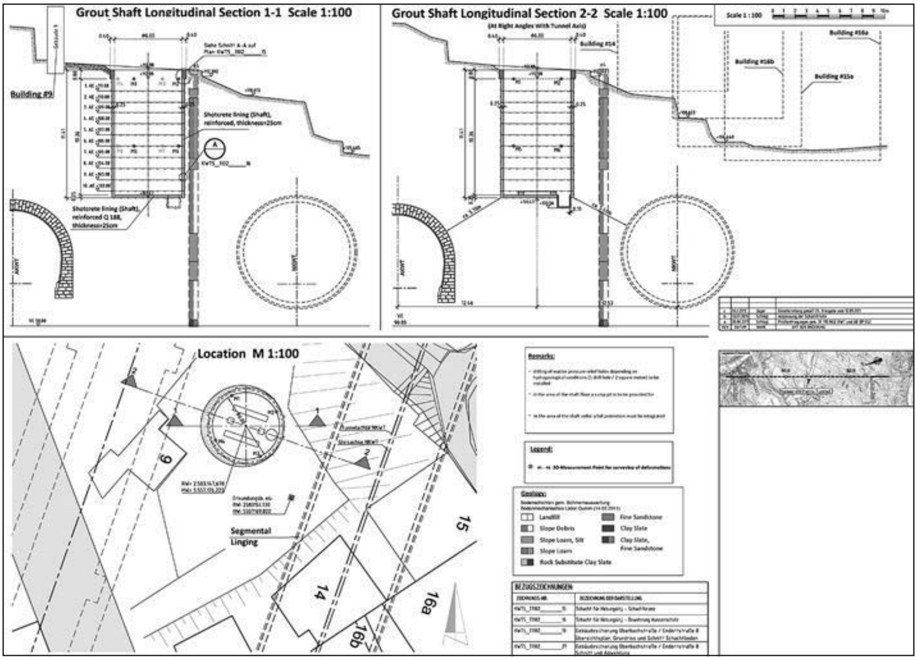

Figure 9. Grout shaft located between the old and the new Kaiser Wilhelm Tunnels (Source: ABT, 2012b)

length of roughly 70m by means of a grout curtain. The grout curtain was installed about mid-way between the lowest part of the foundations and the tunnel crown, namely working from the surface through a shaft located between the Old and the New Kaiser Wilhelm Tunnels (Figure 9). The minimum distance between the tunnel roof and the lowest part of the building foundations was as little as 3 m in some places.

The grouting shaft was produced using reinforced shotcrete and a strengthening ring at the shaft head. The shaft had a clear diameter of 6m and a depth of 13m. Any build-up of water pressure was prevented by drilling relief holes.

Excavation without any advance support along this section was not possible because of the risk to the buildings. Direct underpinning of the buildings, e.g., by installing a pipe umbrella support from special shafts with the buildings secured by supporting jacks, was ruled out as unsuitable because it would entail a large number of points of attack, restrutting and deformation. Safe tunneling under the upper town using a grout curtain would need a defined amount of pre-compensation lift. Depending on the degree of settlement, the possibility to compensate differences had to be foreseen. The Soilfrac method developed by Keller Grundbau GmbH was applied, and the permissible lift calculated by structural analysis was precisely adhered to.

No additional supports were needed for the remaining drive under Cochem's upper town.

Machine and Method Precautions for Tunneling Under Cochem

The basic concept for safely tunneling under Cochem's upper town called for continuous 24/7 operation without any scheduled breaks in tunneling operations. As a consequence, the following pre-emptive steps were taken well ahead of actual driving:

- All machines, particularly the cutting wheel, screw conveyor, foam lances for adding additives, underwent maintenance and were checked for proper functioning. The belt weigher was calibrated. Documentation and immediate repair of recognized problems were performed.
- Additional cutters were installed to cope with the slope debris and slope loam layers.
- Grill bars were replaced.
- Cutting discs were replaced with special disc cutters of high-grade steel and a double seal containing lubricant to prevent blockage.
- Compressed air lock was tested.

In order to ensure that the earth paste and its properties were optimally adapted to the geological conditions and that clogging and stickiness would be reduced, extensive conditioning tests were performed before driving was commenced. In addition to the lab tests, two test sections with actual driving conditions were set up to examine application to the actual geology and evaluate performance under driving conditions.

The tests showed that the conditioning agent Rheosoil 143 by BASF AG (anti-clay polymer) met the demands. The ground was sufficiently plasticized to ensure that the extraction chamber could be completely filled and the pressure maintained. This was achieved in both solid rock and soft ground. At the same time adhesion and cohesion were sufficiently reduced. To monitor temperature development in the earth paste, two temperature sensors were installed in the extraction chamber. Openings for roof ventilation were made to permit foam injections and prevent accumulation of foam in the roof. In this way, clearly defined support pressure conditions could also be achieved in the roof.

Water, bentonite and foam additive and the effective addition of compressed air were finely adjusted according to the power consumed by the cutter head engine. Preliminary tests showed 60% to 80% to be favorable.

Precautions During Execution and Construction Operations

To interpret the pre-construction building protection program a geology and buildings monitoring program (tube water levels, measurement bolts) with the corresponding trigger levels for contingency arrangements was defined. In addition, the deformation behavior of the segmental lining including any changes in joint displacement and gap dimensions was monitored. The measurement data were recorded and transmitted electronically. Moreover, on a separate monitor the shield operator was continuously informed of the measurements and their interpretation. For the shield operator the comprehensive data were reduced to the bare necessity by means of integrated alarm systems.

The support pressure and machine operating settings (e.g., contact pressing force, torque and rpm of cutting head, rate of advance, cutting head displacement, overcut, mass balance, engine power consumption, thrust cylinder pressure, grout injection pressure, settings for foam unit) needed for driving were continuously updated and transmitted directly to the shield operator by the site management in the form of special instructions. The entire driving crew received repeated training and instructions to ensure safe tunneling under Cochem's upper town.

As a fallback plan equipment and materials (drilling tools, conditioning agents) were kept on site including for possible contingency measures (pipe umbrella, grouting material, shotcrete, silica foam).

Local residents were integrated in the technical implementation and realization process at several information evenings and in personal conversations. Response was positive.

If partial settlement and face inspections were to become necessary despite all the precautions taken, the ground would be saturated with bentonite before switching from the earth paste-supported mode to the compressed air-supported mode.

Special catalogs outlined not only the pre-emptive measures, but also the steps to be taken in the event that a contingency occurred.

In such a case it was also planned to perform regrouting through the segmental lining, if needed.

An on-site decision-making team was set up for the critical undertunneling section. It consisted of the client's project management and on-site supervision, the JV's project management and the client's advisors. In addition, an alarm plan with the defined reporting chain was drawn up. The undertunneling project was executed on the basis of the planning documents and structural analysis checked by EBA Test Engineer Dipl.-Ing. Reinhold Maidl and approved by DB ProjektBau GmbH, especially for support pressure and annular gap grouting.

FINDINGS MADE DURING EXECUTION

Support pressures were optimally adjusted by altering the pressure on the earth paste and the annular gap grouting pressure against the ground and the deformation behavior of the buildings. Driving was thus able to proceed continuously and problem-free with settlement far below the forecasts.

Readjustment and regrouting from the shaft to compensate for settlement was limited to a few places where only minimal compensation lifting was necessary. The transitions from rock to soft ground and vice versa proceeded problem-free by promptly and sensitively using the open and closed operating modes. This caused only little deformation. The machine operating data were continuously adjusted to the ground deformation behavior. Chamber inspections with a consequent reduction in the earth paste level were not needed. Hazardous incident scenarios defined in advance on the basis of a comprehensive analysis were safely controlled thanks to sensitive monitoring and pre-emptive measures instituted early, even before driving began. In addition to planned coping measures, a catalog of steps to be taken was drawn up as a fallback solution. For regular operation in open mode, the fallback solution was to switch to closed mode. Further-reaching fallback levels such as additional ground improvements, which could also be performed from inside the machine, were not needed.

SUMMARY

Undertunneling Cochem's upper town posed high demands on machine design and the driving crew. The execution concept was worked out in intense discussions between the client, its advisors and the contractor to ensure that it was optimally adapted to the ground and building properties. Optimal preparation, coordination and close cooperation on the part of all parties involved were key components in the successful execution of this technically challenging project (Figure 10).

In conclusion, it can be stated that tunneling under Cochem's upper town with residential buildings that are highly sensitive to settlement and have minimal distances between their foundations and the roof of the tunnel was accomplished safely and without any serious problems affecting the stability of the face. Driving was performed in EPB mode and proved to be a highly flexible and useful method under the given circumstances. Buildings incurred only slight settlement damage in the form of cracks. The segmental lining was installed in very high quality and fully meets the demands made of a watertight structure.

Figure 10. View into the tunnel showing the TBM back-up (Source: ABT, 2011)

ACKNOWLEDGMENTS

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TRENDS IN THE PERFORMANCE OF METRO-SIZED EPB TBMs: A STUDY OF WORLDWIDE EPB ADVANCE RATES

Desiree Willis ▪ The Robbins Company

ABSTRACT

With massive rail infrastructure projects such as Singapore's Downtown Line requiring dozens of TBMs now underway, it is imperative that performance of these machines is documented and analyzed in the field. By learning from real-world examples of TBM use in soft and mixed ground, patterns for success may be extrapolated. Using data from worldwide rail projects and multiple machine suppliers, this paper will discuss the performance of metro-sized (6 to 7 m diameter) Earth Pressure Balance (EPB) TBMs. Trends will be analyzed and conclusions will be drawn with regard to optimal machine design, operation, and overall project success.

INTRODUCTION

EPB s continue to be the most common TBM type used in the tunneling industry, and the use of these soft and mixed ground machines is only projected to increase. Simultaneous usage for metro projects in Singapore, Russia, China, the U.S., Canada, and elsewhere are on the forefront of this trend. Because of the increase in large metros, it was decided to focus on the 6 to 7 m diameter size range for this paper, with a few outlying examples.

Project owners and contractors are of course interested in speeding up tunnel construction by optimizing TBM advance rates without compromising quality or safety. Doing that, however, is neither easy nor predictable. For this paper, we analyzed advance rates from metro-sized projects worldwide, and combined this with interviews from experts in the tunneling industry. These anonymous experts range from contractors to consultants to machine designers. Using their opinions and experience, we will extrapolate several trends affecting EPB advance.

EPB PROJECT ADVANCE RATES

Analysis of 28 machines from multiple TBM manufacturers on various TBM projects uncovered a wide array of advance rates. These average weekly advance rates in meters are grouped by TBM diameter (see Figure 1). Advance rates ranged from a low of 33.7 m to a high of 135 m.

To further differentiate these projects, the rates were grouped by ground conditions, either mostly uniform soft ground or mostly varied mixed ground (recognizing that almost no projects are truly one or the other). When this was done, many of the lower ranking advance rates were eliminated, implying that the complexity of ground conditions is a significant factor (see Figure 2). The average among these soft ground projects was 102.9 m, compared with an overall average of 74.6 m for mixed ground projects. Interestingly, some of the highest performing projects were in mixed ground, however. This wide-ranging result indicates that there may be an equally wide ranging number of variables that figure into the determination of advance rates.

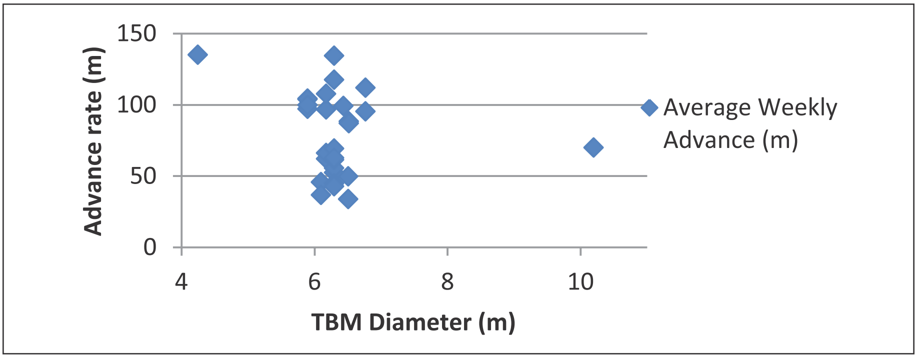

Figure 1. Average weekly advance rates by TBM diameter

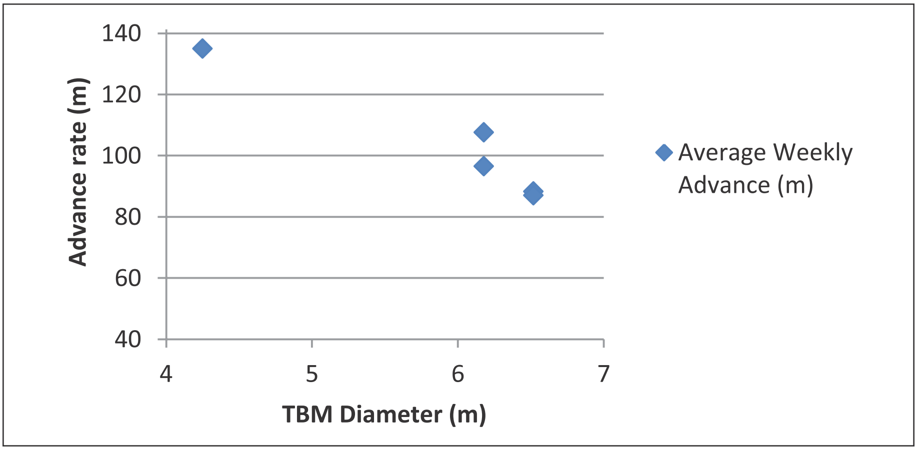

Figure 2. Average weekly advance rates in majority of soft ground

THE EXPERT OPINION

After speaking with a team of experts regarding the factors that determine EPB advance rates, the consensus is clear: there is no consensus. Many factors come into play, but the ones mentioned the most often were: TBM Design, Ground Conditioning, Ground Conditions, Tunnel Length/Profile, and Operator Training.

An interesting addition to this is that not all tunnel markets give advance rates equal priority. Some project owners may limit advance if tunnel quality comes into question or if there is a schedule to adhere to (e.g., a station site will not be ready for the machine breakthrough, so the TBM must slow down). In the most significant example, all TBM projects in Japan are limited to a flat mm/min rate and deviation too far in either direction is penalized.

Tunnel length is another factor that can certainly be seen in some of the lower advance rates examined in this paper. Many of the lower performing projects were in China, in short tunnels with multiple station breakthroughs. In many of these projects, the machine must wait before breaking through into the cut and cover station site, or is delayed before its next launch in order to give time for subsequent station construction. As such, advance rates ramp up and ramp down on each section of the drive even if stoppages are not included in the overall rate.

Ground conditions are necessarily a huge factor that comes into play, and having an accurate GBR can make all the difference, from adequate tunnel design to TBM specification. Ground conditions that are more prone to settlement, or that switch from hard to soft ground can be difficult to excavate swiftly. The more complex the ground conditions, the more difficult a fast advance is to achieve, but this condition alone doesn't make it impossible. The presence of widely varying advance rates in mixed ground indicates that there are differences in TBM design and operation the can make a project successful even in difficult conditions.

For the purposes of this paper, we will focus on the factors that contractors can affect and change, rather than factors that are pre-determined. These are TBM Design, Ground Conditioning, and Training.

OPTIMAL TBM DESIGN

TBM design is necessarily determined by the tunnel requirements and accuracy of the Geotechnical Baseline Report (GBR). Cutterheads for mixed ground may use a combination of cutting tools as well as a lower opening ratio, for example, while a TBM in primarily soft ground may use carbide bits and an open spoke-type head.

Minimizing Downtime

Guaranteeing fast advance is a function of two aspects: fast machine design and minimization of downtime. A machine with a rugged structure, which does not compromise on steel, is more likely to survive mixed ground with sections of rock, for example. Abrasion resistant plating welded to a cutterhead structure can enable EPB machines to excavate abrasive rock sections such as basalt, with minimal wear damage.

Monitoring of the cutters themselves, through specialized wear detection bits, is another way of minimizing downtime. The installation of wear detectors at varying heights on the face of the cutterhead can give warning of excessive wear or of the need to replace cutting tools before damage occurs to the cutterhead structure. Wear detectors use hydraulic pressure that is released when a certain amount of wear occurs and the hydraulic line is sheared, sending a signal to the machine operator (see Figure 3).

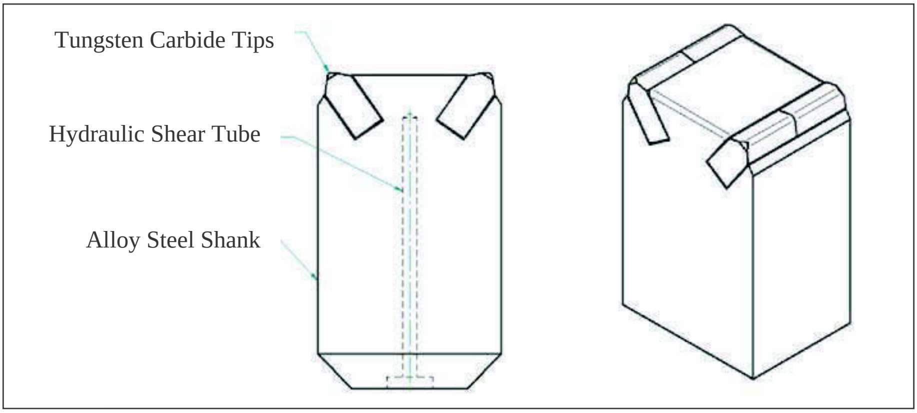

Figure 3. Typical wear detection bit

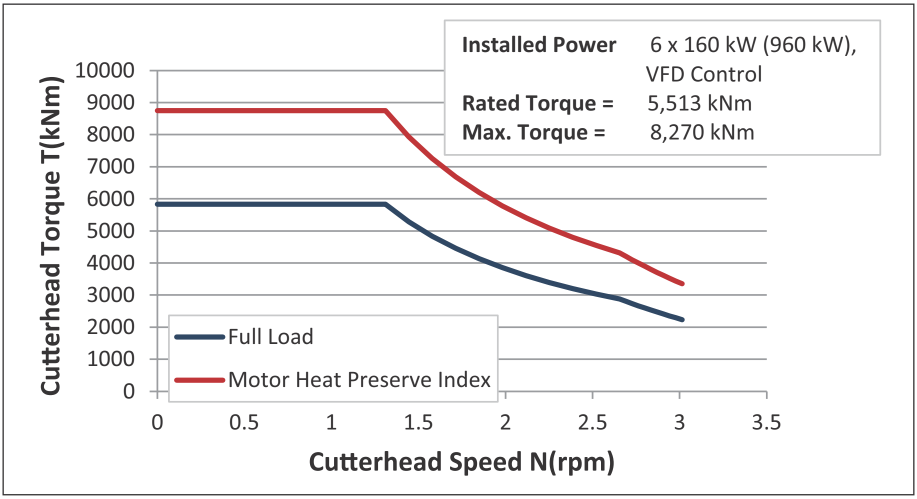

Figure 4. Speed/torque curve for VFDs

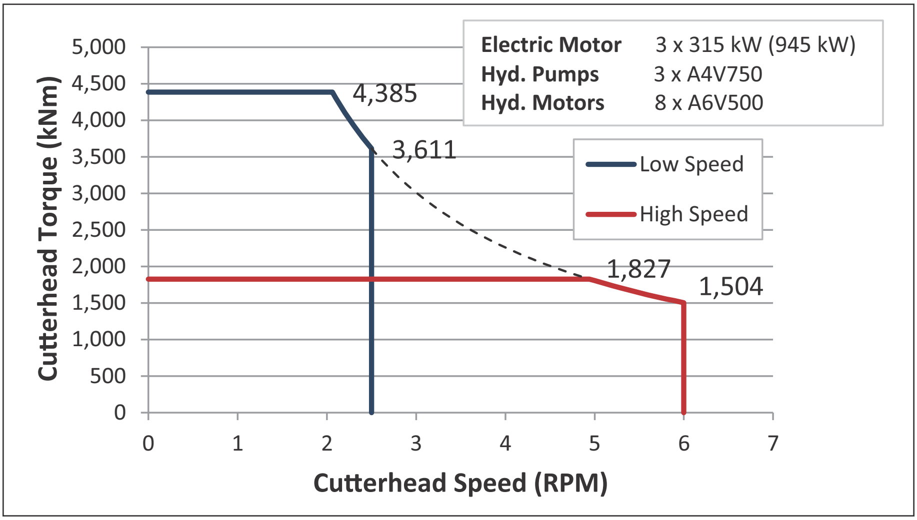

Figure 5. Speed/torque curve for hydraulic drives

Hydraulic vs. Variable Frequency Electric Drives

Unsurprisingly, much of what determines the advance rate of an EPB is the type of motor that drives it. The debate between the usage of hydraulic drives vs. electric Variable Frequency Drives (VFDs) is one based on tradeoffs.

Though more compact, system availability tends to be less for hydraulic systems—about 75%, compared to more than 90% average availability for electric drives. Hydraulic drives are also harder to keep at maximum efficiency, as the oil often becomes contaminated, which drastically lowers efficiency. In addition, hydraulic systems require strict humidity control and regular maintenance.

Given these shortcomings, why would one choose to use a hydraulic drive? Hydraulic drives have a lower initial cost than VFDs, though when maintenance costs

Table 1. Benefits and drawbacks of VFD vs. hydraulic drives

Electrical Drive	Hydraulic Drive
1. High total efficiency, 90–95% 2. Little maintenance 3. Low noise 4. Low heating because of high efficiency 5. Speed control requires simple integration to machine's automation system (PLC). 6. High starting torque (100–150%, rated torque) 7. Overload operation is possible within short time. 8. Larger space is necessary for installation (difficult to use below 5 m diameter).	1. Low total efficiency, 65% 2. Lots of maintenance are required for oil contamination, temperature, leakage, etc. 3. High noise 4. High heating from hydraulic devices 5. Speed control is easy. 6. Low starting torque (70–90% of rated torque) 7. Overload operation is impossible. 8. Smaller space is necessary for installation.

and lower efficiency are factored in they have a higher cost over time. Hydraulic drives also require less space than VFDs, which generally cannot be used on machines below 5 m in diameter. For smaller EPBs, hydraulic drives may be the only choice.

Over the past 20 years, high-efficiency VFDs have become more popular for use in TBMs. The reliability and base of use, as well as the decrease in cost, have now shifted the VFD into a commodity product class similar to circuit breakers and motor starters. While VFDs have a high efficiency limit in terms of speed, a higher cutterhead rotation alone does not equate to faster advance in soft ground.

During excavation, EPB cutterhead rotation is kept low (around 1.5 rpm at maximum), in stark contrast to the higher speeds (around 10+ rpm maximum) used in similar diameter hard rock TBM tunneling. In hard rock, high rpm results in fast advance, while in soft ground high rotational speed often results in ground disturbance and surface settlement of non-self-supporting geology. In soft ground, the same result of high advance rates can instead be achieved by increasing the cutterhead torque, or α value, and thrust, which increases the instantaneous rate of penetration (see Figures 4–5, Speed vs. Torque curves for VFD and Hydraulic Drives).

These relationships can also be theoretically described with the following set of equations:

$$L = 2\pi/60 * T * N$$
$$T \propto Pe^2$$
$$V \approx Pe * N$$

where:

T = torque in kNm
L = power in kW
N = rotational speed in min^{-1}
Pe = penetration rate
V = advance rate

As can be seen in the above equations, torque is directly proportional to the square of the penetration rate, and the overall advance rate is a function of both the penetration rate and rotational speed. Therefore, a fast advance requires both high speed and high torque in rock/mixed ground, though in soft ground this is limited because rotational speed must be kept low to avoid ground disturbance. A summary of the overall benefits and drawbacks of the two motor types is also shown in Table 1.

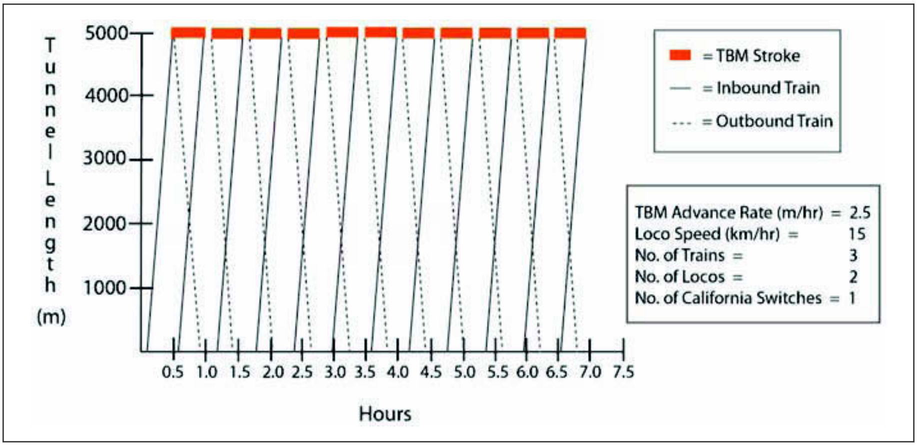

Figure 6. Logistical planning graph for muck car usage

Continuous Conveyors

Most experts agree that for longer EPB tunnels, continuous conveyors are a significant logistical factor that can increase advance rates. The availability of muck cars goes down as tunnel length increases. Cars traveling into and out of the tunnel must be accurately timed with each TBM push, and this can lead to increased TBM downtime in longer tunnels if not properly planned (see Figure 6, an example of logistical planning for a 5 km tunnel).

While conveyors in soft and mixed ground conditions are gaining acceptance, muck cars are still considered the standard. In tunnels below 2 km, they are certainly more efficient, but in longer tunnels, less so. It should be noted that the project with the highest average advance of 135 m did use a continuous conveyor system rather than muck cars.

Segments

The choice of segment type may have some bearing on advance rates, particularly when bolting is considered. Many projects opt not to use bolted segments, in favor of pin-type joints for a faster ring build. While the timing of each has not been studied in detail, it is an interesting observation.

THE ROLE OF GROUND CONDITIONING

Ground conditioning can be equally as important as the machine design and logistical aspects. Additives are used to consolidate ground and maintain a smooth flow of muck through the cutterhead, thereby maintaining consistent earth pressure. Back-filling is further used to stabilize segments and prevent settlement behind the machine.

The use of ground conditioning at the cutterhead has further been shown to reduce wear and increase advance rates. The type of additive used, and indeed whether or not additive is needed at all, is determined by soil permeability, ground water pressure, and the risk of clogging/adhesion (Langmaack, 2006). These properties can be lumped into different kinds of soil and their particle size (see Figure 7, Japan Society of Civil Engineers, 2007).

Ground with less than 30% fines, or particles less than 0.2 mm in diameter, is difficult to fluidize. In this type of non-cohesive ground, Bentonite is used for consolidation. For other types of ground with fewer fines, foam consisting of water, surfactant, and

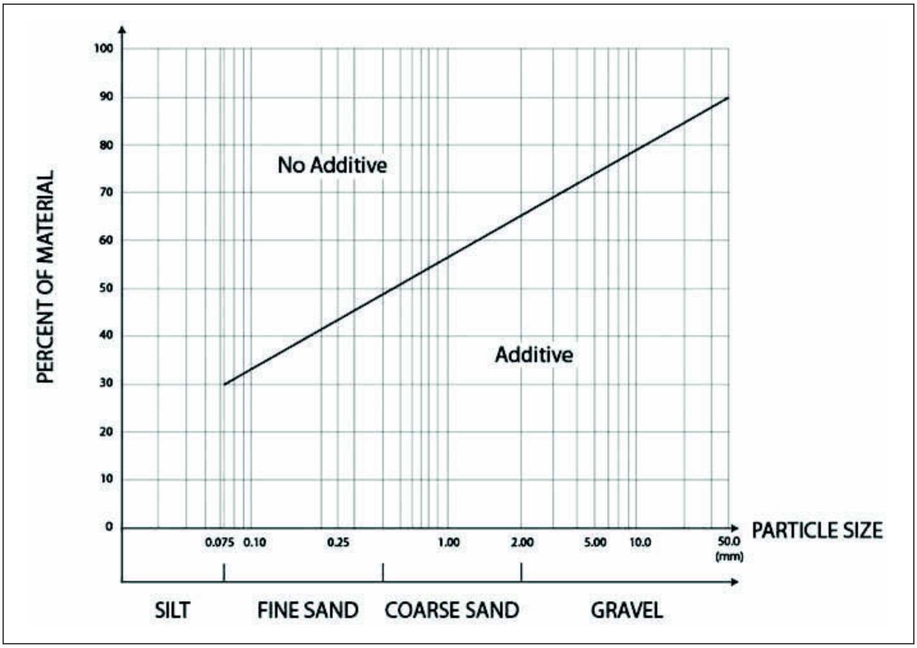

Figure 7. Distribution chart based on particle size and percent of material

additive is used. If water pressure is high and small particles are present, a polymer can be injected in addition to the foam to increase cohesiveness of the material.

The use of foam reduces the required cutterhead torque and reduces overall machine wear. Insufficient foam injection has been associated with increased thrust and required power, as well as higher cutter consumption, all of which can affect the advance rate.

THE IMPORTANCE OF TRAINING

While the experts interviewed for this paper had some differences in opinion, one factor they all agreed on was that crew/operator experience is invaluable and worker training is key. One expert for example, had this to say: “Operators must understand what an EPB is and how to operate it. It is essential to know the machine. Balancing thrust and torque while maintaining required pressures takes experience that can only come from hands on.”

In response to the obvious need Robbins developed a training program for EPBs (and all TBM Types) if one is requested by the contractor. Training programs may last anywhere from several days to a month, and cover both tested classroom courses on EPB function as well as hands-on training in the tunnel. This training at EPB projects such as Mexico’s Emisor Oriente Tunnels has shown to result in good TBM operation and improvements in machine performance. It is therefore recommended that if the experience of the crew is not optimal, training should be offered to supplement experience and provide the crew with an adequate knowledge base to operate and maintain the machine properly.

Figure 8. Breakthrough at Zhengzhou Metro

CASE STUDY: ZHENGZHOU METRO

China's Zhengzhou Metro Line 1 is a good example of the right variables coming together in the right proportions to produce fast advance rates. The two 6.3 m Robbins EPBs were designed for parallel 3.6 km tunnels in sand, clay, and pebbles. Properly trained crews from the 11th Bureau of contractor CRCC efficiently guided the machines through difficult tunnel sections including a section of low cover (7 m) in permeable, water-bearing soils below Xiliu Lake. Prior to entering the known section, crews checked all the machine systems and changed out the tail seals. Crews then carefully maintained earth pressures of between 1.1 and 1.3 bar while boring at a low cutterhead speed of 1 RPM below the lake, reducing advance rates in this section to 30 to 40 mm/minute.

Ground for much of the tunneling was under approximately 8 m of cover in soft and powdery soils, and below building foundations and a highway interchange. Settlement levels remained within limits during the entire drive.

Advance rates remained high throughout the project, with the machines averaging 96.6 m and 107.6 m per week, respectively, despite stopping and starting to break through into multiple cut and cover station sites. More importantly, one of the machines achieved a Chinese record in its size class of 6 to 7 meters, after excavating 720 m in one month.

The machines completed tunneling in October and November 2011, on schedule and with little cutter or machine wear due to the use of water and foam where needed (see Figure 8).

CONCLUSIONS

While we have focused on the aspects that contractors, manufacturers and project owners can change to optimize a project for fast advance, there will always be factors beyond anyone's control. The ground conditions play a vast part in determination of advance rates—high water pressures make for a slower advance for example, whereas if the geology is stable and the pressure low, a good advance can be made. To excavate a length of tunnel in soft ground, cutterhead rotation must be kept low with high torque, while the same length of tunnel in mixed ground including rock requires higher rotation that can increase cutter wear and abrasion and lead to downtime. Ground conditions also determine the amount of additive that may need to be injected, which can also affect the advance rate.

The question of EPB advance is obviously a complex one, but the overall focus should be on the parameters of the equation that can be changed, so that each project has the best chance possible to be safe, successful, and swift. With proper communication between all parties involved, the right decisions can be made for each variable, whether that involves the TBM design, machine operation, or worker training. In this way, the fastest advance that is possible for each project can be achieved.

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UNPRECEDENTED EPB EXCAVATION IN PRESSURIZED MIXED GROUND CONDITIONS: STUDY OF PERFORMANCE AT THE EMISOR ORIENTE WASTEWATER TUNNEL

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Andrei Olivares ▪ The Robbins Company

ABSTRACT

Mexico City's Emisor Oriente Wastewater Tunnel, a 63 km long mega project, is arguably one of the most challenging TBM tunnels in the world today. High water pressures, watery lake clays, mixed soil and rock, abrasive basalt, and boulders up to 800 mm in diameter combine to make the use of six 8.93 m diameter EPBs on this project not only challenging, but also unprecedented. This paper will analyze the excavation of Emisor Oriente Lots 1B and 3, some of the most difficult bores operating under tight excavation schedules. Startup configurations, advance rates, wear results, geological findings, muck removal using continuous conveyors, and any necessary hyperbaric interventions will also be discussed.

INTRODUCTION

Mexico's largest infrastructure project is a 62 km long emergency wastewater pipeline being built to prevent flooding in the downtown area of the capital. Ground settlement in Mexico City has caused the existing gravity feed wastewater system, built in 1975, to lose its slope. Much of the main pipeline, Emisor Central, is severely corroded and at high risk of failure, which could potentially cause up to 5 m of wastewater to flow into Mexico City's surface streets.

To remedy the problems, the Mexico National Water Commission, CONAGUA, released a contract for a 7.0 m ID long pipeline known as the Emisor Oriente Wastewater Tunnel. To meet the demanding schedule, six 8.93 m diameter Earth Pressure Balance (EPB) TBMs were required.

The ground conditions of the pipeline are some of the most difficult in the world. Located in the Valley of Mexico, geology consists of a drained lake bed with clays interspersed with volcanic rock and boulders from long dormant, buried volcanoes in the area. The TBMs will utilize knife edge and drag bits that can be changed out for 17-inch diameter disc cutters depending on the geology. Two-stage screw conveyors will help to regulate varying water pressures of 4 to 6 bars—some of the highest pressures EPBs have ever operated under. An initial 900 mm diameter ribbon-type screw conveyor will accommodate expected boulders up to 600 mm in diameter.

Despite its obvious need, the project is not without some controversy, particularly for the valley's farmers. Mexico's untreated wastewater currently flows through a system of open canals that feed much of its farmland. The lush bounty of crops is the direct result of fertilization by the so-called 'black waters,' which will be stopped by the commissioning of the new Emisor Oriente line and wastewater treatment plant.

HISTORY AND PROJECT BACKGROUND

The history of Mexico City is inextricably linked to the issue of its geographic location. Tenochtitlan, the ancient capital of the Mexica civilization, covered an estimated 8 to 13.5 km^2 (3.1 to 5.2 sq. mi), situated on the western side of the shallow Lake Texcoco.

The city was connected to the mainland by causeways leading north, south, and west of the city. These causeways were interrupted by bridges that allowed canoes and other traffic to pass freely. The bridges could be pulled away if necessary to defend the city. The city was interlaced with a series of canals, so that all sections of the city could be visited either on foot or via canoe.

After the Conquest, the Spanish rebuilt and renamed the city. The valley contained five original lakes called Lake Zumpango, Lake Xaltoca, Lake Xochimilco, Lake Chalco, and the largest, Texcoco, covering about 1,500 square kilometers (580 sq. mi) of the valley floor, but as the Spaniards expanded Mexico City, they began to drain the lake waters to "control flooding."

The idea of opening drainage canals first came about after a flood of the colonial city in 1555. The first canal, known as Nochistongo, was built in 1605 to drain the waters of Lake Zumpango north through Huehuetoca, which would also divert waters from the Cuautitlán River away from the lakes and toward the Tula River. Another canal, which would be dubbed the "Grand Canal" was built parallel to the Nochistongo one ending in Tequixquiac. The Grand Canal consists of one main canal, which measures 6.5 meters (21 ft) in diameter and 50 km (30 mi) long, and three secondary canals, built between 1856 and 1867. The canale was completed officially in 1894 although work continued thereafter. Despite the Grand Canal's drainage capacity, it did not solve the problem of flooding in the city. From the beginning of the 20th century, Mexico City began to sink rapidly and pumps needed to be installed in the Grand Canal, which before had drained the valley purely with gravity. Currently, and despite its age, the Grand Canal can still carry 2,400,000 US gallons per min (150 m^3/s) out of the valley, but this is significantly less than what it could carry as late as 1975 because continued sinking of the city (by as much as seven meters) weakens the system of water collectors and pumps.

As a result of the decreased capacity, another tunnel, called the Emisor Central, was built to carry wastewater. Although it was considered the most important drainage tunnel in the country, it has been damaged by overwork and corrosion of its 20 ft (6 m) diameter walls. Because of lack of maintenance and gradual decrease in this tunnel's ability to carry water, there was a big concern that this tunnel would eventually fail. It is continuously filled with water, making it impossible to inspect it for problems. If it fails, it will most likely be during the rainy season when it carries the most water, which would cause extensive flooding in the historic center, the airport and the boroughs on the east side of Mexico City.

Because of this main issue, Mexico City's Emisor Oriente Wastewater Tunnel was planned as part of Conagua's Mexico Valley Water Sustainability Program (PSHCVM). The project includes new pumping stations, and a 40 mile (62 km) drainage tunnel running from under Mexico City to the neighboring state of Hidalgo. Repairs to the current 7,400-mile (11,900 km) system of pipes and tunnels is also taking place to clear blockages and patch leaks.

The new tunnel will operate along with the existing Emisor Central tunnel. Discharge from the tunnels will be primarily treated at Atotonilco wastewater treatment plant, which is also under construction by Conagua (see Figure 1).

PROJECT DESIGN

The tunnel will have a diameter of 8.7 m at excavation and a diameter of seven meters once final lining takes place. It is being built 200 m below the surface level and will run

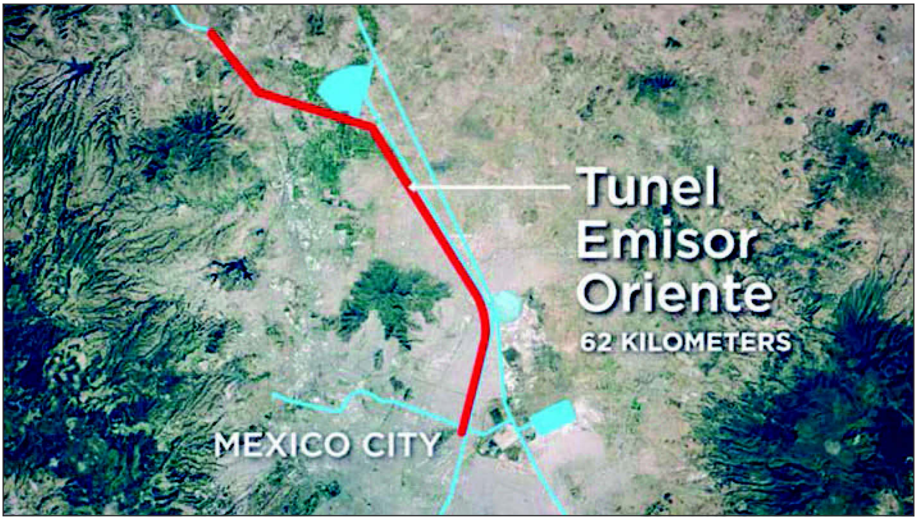

Figure 1. Layout of the Emisor Oriente wastewater tunnel

below three different urban districts. The tunnel includes 24 shafts, measuring 12–20 m in diameter at various depths between 30 m and 150 m (see Figure 2).

Construction of the tunnel has been divided into six lots measuring ten kilometers each. Drilling of the tunnel is being undertaken using six custom-designed Earth Pressure Balance (EPB) tunnel boring machines (TBMs). Lots 3, 4, and 5, as well as half of Lot 1 are being excavated using three Robbins 8.93 m machines while the remaining Lots are using three Herrenknecht machines (see Figure 3).

The TBMs have been designed to suit the complicated ground conditions in Mexico, which include high groundwater pressures up to 6 bar. Volcanic rocks, lake clays and massive boulders up to 600 mm in diameter are expected to be encountered during drilling operations. To keep the tunnel stable in the difficult conditions, it will be lined with steel and concrete reinforced segmental rings able to withstand varying earth pressures.

TEO Geological Conditions

Originally geology was based on 64 borehole tests conducted along the tunnel length, as well as six cross tunnel locations that were considered. The results:

- *Lot 1:* Quaternary lacustrine deposits of northern Mexico Basin.
- *Lot 2:* Basaltic ashes and pumice Quaternary strata, and northern flank lavas from Nochistongo.
- *Lot 3:* Clay from the Pre-Quaternary lacustrine Basin of Mexico.
- *Lot 4:* Fluvial Sands of the Plio-Quaternary Nochistongo Mountains.
- *Lot 5:* Pliocene volcanic formations from the upper part of Huehuetoca.
- *Lot 6:* Pliocene lacustrine deposits, Taximay medium and Taximay Superior.

With this basic information the machines were designed. In 2009 and 2010 when the excavation of the last shafts took place, it revealed a much more difficult geology than originally expected. These changes in geology resulted in modifications and new plans to complete this challenging tunnel that are now in process. The remainder of this paper will take the case of two lots (Lot 1B and Lot 3) to analyze the expected scenario

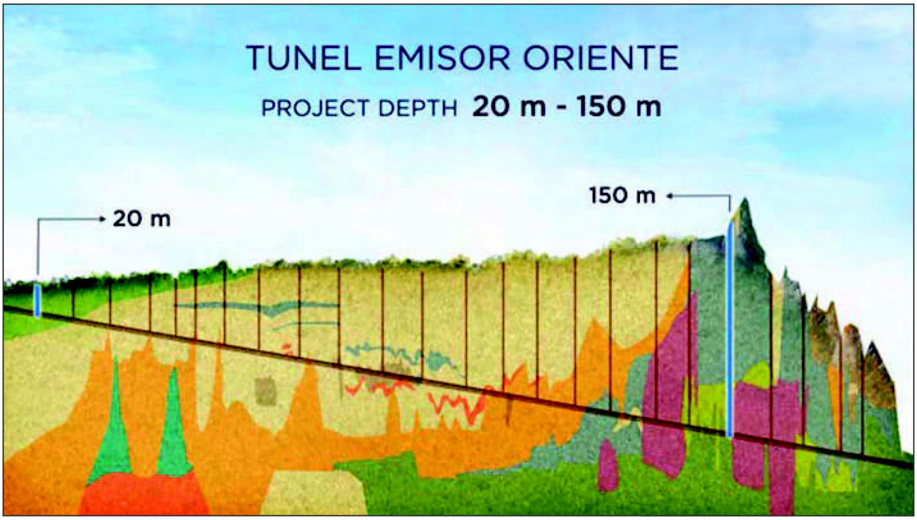

Figure 2. Project shafts and varying depths

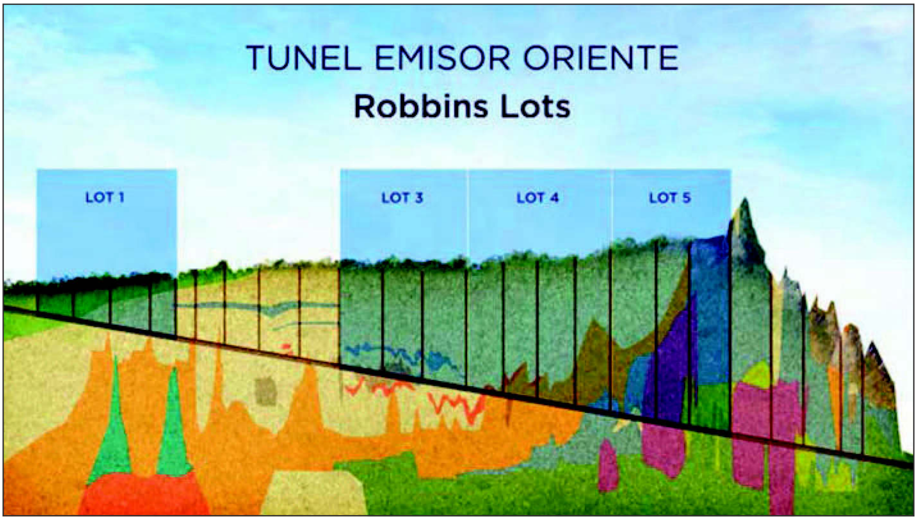

Figure 3. Robbins lots at the Emisor Oriente Tunnel

back in 2008 and update the project scenario, the geological conditions, the design of the machines and the upcoming challenges.

TBM DESIGN

The three Robbins machines were built for abrasive basalt sections up to 80 MPa UCS mixed with sections of watery clay that have been compared to a soup, with water pressure estimated in the range of 4 to 6 bar.

Adaptable Cutterheads

The custom designed EPBs were engineered with mixed ground, back-loading cutterheads to tack variable conditions. High pressure, tungsten carbide knife bits can be interchanged with 17-inch diameter carbide disc cutters depending on the ground conditions. During tunnelling a number of small shafts, spaced every 3 km between the larger launch shafts, can be used to perform cutter inspection and changes. Specialized wear detection bits lose pressure at specified wear points to notify crews of a needed cutting tool change. The knife edge bits are arranged at several different heights to allow for effective excavation at various levels of wear.

The design also allows for bearing and seal removal from either the front or back of the cutterhead. Twenty-five injection ports spaced around the periphery of the machine are used for injection of various additives depending on ground conditions, and for probe drilling. Additives such as Bentonite are currently being used to condition the muck for removal by belt conveyor (see Figure 4).

Figure 4. Mixed ground cutterhead for the Lot 1 machine

Two-Stage Screw Conveyor

High pressure conditions in concert with large boulders necessitated a two-stage screw conveyor design for the Emisor Oriente EPBs. An initial 900 mm diameter ribbon-type screw is capable of transporting boulders up to 600 mm in diameter up the center shaft for removal through a boulder collecting gate. Each of the three machines may encounter pressures of up to 6 bar, necessitating a two-screw setup with a ribbon screw and shaft-type screw in order to smoothly regulate pressure and maintain water-tightness (see Figure 5).

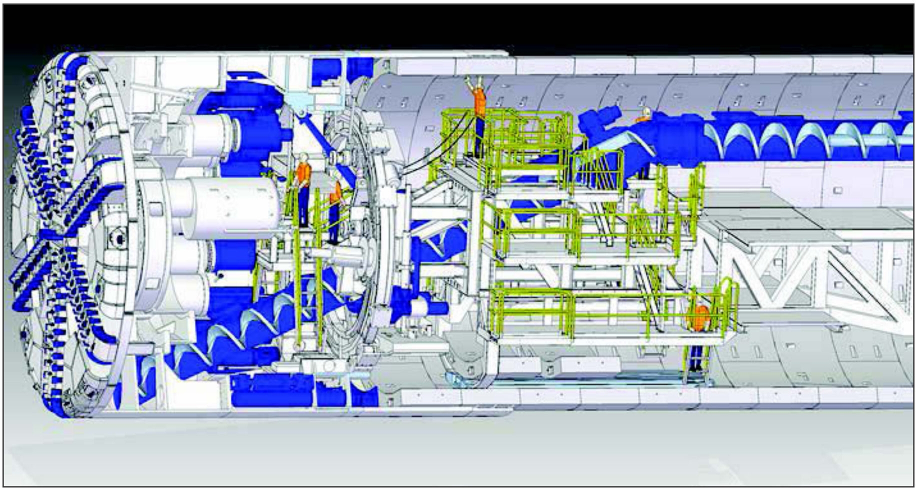

Figure 5. EPB cross section showing two-stage screw conveyor

Continuous Conveyors for Limited Space

Muck from all three machines is deposited from the screw to a fabric belt conveyor mounted on the trailing gear, which transfers to a Robbins side-mounted continuous conveyor. The continuous conveyor carries the muck to a vertical belt conveyor located at the launch shaft. Once at the surface, a radial stacker deposits muck in a kidney-shaped pile for temporary storage.

Due to the narrow shafts and small launch sites, the conveyor systems have been optimized for space efficiency and safety. The belt is surrounded by a guard with recycle hopper to prevent hazardous falling muck while returning the material to the vertical conveyor.

A unique vertical belt cassette allows for splicing of belt with a footprint 170% smaller than a typical horizontal belt cassette. The 34 m tall belt cassette is used to splice in a 450 m length of belt, which takes roughly 12 hours and allows the machine to advance for roughly 200 to 225 m (see Figure 6).

Figure 6. Vertical belt conveyor with vertical belt cassette in the background (tower on the right)

EMISOR ORIENTE PROJECT IN 2012

These are the actual geological conditions of the project in Lots 1 and 3—during the last four years after more boreholes were done, allowing us to present more detailed data. The intention is to review the advance of the project (see Tables 1–7).

The first 1,520 meters of Lot 3 from Shaft 10 to 11 will be used as a case study and example to make an analysis of the particularly challenging mixed ground with highly abrasive conditions.

Table 1. From Shaft 0 to 1A (2.7 km)

	1st Stretch	2nd Stretch
Ground condition	Clays and silts of high compressibility and microfossils. Silts and sandy silts, carbonated.	Clays and silts from medium to high compressibility.
Water pressure	0 bar	0 bar
Advance	Complete 100%	

Table 2. From Shaft 1A to 3A (2.6 km)

	1st Stretch	2nd Stretch
Ground condition	Clays and silts from medium to high compressibility. Silts and sandy silts, hard consistency.	Clays and silts of medium to low compressibility and volcanic ash. Sandy silts, hard consistency.
Water pressure	0 bar	0 bar
Advance	Complete 100%	

Table 3. From Shaft 3A to 4 (1.8 km)

	1st stretch	2nd stretch
Ground condition	Sandy silts, hard consistency.	Silts and sandy silts, hard consistency. Clays and silts of medium to low compressibility and volcanic ash.
Water pressure	0 bar	0 bar
Advance	Complete 100%	

Table 4. From Shaft 4 to 5 (1.8 km)

	1st stretch	2nd stretch
Ground condition	Silts and sandy silts, hard consistency Clays and silts of medium to low compressibility and volcanic ash.	Clays and silts of medium to low compressibility and volcanic ash. Small boulders.
Water pressure	0 bar	0 bar
Advance	Complete 100%	

Table 5. From Shaft 10 to 11 (3.1 km)

	1st stretch	2nd stretch	3rd stretch
Ground condition	Andesite massive deposits with three events in which rock is fractured, the fractures are filled with clay. Water inflow is possible through the fractures.	Lacustrine deposits silt—sand, river sand interbedded clays semi-compacted and black, some water in sand and ash horizons.	Massive andesite interbedded with lacustrine deposits in some areas will be excavated in full face rock mass and occasional black sandy silt and clay in the lower half of the face. Water inflows may occur through contacts between the rock and sandy deposits.
Water pressure	0 bar	0 bar	maximum 1.5 bar
Advance	1516 meters		

Table 6. From Shaft 11 to 12 (2.8 km)

	1st stretch	2nd stretch	3rd stretch
Ground condition	Massive andesite interbedded with lacustrine deposits in some areas. Will be excavated in full face rock mass with occasional black sandy silt and clay in the lower half of the face. Estimated inflows of water bearing fractures occur between the rock and contacts with sandy deposits.	Lake deposits are expected with sandy silt and intercalated basaltic ash in the upper half of the face. Fractured rock mass may also be encountered in the crown. At this junction is expected considerable water.	In this interval will be silt—tight sands with abundant lenses and horizons of fluvial sand and gravel to loose pumice, which could have considerable water inflows.
Water pressure	maximum 1.5 bar	>2.5 bar	maximum 3 bar
Advance	0%		

Table 7. From shaft 12 to 13 (3.2 km)

	1st stretch	2nd stretch
Ground condition	Silt—tight sands with abundant lenses and horizons of fluvial sand and gravel to loose pumice, which could have considerable water inputs.	Collations on the excavation face of silt—sand, clay lacustrine, fluvial sand and some gravel, pumice. River water input variable.
Water pressure	maximum 3 bar	maximum 3 bar
Advance	0%	

CASE STUDY: LOT 3 EXPERIENCE

Shaft 10, is a 16 meter diameter wide and 86 meter deep shaft. Shaft 13 was originally target as the launch shaft, but water inflows and difficult geology made shaft 10 a more attractive launch point. Not only did this decision affect the startup shaft location, but it also affected the original design of the machine, which had to be modified to excavate in higher pressure conditions and more abrasive rock, including basalt, volcanic rock and abrasive ash (see Table 5).

Morelos EPB Machine Design

The 292-332 EPB, named "Morelos" in honor of a revolutionary leader of Mexico's recent history is one of three EPB's supplied by Robbins for this project. Morelos is an 8.93 meter diameter machine designed for mixed ground conditions. The cutterhead design, screw conveyor, and conveyor were designed as detailed earlier.

Morelos was additionally designed to handle curves, with a minimum of 700 meter curve radius. To better handle curves, an active articulation system was included in the design of the EPB. Active articulation engages articulation cylinders between the front and rear shields to steer the machine independently of the thrust cylinders.

Morelos EPB Modifications

Some modifications were made to the machines to accommodate the longer sections of hard, abrasive rock coupled with high water pressures that were discovered during shaft construction. Modifications include:

- A 7 bar man lock with an additional decompression chamber to allow two teams to work at the same time. Also a material lock to be able to handle cutting tools more easily.
- A redesigned Bulkhead to allow the new configuration of the man and material locks and high pressure in the tunnel.
- Hardox plates to reinforce the screw conveyor and hard facing plates added to each turn of the screw conveyor in order to withstand abrasive hard rock.
- An air compression system in order to control the water inflows in the chamber during excavation.

Tunnel Excavation at Shaft 10

The machine was assembled in the launch shaft and commissioned in January 2012, with the bridge and all the back-up gantries at the surface. One month later in February 2012, after advancing 150 meters, the machine and its back-up were completely assembled in the tunnel. In March 2012, the continuous conveyor system was installed and running.

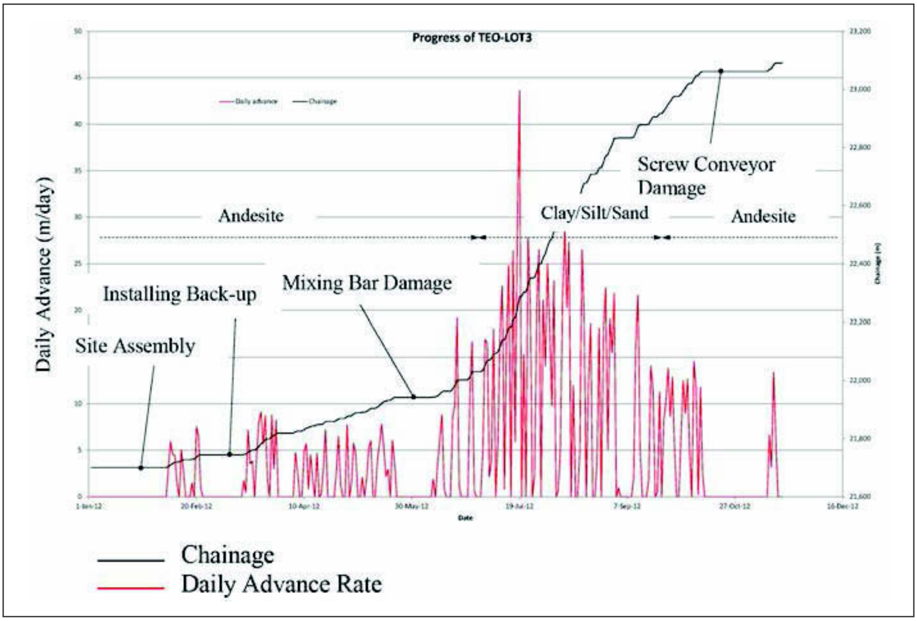

Figure 7. Progress at TEO Lot 3

During the first 400 meters, massive andesite deposits created wear problems in the cutting discs, which required a strict program of interventions in order to stop and check the cutterhead every 7 to 10 rings, depending on the last inspection results.

After this initial 400 meters, the cutting discs were changed out with tungsten carbide cutting tools based on the probe drill results and geologic prediction. Crews encountered silt, clay and sand in the next 1100 meters, and found that this soft ground was also very abrasive, due to the volcanic ash content. This problem, paired with more water than expected and continued interventions, limited production rates.

In the next 4,000 meters, the expected scenario is massive andesite interbedded with lacustrine deposits in some areas, which, during excavation, will consist of a full face rock mass with occasional black sandy silt and clay in the lower half of the face. It is estimated that inflows through water bearing fractures will occur between the rock and contacts with sandy deposits.

In conclusion, production in this lot has been limited by abrasive wear (see Figure 7). Regular maintenance and continued interventions will allow crews to take care of the screw conveyor and cutterhead, though the abrasivity of the rock and ashes is leading to more disc cutter changes.

CONCLUSIONS

The Emisor Oriente Tunnel is a project that is not only logistically complex, but also geologically daunting. The conditions test the limits for EPB tunneling, and have necessarily limited advance rates. The project is not without its successes, however: The Robbins EPB at Lot 1 completed tunneling its 3.5 km section in Autumn of 2012, while achieving good advance rates in silty sands and clays. The lessons learned from this project, once complete, will be invaluable in terms of proper EPB design for extremely abrasive and high pressure conditions.

TBM CONVEYOR BELT SCALES: THE UNIVERSITY LINK PROJECT EXPERIENCE

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ABSTRACT

The introduction of conveyor belt scales on Tunnel Boring Machines on recent high profile projects with the intent of mitigating over excavation, aiding in the control of ground movements, and limiting adverse impacts on surrounding structures has introduced a new set of challenges for clients and contractors.

This paper evaluates the performance of two sets of dual conveyor belt scales used during tunnel excavation on the Central Puget Sound Regional Transit Authority University Link U220 Project in Seattle, WA. The scale data from the tunnel muck excavation is reviewed and then compared with other key TBM parameters such as face pressure and grout volumes demonstrating how well they can indicate normal excavation versus over-excavation. Additionally shown is the differentiation between theoretical volumes and actual volumes and why there could be discrepancies between the two. Based on the findings, recommendations are made for an appropriate approach to use of muck scale data for TBM excavation assessment by contractors, construction management teams and clients.

INTRODUCTION

Tunneling within densely populated urban environments is challenging. In particular, controlling settlement becomes one of the paramount objectives. One of the primary approaches to guarding against over excavation, and therefore settlement, is to monitor the volume of spoil excavated by measuring its weight as excavation proceeds.

Earth pressure balanced tunnel boring machines (EPBM) are being increasingly favored in urban environments due to their ability to carry out controlled tunneling through a wide range of ground conditions. The introduction of conveyor belt scales on recent high profile projects has introduced a new set of challenges regarding interpretation of the belt scale data and their use in determining any necessary modification of the machine operating parameters. The two sets of dual conveyor belt scales used on the U220 project have provided an excellent opportunity to study belt scale performance and recommend their appropriate usage.

PROJECT DESCRIPTION

University Link (U-Link) is the $1.8 billion, 3.15 mile extension of the Central Puget Sound Regional Transit Authority (Sound Transit) light rail system. It will run in twin-bored tunnels from Downtown Seattle north to the University of Washington, with stations at Capitol Hill and on the University of Washington campus near Husky Stadium.

Figure 1. University link project—Contract U220

The U-Link project is divided into multiple contract packages with the two tunneling contracts being Contract U220 and Contract U230.

A joint venture of Traylor Bros Inc. and Frontier-Kemper (TFK) was awarded the U220 contract for construction of the northern tunnels section. The major work on contract U220 includes construction of approximately 11,400-foot long segmentally lined twin-bored tunnels using pressurized face techniques including 16 cross passages excavated at regular intervals between the bored tunnels using sequential excavation methods between University of Washington Station (UWS) and Capitol Hill Station, and civil and structural work for the University of Washington Station crossover. The tunnel alignment passes beneath dense residential and commercial neighborhoods of Seattle and includes the critical crossing of the Montlake Cut, a man-made canal connecting Lake Washington to Lake Union. See Figure 1.

The U220 tunnels were constructed by two 21.5 foot diameter EPBMs supplied by Herrenknecht. The excavated muck was conveyed out of the tunnel by a system of conveyors supplied by Robbins. Dual conveyor belt scales from Siemens Milltronic were used for muck weight measurement.

A joint venture of CH2M HILL Inc. and Jacobs Engineering provides Construction Management Services for the U-Link Project.

GEOLOGY

The Puget Trough in Western Washington, flanked by the Olympic Mountains to the west and the Cascade Range to the east, was created by the convergence of the Juan de Fuca oceanic plate and the North American continental plate. The University Link Project is located within this complex tectonic environment of the Puget Lowland.

The soils encountered in the tunnel route consist of highly overconsolidated clays, silts, sands and gravels, of both glacial and non-glacial origin, in varying proportions.

The Geotechnical Baseline Report (GBR) grouped the geologic units expected to exhibit similar engineering characteristics into Soil Groups in an effort to better represent the wide variability present within each of the geologic units. The GBR indicated presence of “buried valleys” of mixed soil groups and boulders at the interfaces of those valleys.

The tunnel alignment lies below the groundwater table which ranges from 55 ft to 200 ft above the tunnel. The ground cover ranges from 12 ft to 300 ft above the tunnel. See Figure 2.

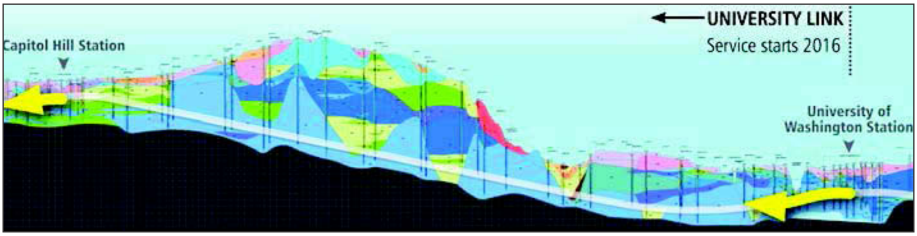

Figure 2. Complex geology along the tunnel alignment—Contract U220

INDUSTRY PERCEPTION

From the combined experience of the authors, it has been noted that the general perception of the owner/CM and the designer is that the belt scales could be calibrated accurately and regularly to record the actual weight of the excavated muck that is passing through the belt conveyor with a high degree of accuracy.

On the other hand, the common belief in the contracting community is that the belt scales on the TBM could never show the actual weight of the excavated muck. Some contractors even consider the belt scales to be of little use and depend on other TBM parameters for excavation control.

From their review of bored tunneling for Singapore Mass Rapid Transit system, Osborne et al. (2004) commented that belt scales can be used as a general guide in the monitoring of the excavated spoil volume and a gross check against overexcavation. They observed that the weighing accuracy may be affected by the unit weight of muck, angle of belt conveyor, thickness and width of belt, belt tension and eccentricity of carrier roller.

Ciamei et al. (2009) noted that frequent calibration of belt scales was a risk mitigation measure during tunneling for Canada Line in Vancouver, Canada as the scales are adversely affected by very wet soils and the curvature of the alignment.

Slinchenko (2009) noted that conventionally two belt scales are installed and the final weight of the excavated material is calculated as an average of the readings from the two scales.

Fonseca et al. (2010) recorded that an accuracy of 3–4% was obtained from two belt scales during tunneling for Metro in Porto, Spain.

Robinson et al. (2012), from their experience in the Beacon Hill Project, Seattle, concluded that in a controlled environment, the belt scale accuracy could be between ±2% to ±5% when averaged over several rings. They recommended use of two belt scales connected to the TBM data acquisition system and frequent comparison of belt scale data with weights of representative muck samples.

U220 CONTRACT REQUIREMENTS

The Contract Specification required twin conveyor scales connected to the tunnel boring machine (TBM) programmable logic controller (PLC) capable of measuring the amount of excavated material during advance for each segmental ring. The scales should provide instantaneous and total weight measurements of excavated soil to the TBM PLC and should be calibrated monthly, as a minimum.

It is worth noting here that the above requirement is in line with the recommendation by Robinson et al. (2012).

The specification did not accept use of non-weighing nuclear density sensors.

The contract document directed the contractor to immediately implement corrective actions if verified muck weight and volume exceeded theoretical excavation weight and volume by more than ten percent per ring.

Figure 3. Belt scale on the TBM conveyor and the Siemens Milltronics belt scale unit

EQUIPMENT SETUP

As mentioned above, the U220 Contract required that the Tunnel Boring Machines utilize dual conveyor belt scales. TFK recognized the importance to the Owner of tracking muck weight on this project. TFK therefore worked very closely with the TBM supplier, Herrenknecht AG, of Germany (HK), and their subsidiary H+E Logistik GmBH, of Germany to select the most appropriate equipment for the project.

During the selection process, several key parameters were examined. First, the geology of the material plays a key role in the selection process because material density can affect accuracy, as we will show. Second, the machine advance rate must be accounted for as this will determine volumetric flow rate of the material. Finally, the installation geometry constrains the size of the conveyor belt scales.

Based on these project requirements, TFK, along with HK and H+E, selected two each, Siemens Milltronic belt scale units, installed into two separate self-supporting steel structures. At the heart of each scale is a load cell. This load cell, Siemens model MSI BB, is what is actually converting the weight of the material as it passes by into a proportional electrical signal. The electrical signal is then wired to the integration unit, Siemens BW100, which totalizes the weight. The load cell is flanked by four additional carrying idlers, two forward, and two in the rear. The installation of the idlers with respect to the load cell is measured in millimeters. It is this precision built construction that enhances the overall accuracy of the system. See Figure 3.

The TBMs used on the U220 Contract would have to be modified to accept the belt conveyors, both the TBM conveyor with the scales, and the advancing tail pulley for the continuous tunnel belt. To accommodate this, the forward scale would be inline, or tangent with the screw discharge point, and the rear scale would have to be located in a 150m radius curve. The TBM belt needed a slight curve to facilitate a discharge point at spring line of the tunnel.

RESULTS

Calculations

Prior to TBM launch, TFK provided a submittal outlining calculations for the muck scales as well as their expected performance criteria. These calculations were based upon GBR information provided to TFK as well as TFK's past experiences with ground conditioning. Figure 4 represents the logical flow of the calculations.

The tunnel excavation is a mass flow rate calculation, wherein the advance rate of the TBM is multiplied by the cutterhead cross-sectional area producing a volumetric flow rate of muck. This volumetric flow rate is then converted to a mass flow rate based on GBR information using the wet unit weight of material. Because the soil needs to

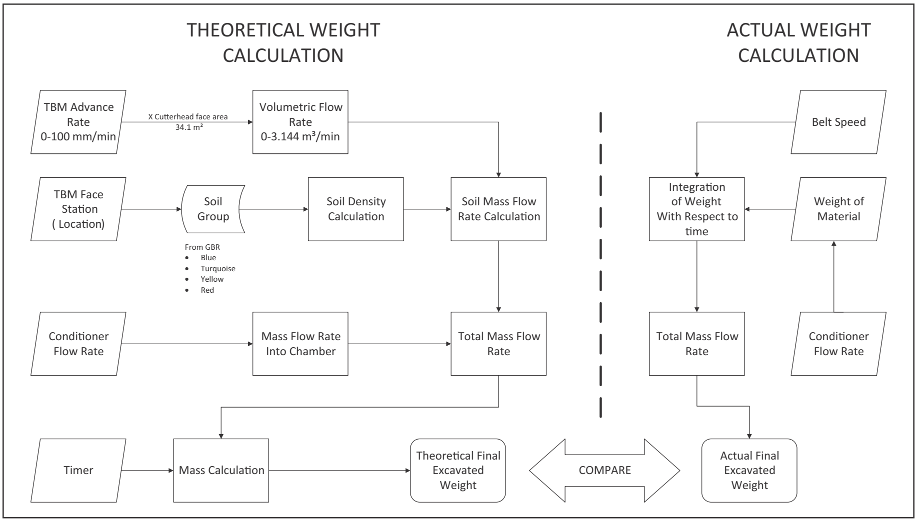

Figure 4. Flow chart of theoretical and actual muck weight calculations

be conditioned to move through the cutter head and screw conveyors, the additional conditioning material needs to be taken into account as part of the weight calculation.

To provide feedback in this process, we add the muck scales to the TBMs. The muck scales are simple load cells plus a timer. When muck passes over the load cell at a known speed, the system measures the weight. Using a bit of calculus, the system then integrates the weight over time using a speed sensor to measure the actual speed at which the belt is traveling. The PLC then compares the calculated volume based on cutterhead advance rate to the output from the Milltronics Scale integration device. The operator then reviews that information for any indication of over or under excavation.

Theoretical vs. Actual

During the initial drives sections from the UWS muck was removed using 18cy muck boxes. Once the TBMs had tunneled far enough (900 ft), the muck removal system switched over to a 26 inch wide continuous conveyor belt. Prior to the switch from boxes to conveyor belt, TFK put into place a rudimentary weighing system that utilized the mucking crane's load cell, a Liebherr HS 895. The load cell data was then compared to the theoretical weight minus the amount of conditioner used. Figure 5 shows a three ring average of muck weight for the first 60 rings of excavation. TFK used a three ring average of the scale data because of the longer than average screw conveyor design as well as accounting for when an operator may empty the screw conveyors.

Using the muck boxes in conjunction with the crane scale, the theoretical weights were in line with actual readings. Once the TBMs were converted from using the muck boxes to the conveyor belts and load cells a new correlation had to be established.

Typically, concerns with muck scales are with weights being over the theoretical values. In the case for the U220 Contract, the opposite was true; the data being read by the Siemens Milltronic system was typically about 10% less than the theoretical values. Additionally, it can be seen that the Rear Scale read 5–10% lower than the Forward Scale. Refer to Figure 6. Although Figure 6 only shows data for 22 rings, it is representative of nearly all data from both TBMs for the duration of the tunnel drives.

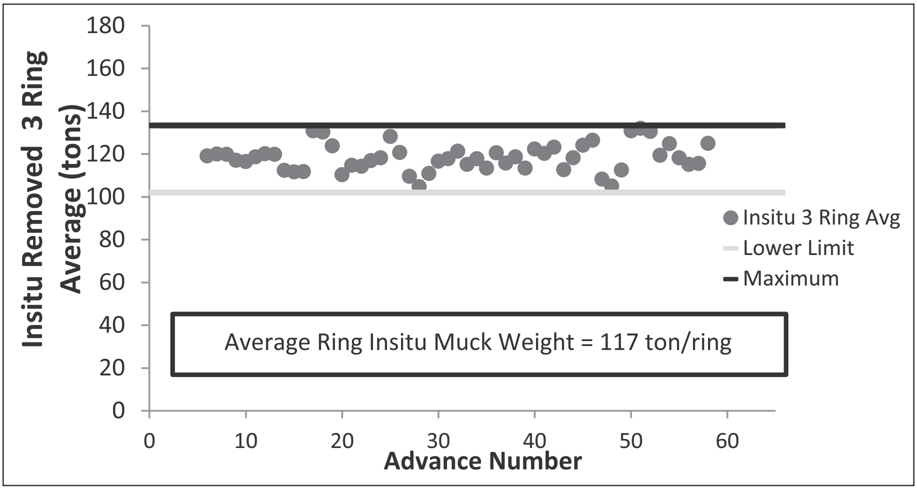

Figure 5. Graph of muck box weights

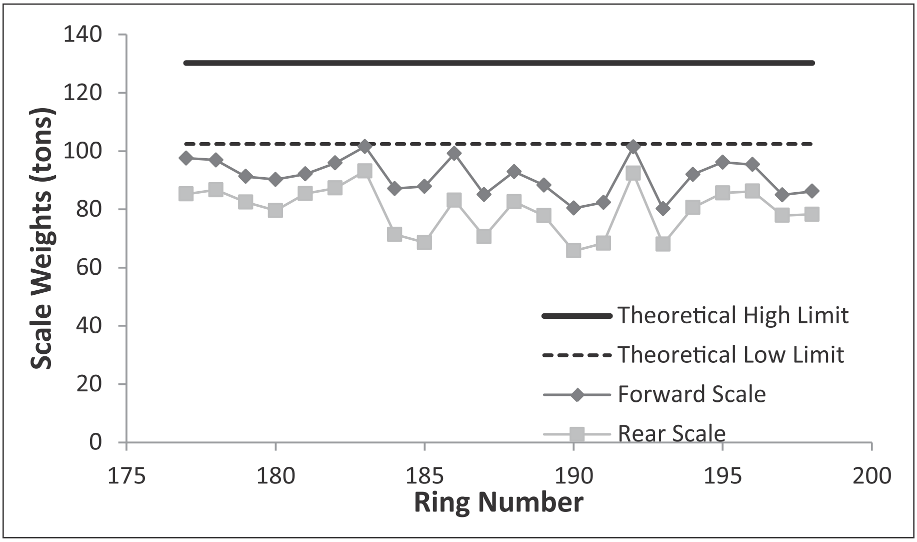

Figure 6. Graph of muck scale data

CHALLENGES AND A PATH FORWARD

The big question was, "Why was there a discrepancy in the data?" Were the scales calibrated correctly? Were they installed properly? Was the system functioning properly? Why were the forward and rear scales reading differently? All of these questions were being asked, along with many more.

TFK anticipated that the belt scales may not perform to the expectations of everyone involved, including the Owner, CM Team, and Design Team because of challenges other contractors have had with belt scales. To alleviate some of concerns, during TBM assembly and conversion to the tunnel belt from muck boxes, TFK had an H+E technical representative on site during the belt commissioning to ensure that the scales were installed properly and that the system was performing as intended.

With a bit of research and some logic, the reasons became clear. As mentioned above, the material which the TBM was advancing through was an over consolidated lacustrine clay. It had a baseline weight of 128 ± 4 pcf. Clay happens to be one of the more difficult materials through which to mine with an EPBM as the clay has a tendency to be very sticky. A common approach to condition the clay is to add vast amounts of water transforming it from a solid to a plastic and then beyond the liquid limit, into a slurry. However, for the U220 Contract, to keep the material on the conveyor belt, the material had to be relatively dry. When watching the material come out of the screw, the clay would oftentimes come out in large blocks as shown in Figure 7. These large blocks could weigh 150 kilograms or more. A bit of research into the Siemens load cell revealed that its accuracy was <1% in a range of 20–100% of loading. The particular model used had a maximum reading of 100 kilograms. This meant that when a block went over the load cell, if it weighed more than 100 kilograms, the weight would be truncated reporting the weight at only 100 kilograms. Conversely, when small bits of material would trickle over the scales, they wouldn't be measured. This explained why the scales were reading under the theoretical amount because the measuring system was clipping the high end of material and unable to account for the small amounts of material.

TFK discussed with Herrenknecht increasing the scale range with a different transducer with a higher capacity. The next size transducer could handle up to 225 kilograms, but again would only have an accuracy of <1% within a loaded range of 20–100% and therefore would still not be able to measure the smaller bits of material. Furthermore, when the TBMs would reach the sand valleys along the alignment and where there would be a more uniform loading, the larger capacity transducer would be reading on the lower end of its loading capacity and consequently wouldn't necessarily improve the accuracy of the scales.

The discrepancy between the forward and rear scale still needed to be address. A quote from the Siemens application manual sheds some light on the discrepancy between the two scales. "Any conveyor that is not a permanent structure or that varies in its incline, elevation or profile is not considered a good installation for an accurate belt scale" (Siemens Milltronics, 2003).

Clearly the TBMs are neither permanent structures, nor constant in incline, elevation or profile, in fact they are the direct opposite. Because each scale was mounted on individual TBM backup gantries, the installations of the scales were not identical. In reality, as the TBM advances the geometry between the two scales is continuously changing due to both horizontal and vertical curves. Because the geometry between the two scales is not identical, the feedback from the scales is dissimilar.

On August 5, 2011, TFK held a joint meeting with Sound Transit, the CM Team, the Design Team, Herrenknecht, and H+E Logistiks to discuss the situation with the belt scales. TFK, along with HK and H+E explained the shortcomings of the scales, as mentioned above. All members in the meeting agreed that the TBM is neither a fixed structure nor the ideal condition for belt scale installations. A path forward would need to be laid out.

Figure 7. Dry clay material on conveyor belt

In order to use the scales effectively for the project, two options were discussed. The first option, based on previous muck data, H+E could place an offset in the scale integrating unit for each of the scales. The offset would account for the differences in installation for both scales as well as the consistency of the material. The second option was also to use previous muck data, but establish new criteria by which to assess future muck weights. All parties agreed that the raw data should not be altered, and the second option was the best path forward.

TFK along with the CM team worked to determine the new operational limits for excavation. Figures 8 and 9 show the new limits that were used for each of the scales on the first TBM. The approach was to use the previous 30 ring muck information and use a statistical average of each scale plus or minus 10%. Both teams verified that other TBM telemetry data such as face pressures and grout volumes were in line with proper TBM tunneling execution. A similar approach was used for the Second TBM with slightly different results. Additionally, it was agreed that a continual evaluation of the limits would take place to account for change in ground conditions.

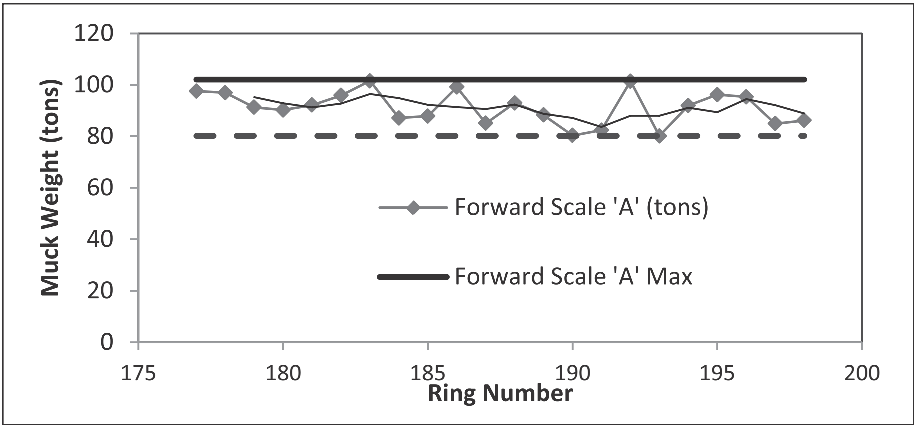

Figure 8. S-560 Forward scale "A" operational limits

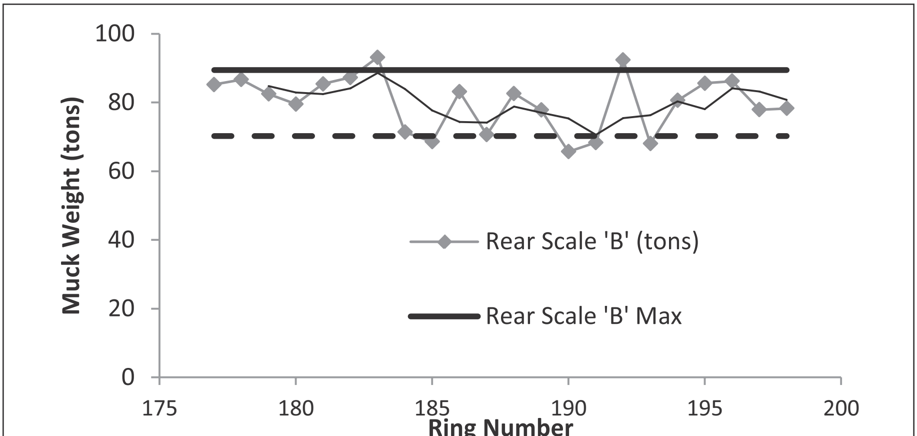

Figure 9. S-560 Rear scale "B" operational limits

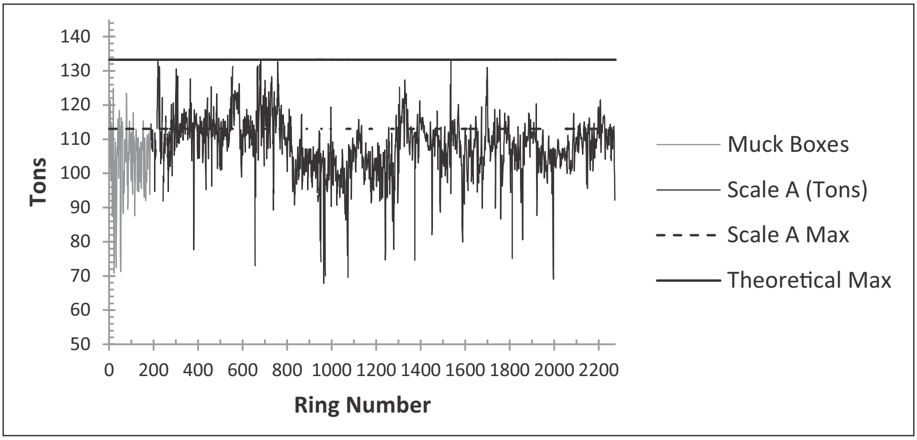

Figure 10. Muck weights from the S-561 TBM drive

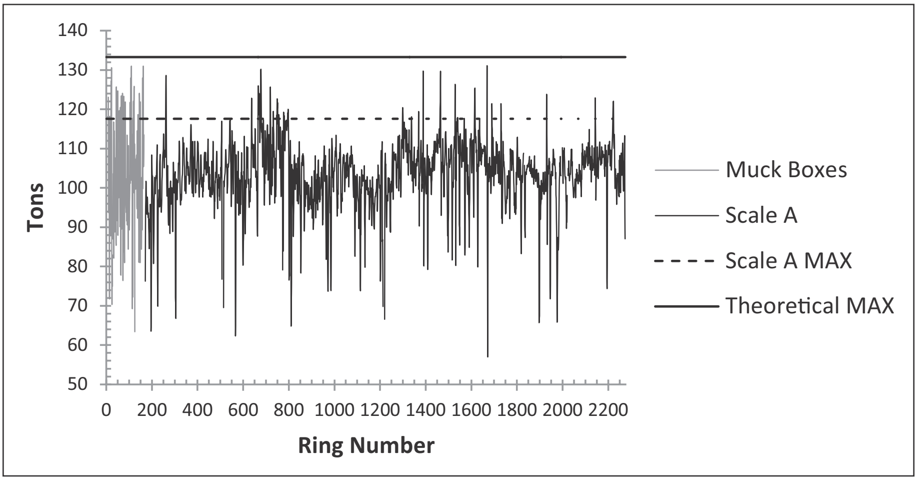

Figure 11. Muck weights from the S-560 TBM drive

TELEMETRY DATA REVIEW

For risk management of the EPBM operation, a straightforward observation-reaction approach was adopted by TFK and the CM team. Face pressure, weight of the excavated muck and tail void grout, the leading indicators of potential overexcavation, were closely monitored during tunneling. For each indicator, acceptable parameters were established. Explanations were obtained and, if necessary, corrective actions were immediately implemented in case of any deviations from those parameters. Due to the smooth working relationship, both sides provided near immediate feedback to each other, providing adequate redundancy in the approach. The muck scale data from both tunnel drives is presented in Figure 10 and Figure 11.

Review of the information shows the following:

- The muck weights for the tunnel drives have stayed within the theoretical maximum value for controlled excavation.

- The weights recorded by Scales "A" and "B" are generally not close enough that they could be averaged.
- It is difficult to establish whether Belt Scale "A" or "B" is reading true.
- The high muck weights around Ring 200 occurred during the scale commissioning with H+E. Face pressures and the tail void grout takes for the corresponding location were found to be within their limits.
- All instances of high muck weight were investigated in conjunction with the face pressure readings and tail void grout intakes. Discussions were carried out between CM team, Designer and TFK to find suitable explanations. Cleaning of screw conveyor after blockages and partial emptying of the excavation chamber for tools check or during inspection stops was the most common reason for high muck weights.
- If a reasonable explanation was not there, TFK implemented follow up actions involving inspection and calibration of face pressure sensors and muck scales, proof drilling and secondary grouting operations.

CONCLUSION/RECOMMENDATION

- The decision to reevaluate the operational limits for muck excavation based on statistical averages and continued evaluation of the limits to account for changes in ground condition established a process through which the project team could ensure that all potential areas of overexcavation were adequately addressed.
- Belt scale sensors can be accurate under controlled conditions which does not include TBM belt conveyors. For TBM tunneling, the variations in belt scale readings averaged over three consecutive rings could be used as a good indicator of potential overexcavation.
- The approach described in this paper has been found to be highly effective for the U220 Contract ground conditions, which was primarily silty clay. Additional data collection and application of this method on other types of ground condition need to be carried out to confirm broader applicability of the method.
- Implementation of the discussed approach, which helped in successful control of the EPBM excavation, has been possible due to the effective working relationship and cooperation between TFK, CM team, ST and the designer.

Based on our observations in the U220 project, we would like to make the following recommendations:

- Use two belt scales linked to the TBM programmable logic controller
- Review the data from the scales in conjunction with other TBM operational parameters such as face pressure and tail void grout volumes and pressures
- Realize that the scales are a useful guide to indicate potential areas of overexcavation and will not provide precise measurement of the actual weight of the excavated muck
- Calibrate the scales as soon and as often as unusual trends in the readings are observed
- Evaluate the need to establish new criteria for better assessment of scale data if there is conclusive evidence of a continuing trend of incorrect scale data
- Do not average the data from two belt scales as they will have differences due to dissimilarities in their installation and the geometry between them.

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PART 17

Pressure Face TBM Case Histories—II

Chairs

Tim Backers
Jay Dee Contractors, Inc.

Mike Krulc
Traylor Frontier-Kemper JV

SLURRY TBM TUNNELING ON THE EAST SIDE CSO TUNNEL PROJECT, PORTLAND, OREGON

Martin Holfelder ▪ Bilfinger Berger Civil Engineering

Niels Kofoed ▪ Kiewit Infrastructure Group, Inc.

Gary Irwin ▪ City of Portland, Bureaus of Environmental Services

ABSTRACT

Challenging ground conditions were encountered during the 29,200-lf Slurry TBM tunnel drives for the Portland East Side Combined Sewer Overflow Tunnel Project in Portland, Oregon. A unique solution for tunneling through four intermediate tunnel shafts was successfully implemented. Compressed air interventions at pressures up to 3.3 bar were used to access the TBM cutterhead during the tunnel drives in order to change cutting tools and maintain the cutterhead. A variance was obtained from OR-OSHA to adopt the German Compressed Air Intervention Regulation on the job. Our tunneling crews self-performed the challenging intervention work.

The TBM tunnel excavation was divided into a North and a South drive; this paper discusses the setup for the TBM launch on each of the two drives, as well as the transition from the North to the South drive.

INTRODUCTION

The Portland East Side CSO Tunnel Project (ESCSO) is part of the Willamette River CSO Program in Portland, Oregon. The Project Owner is the City of Portland–Bureau of Environmental Services and the Contractor is Kiewit–Bilfinger Berger, AJV (KBB). Construction started in March 2006 and was finalized, on time, in December 2011.

The scope of work for the ESCSO tunnel project included a 29,200-lf (8,900-m) tunnel, 100 to 150-feet below grade and 22-ft (6.71-m) finished inner diameter; seven main shafts; about 7,600-lf (2,316-m) of micro-tunneled pipeline; 2,600-lf (792-m) of open cut pipeline; and thirteen diversion structures. The alignment of the ESCSO project is illustrated in Figure 1.

The Slurry TBM for the ESCSO project was fabricated by Herrenknecht. The first of two tunnel drives—the 20,340-lf (6,200-m) North Tunnel drive—was launched from the Opera shaft and holed through at the Port Center shaft. The TBM was then transported back to the Opera shaft, refurbished and re-launched for the second of two drives—the 8,860-lf (2,700-m) South Tunnel drive. The South Tunnel drive holed-through at the McLoughlin shaft. The following sections will provide a detailed discussion of the TBM operations.

The issuance of a Professional Service Agreement (PSA) prior to the unique Cost plus Fixed Fee Construction Contract allowed the Owner, the Designer, Parsons Brinkerhoff, and the Contractor adequate time to finalize design documents and plan field operations. The PSA phase of the project created a strong working partnership between the Owner, Designer and Contractor which lasted throughout the job as previously reported by Metzger et al. 2009.

Geology

The Geology encountered along the tunnel alignment proved to be a challenge for the project team. A shallow groundwater table fluctuating with the tides of the Willamette River delivered hydrostatic pressures up to 3½ bar at tunnel elevation. Three unique and very different soil strata made up the majority of the material being excavated, with the primary stratum being the Troutdale Formation covering approximately 85% of the tunnel alignment. The Troutdale Formation, as illustrated in Figure 2, is a heterogeneous geologic unit comprising gravel and cobbles in a very dense and occasionally cemented sand/silt matrix with lenses of sand and inter-bedded gravel and boulders up to 48 inches in size. Approximately 10% of the tunnel alignment was excavated in the Sand/Silt Alluvium. This formation consists predominately of inter-bedded sandy silt and fine sand with an often loose or soft consistency and high content of fines which proved challenging for the slurry separation plant. The remaining 5% of the tunnel alignment was excavated within the very abrasive Gravel Alluvium. The Gravel Alluvium is a coarse gravel deposit containing up to 48-inch boulders. The broken boulders of this formation caused extensive wear on the TBM cutting tools, slurry discharge system of pipes and pumps used to transport the material from the heading of the slurry TBM to the separation plant.

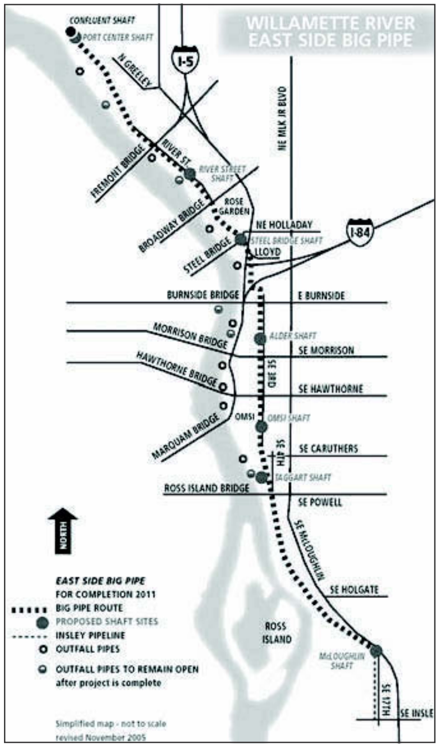

Figure 1. Alignment of the Portland ESCO Project—project map

Site Layout

The scope of the PSA Phase of the Project included the design and procurement of tunnel related key equipment as well as other contract requirements. A tower crane was analyzed to be the best solution for supporting the tunnel logistics. The tunnel site layout is shown in Figure 3.

Muck Disposal

A total of 750,000 Cy of tunnel spoils were removed from the Opera Tunnel site over the life of the job. The barge load-out facility included a 420 ft long

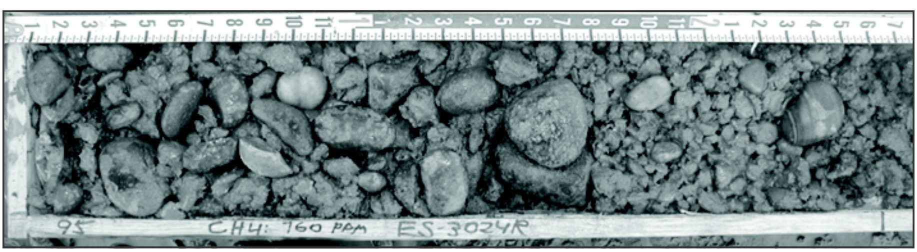

Figure 2. Troutdale Formation

conveyor system extending 250 ft out into the Willamette River. Figure 4 displays the installation of the conveyor, supported by a cable stayed structure centering around the 80-foot tall center mast. To facilitate the barge loading, a winching platform supported by driven steel piles was also installed in the river as shown in Figure 5. The capacity of the conveyor was 750 tph and a typical barge would transport 800–900 tons, which could be loaded in about two hours. The conveyor was outfitted with a chute to direct tunnel muck onto barges and avoid spillage into the river. The barges traveled half-a-mile up-river to the Ross Island Lagoon where the muck was used to support an ongoing reclamation effort for the island.

Slurry Separation Plan and Circuit

Based on the water pressure of 3.5 bar in permeable soft soil, a slurry TBM was required for tunnel excavation. In the design phase of the contract, our team designed, and procured, a Separation Plant and TBM system with a capacity of 1400m^3/hr (6,140 gpm).

Figure 3. Main construction site—Opera shaft

Figure 4. Installation of conveyor for tunnel muck loadout

Figure 5. River barge for tunnel muck loadout

Table 1. Grain size analysis for slurry plant design

Material	Particle D	Separation Equipment
Clay	<2 Micron	Centrifuge
Silt	<60 Micron	Centrifuge/Cyclons
Fine Sand	<200 Micron	Cyclons/Sieve Shakers
Coarse Sand/Gravel	>200 Micron	Sieve Shakers

The mucking material was pumped from the heading to the separation plant by 4 each 723KW Booster Pump Stations located throughout the tunnels. An analysis was performed, based on the particle size of each formation, to determine the appropriate separation system to be used—as illustrated in Table 1.

Muck pumped to the separating plant was pre-screened with material of particle sizes exceeding 2–4mm taken directly out to the disposal area. The remaining muck/fluid was pumped to the second step, which exited out of two cyclone stages where material exceeding 60 micron was separated and particles less than 60 microns sent for further treatment.

Based on the fine content with particle sizes less than 60 micron, the bentonite slurry increased in density and was to be refreshed periodically. To minimize expensive bentonite slurry disposal to the permitted disposal site, a Hiller Centrifuge DP 84 was used to separate fines and water. Figure 6 shows the separation plant used for the project.

Prior to tunnel excavation, different bentonite types were tested with the Troutdale formation and the Alluvium soils and the required shear strength engineered based on the valued provided by the GBR. Figure 7 displays the slurry exchange criteria defined for the tunnel operation.

Lab tests of the Bentonite slurry were performed by the separation plant operators for each tunnel ring excavated. The separation plant provided flexibility to use different bentonite slurry mix designs with two mixing stations with a 50m^3/hr (220 gpm) capacity each. This capacity allowed the plant to support the Microtunnel operation as well as the main tunnel. During compressed air interventions on the TBM Permeability and air loss was minimized by pumping a heavier bentonite to the heading before the intervention took place.

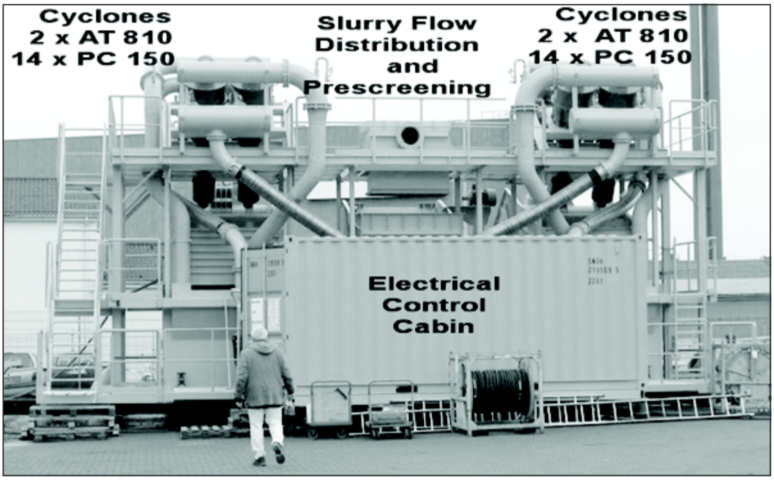

Figure 6. Slurry separation plant MAB 1400

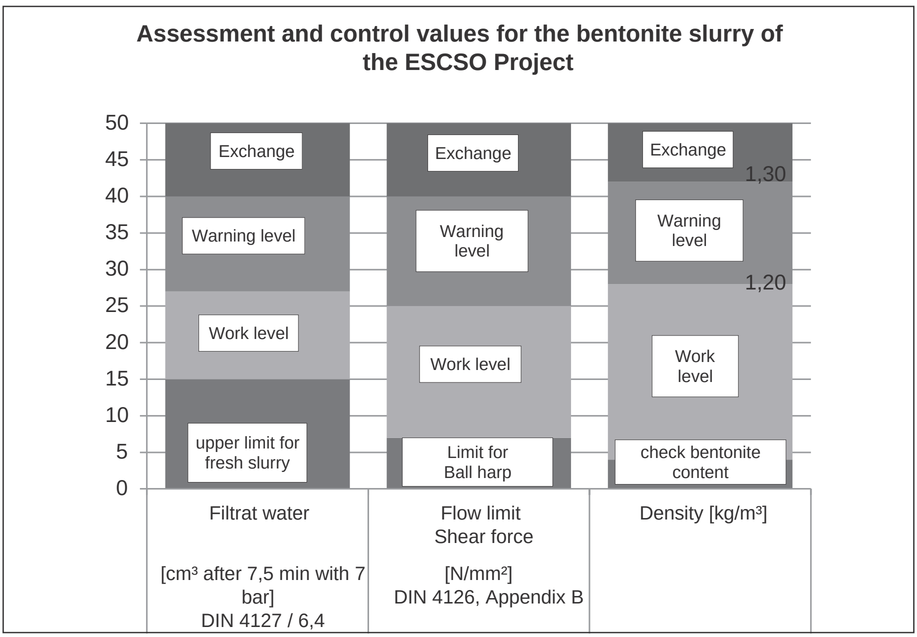

Figure 7. Bentonite slurry assessment

Slurry TBM—General Requirements

The technical specification for the Slurry TBM is outlined in Table 2. Special consideration was given to the heavy-duty construction of the stone crusher and hardface-welding to minimize the wear of the structure. Primary TBM components were designed to withstand up to five bar of pressure.

All electrical systems were required to be compatible and suitable for operations with primary power that was supplied at 13.2kV.

Table 2. Technical specification TBM

Type:	Mixshield TBM (Slurry)
Diameter:	25-feet, 3-inches
Tail-Can seal:	3 rows of brushes
Grout Lines:	4 each + 4 redundant
Cutterhead Rotation:	Bi-directional
Cutting Tools:	Single and double disc cutters (17"), Scrapers
Cutterhead Drive with Electrical Motors:	8 each
Double Chamber Man Lock:	2 each
Persons in Man locks:	4 each primary, 2 each secondary
Erector Gripping System:	Vacuum
Thrust Cylinders Quantity:	30 each
Thrust Force:	10.7 million pounds
Maximum Stroke:	7.5 feet
Speed:	2.3 inches per minute
Stone Crusher Type:	Hydraulic Jaw Type
Maximum Boulder Size for crusher:	800 mm
Grouting Equipment Number of Pumps:	2 each double piston
Slurry Circuit Discharge/Feed Line Size:	14-inch / 16-inch diameter

TBM systems were designed to operate in contaminated and hazardous conditions. It was critical that the TBM and all associated systems complied with applicable provisions of 29 CFR 1926.800 for use in hazardous locations—Class I, Division 2.

TBM TECHNICAL FEATURES

The bulkhead was equipped with four bentonite supply nozzles, which were capable of being cleaned from both the "free" (atmospheric) air and compressed air sides. Two additional nozzles were installed to optimize the condition of the excavated soils.

The front of the TBM shield was divided into two chambers that were separated by a submerged wall (bulkhead). The front/excavation, chamber contained the cutting wheel and was filled with a mixture of slurry and excavated material that provided pressure balance to the excavated face. The slurry was pressurized by an air bubble in the second chamber. The accesses between the two chambers were provided through pressure-tight doors in the bulkhead and the double chamber man-locks.

Grouting Annulus

The annulus grout mixture consisted of fill sand, slag cement and bentonite. During the shield advance, the annular space was continuously filled with grout through injection pipes integrated into the tailskin of the TBM shield.

SAFETY FEATURES

Fire Detection and Suppression

The TBM was equipped to fight both electrical and liquid fires. Handheld fire extinguishers were provided, including powder and CO_2 unites located throughout the TBM. The CO_2 fire extinguishers were installed near electrical switchboards, generators, and other electrical facilities. The ABC Dry Powder Extinguishers were installed at the control cabin, air lock, and entry/exit of each back-up trailer.

A water curtain was installed at the end of the TBM trailing gear to limit the potential dispersion of toxic smoke during a tunnel fire. An automatic fire alarm system was installed in the shield area and on the TBM back-up.

Gas Detection

The gas detection system was capable of monitoring Carbon Monoxide, Methane, Oxygen, Nitrogen and Hydrogen Sulphide as per specification.

SEGMENT DESIGN

The basic segment design parameters are displayed in Table 3.

Steel Fiber v. Reinforcing Steel Technical Analysis

A cost evaluation was performed for the use of steel fibers in lieu of reinforcing steel for the precast segmental lining. This cost evaluation was prepared as a result of a Technical Review Committee Analysis conducted in October 2005.

Based on the analysis and the recommendation of the Technical Review Committee, steel fibers were used for 85 percent of the tunnel alignment in the Troutdale formation. Standard reinforced rebar segments were used in the remaining 15 percent of the tunnel alignment—in Sand/Silt Alluvium and Gravel Alluvium. This was also used during the break-in and break-out from the shafts. The breakdown of the two types of segmental liner is provided in Table 4. Figure 8 shows the excellent distribution of the steel fiber in the fiber reinforced segments.

KBB self-performed the casting of the tunnel segments. The original segment plant was designed with a stationary production by using eight sets of molds. Due to high mining performance rates achieved shortly after launch of the TBM, the segment production was increased through the procurement two additional sets of mold and implementation of a steam curing system**.** The steam curing allowed stripping of the segments after four hours and thereby provided sufficient time to include a second shift

Table 3. Segment design parameters

Segment ring outside diameter	24.33 feet
Segment inside diameter	22 feet
Geometry	Trapezodial
Tapering	2-inches per ring
Segment thickness	14-inches
Segment curvature design	600-feet
Ring Length	5-feet
Ring Configuration	7 piece plus keystone
Handling System	Vacuum

Table 4. Design characteristics of the two segmental linings

Description	Steel Fiber Segments	Rebar Segments
Ring Configuration	7 pieces plus key	6 pieces plus key
Ring Thickness	14-inches	12-inches
Rings with Steel Fibers (Troutdale Formation)	5014	0
Rings with Rebar	910 each	5,924 each
Steel Fiber Content	50 pounds per Cy	

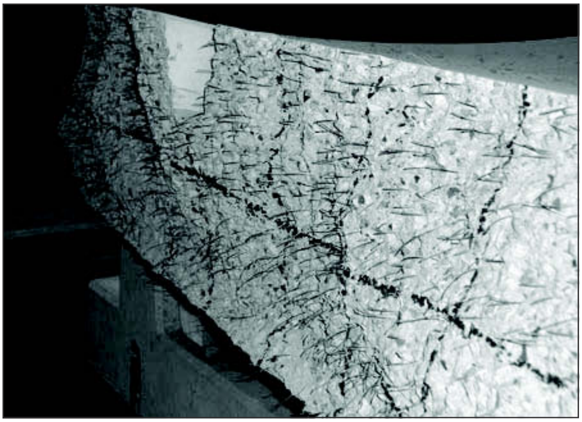

Figure 8. Fiber distribution out of the production of the ESCSO Project

of segment production. These adjustments increased the capacity of the plant from 8 rings per day to 20 rings per day.

OPERATION SPECIFICS

The ESCSO tunnel is the longest slurry driven tunnel in the nation, at 29,200 feet. A number of unforeseen incidents occurred during this long tunnel drive, which called for immediate action. Some of these issues included: the launching under high pressure, a gear box failure, compressed air interventions (including hot work), an extensive TBM overhaul program, and a stone crusher repair.

Launching at Opera

Slurry TBM requires a sealing system to enter the ground water area in front of the shaft. The ESCSO used three rows of independent seals to seal the TBM against the can. Once the TBM left the can, a bullflex hose is designed to seal the gap between the can and liner as illustrated in Figure 9.

A round bullflex hose was used for the initial launch, which provided less friction than a rectangular hose. The hose was filled with foam at the North Drive. The foam began to shrink over time, which caused a leakage. To prevent these leaking issues, the can for the South Drive was redesigned.

The Bullflex hose was enlarged from diameter 320mm to 500mm, becoming more of a rectangular shape to provide increased friction. In addition, a concrete ring was poured behind the Bullflex hose that was supported by Nelson studs.

The new design displayed in Figure 10 allowed a dry launch under a high water pressure of 3.5 bar.

Stone Crusher Repair

The stone crusher, Figure 11, is a central part of the slurry TBM. In July 2009 it was recognized that the stone crusher was not operating correctly. Additionally, three picks were found at the magnet of the separation plant. An intervention was quickly scheduled to replace tools and inspect the crusher. Major stone crusher damage was determined during this intervention. In order to perform the repair under atmospherical conditions, the dive gate needed to be closed. Cleaning under pressurized conditions was necessary in order to close the dive gate. The left arm was sheared off at the back

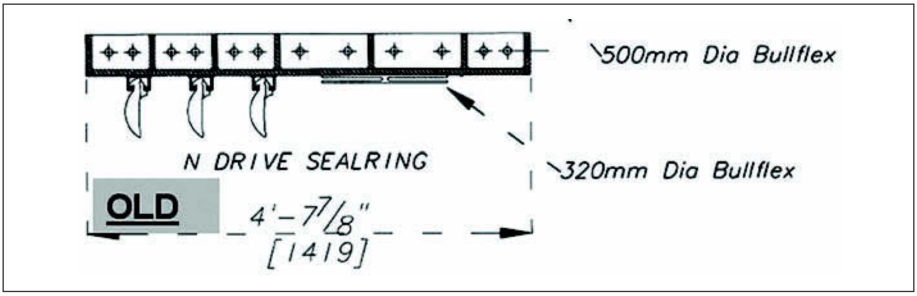

Figure 9. Seal can design for the North Launch

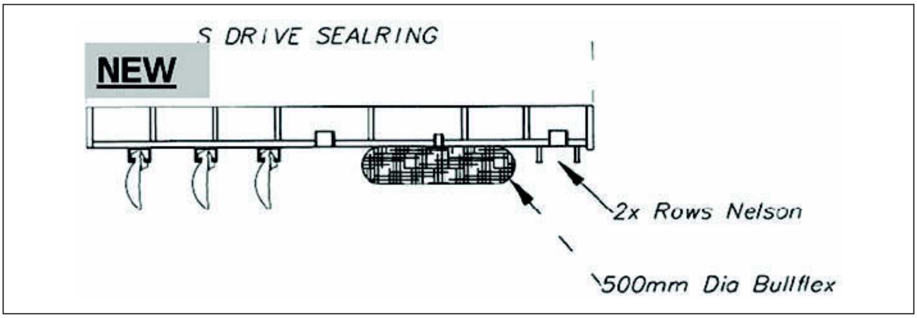

Figure 10. The seal can design for the South Launch

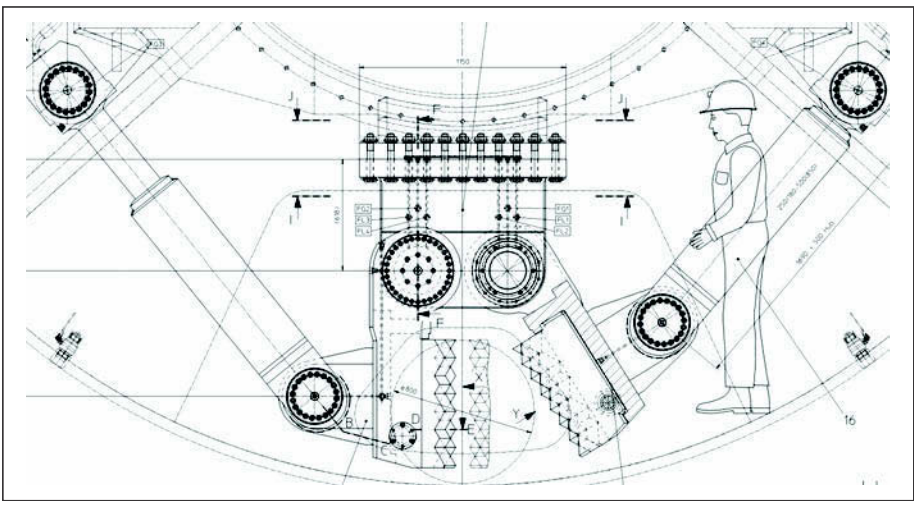

Figure 11. ESCSO stone crusher

pin connection to the yoke. An intensive repair plan was developed and all parts, could be removed through the man-lock.

Prior to the arm disassembly, all non-essential slurry and grease lines were to be capped and removed, pad eyes installed, and welding procedures were performed based on the proximity to the main bearing. Even the access ladders to the crusher area had to be removed to gain required space for the movements. Before removing the

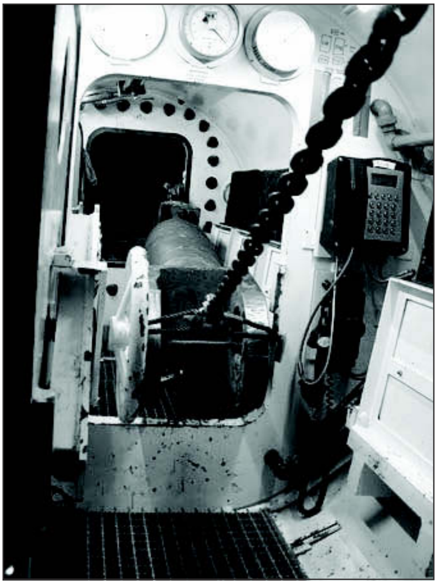

Figure 12. Moving the crusher cylinder through the manlock

Figure 13. Moving the yoke through the manlock

yoke, six welded gussets at the bulkhead to the yoke were removed. Figures 12, 13 and 14 illustrate the removal and repair of the stone crusher.

The disassembly allowed torching into smaller pieces, which made the installation of the replacement parts even more challenging due to the dimensions. Within three weeks, our tunneling crews successfully completed this operation under atmospherical conditions.

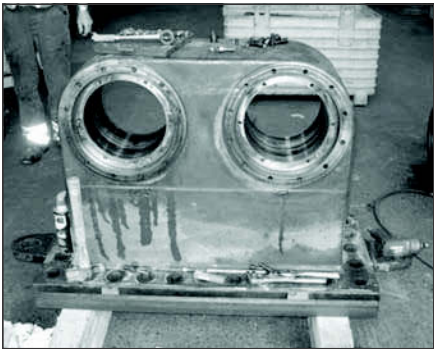

Figure 14. Refurbished yoke on site

INTERVENTIONS

Interventions under compressed air are a common occurrence during slurry TBM projects. Interventions often require the completion of complex tasks under difficult work conditions. This section will discuss some of the challenges faced during this project related to compressed air interventions.

To inspect and replace cutting tools, the level of bentonite within the excavation chamber must first be lowered and replaced with compressed air. A film of bentonite (filter cake) remains on the tunnel face to help seal the exposed ground and reduce loss of compressed air. The permanently pressurized areas are the excavation and working chamber of the Front Shield.

Both scheduled and unscheduled interventions of the cutter head and stone crusher occurred on the ESCSO project. There were many different reasons for this; however, it was primarily due to the wear of cutting tools and the need to inspect and replace tools, scrapers and stone crusher parts. Our evaluation of the ground conditions, design of the TBM, and personal experience helped provide the input into the frequency, duration and methods of interventions expected on this project. The master plan for interventions included:

- Major interventions by utilization of intermediate shafts to perform maintenance, cutter head inspection and tool changing under atmospheric conditions
- Minor interventions between shaft under compressed air with a pressure up to 3.3 bar

Major Interventions/Intermediate Shaft

As we gained experience and operations progressed, the concept for driving into intermediate shafts was modified from shaft to shaft. Each solution was effective with some common approaches. Prior to the TBM arrival, each shaft was backfilled with soil material a few meters above the TBM cross section and flooded 1.5 meters above groundwater level. The backfill material was placed to prevent any flow of the annual grout into the shaft area. The positive water pressure was used to induce a slight water flow away from the shaft once the connection was made and prevent cement from the annuals grout from being washed out into the shaft.

All drives, in and out of the shafts, were performed by adding additional cement to the annular grout, which included a mixture of 177kg/m^3 of slag cement. In the same amount, cement was added within the area of the tunnel eyes.

=> 177 kg/m^3 of slag + 177 kg/m^3 of Portland cement

This led to a solid concrete plug in the panel area, which sealed all shaft walls efficiently—even with an existing water pressure of up to 3.5 bar.

After successfully sealing of the shaft, the dewatering process and maintenance program was performed under atmospheric condition. The procedure will be described by using the Alder shaft as an example in Figure 15.

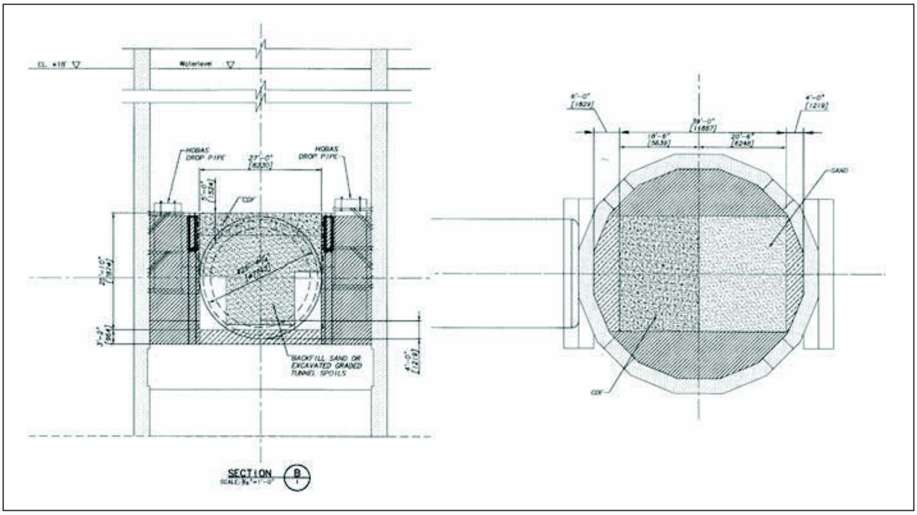

Figure 15. Layout of Alder shaft

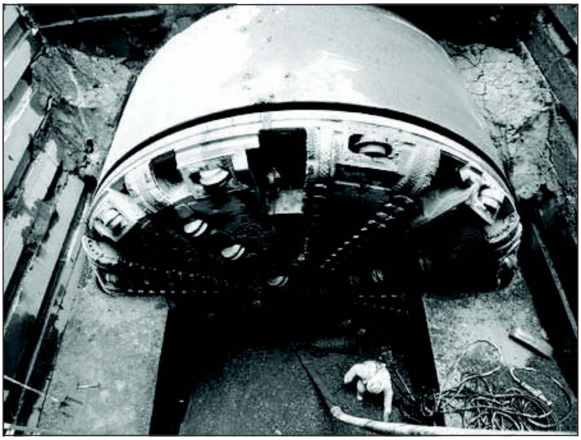

Figure 16. TBM arrival at Alder shaft

Solution:

- Sand fill from the invert up to the crown of the cross section (see Figure 15)
- Controlled Density Fill (CDF) in the upper area of the cross section, from drive in portal to middle of the shaft. The other half was just sand fill up to the top.
- Water, flooded 1.5m higher than the groundwater level

Once the TBM drove into the shaft, the CDF prevented uncontrolled excavation of material. After passing the middle of the shaft, the shaft backfill was excavated entirely, with no CDF on top in this section. This allowed a maintenance program of the cutter-head once the TBM was in position and the shaft sealed as shown in Figure 16.

Excavation of material in front of the TBM was allowed in areas without CDF. It took us about 12 days from the time the TBM reached the shaft wall until maintenance could begin—this included time for dewatering, shaft excavation and cleaning.

Minor Interventions/Compressed Air Work

Variance

It was necessary to obtain a Permanent Variance for OR-OSHA standards, prior to the start of tunneling to perform compressed air operations. This variance covered following standards:

- Oregon Administrative Rules, Chapter 437
- 1926.803 Compressed Air
- Section (f) Decompression
- Section (g) (iii) man lock equipped with automatic controls
- Section (g) (1) (xvii) Special decompression chamber needed
- Decompression exceeding 75 minutes
- Section (g)(2) Special Decompression Chambert
- Subpart (S) Appendix A—Decompression Tables

Under Variance number V-001-06, KBB was allowed to replace the OR-OSHA Decompression Table (Appendix A) with the German Compressed Air Regulations.

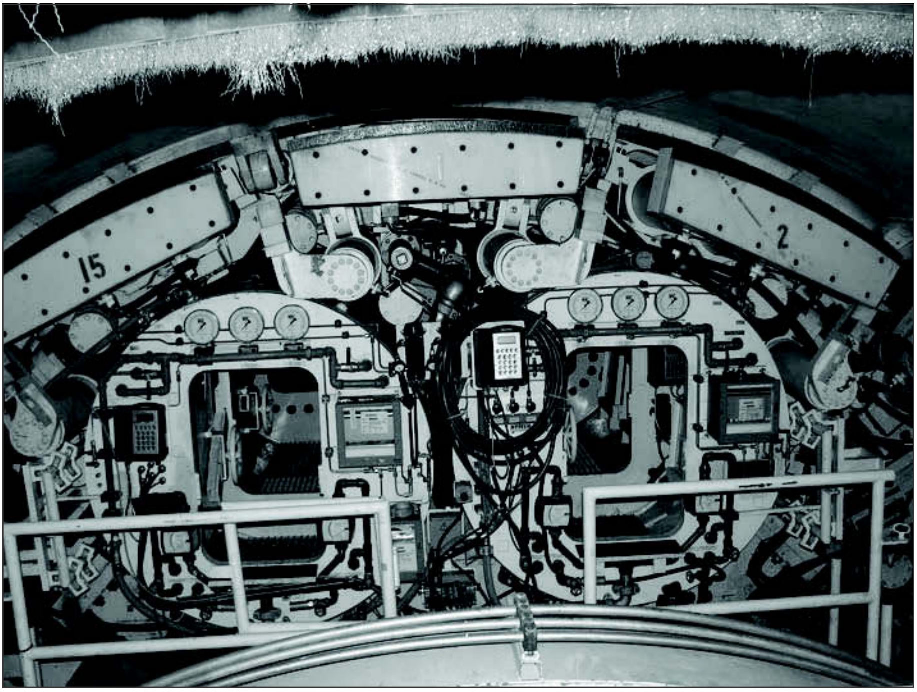

Figure 17. Double man locks of the ESCSO TBM

This allowed the use of gas, especially Oxygen, for decompression purposes. In addition to the German RAB 25, Regulations on Occupational Health and Safety on Construction Sites and Work in Compressed Air provided a higher safety level for this special application.

The German Decompression Tables also include some Emergency Decompression Tables such as “Failure of the Oxygen System” or “Exceeding the allowable working time." The European Standard—CEN 12110—Tunneling Machines Air Locks Safety requirements and the—German BGI 690—Leaflet for the treatment of illness in compressed air (Diving and compressed air work), provided additional up to date standards for the sensitive intervention work. While the OR-OSHA requires automatic controls for decompression works exceeding 0.8 bar our variance allowed us to use manual decompression as the primary operating standard.

Basics for Compressed Air Work

Through the use of oxygen during decompression, we were able to balance the maximum allowable working time of approximately 2 hrs (under a working pressure of 3 bar) with the decompression time. Equipping the TBM with two independent air locks, see Figure 17, provided the highest efficiency for compressed air work as it was possible to rotate crews and eliminate interruptions of work at the tunnel face.

Only medically checked employees were allowed to enter the compressed air area. The compression and decompression operations were performed by an air lock attendant. A medical team did pre-examinations of the divers, supported the operation during intervention time and checked the employees out after finalization of their dive. A physician was on standby by phone.

Table 5. Intervention summary

Total teams sent in compressed air	261
Total of dives	712
Average number of teams per intervention	3.4

Table 6. Analysis of intervention reasons

Inspection purpose	20
Tool changes	28
Dive gate works (Stone crusher)	9
Muck out / cleaning of chamber	5
Welding/Hot works	8
Other	6

Compression and Decompression Procedure

Under regular conditions, three employees were sent to the face as the intervention crew. Two employees had to perform the work and the third one assisted and established communication to the lock attendant.

Upon completion of the work, the employees had to decompress in the air lock before transferring out of the pressurized area.

A case study: The Most Critical and Risky Type of Intervention Work Performed on the Project

The most critical and risky type of intervention work is welding work—“Hot work." Under compression the size and function of a lung is limited and to breath welding fumes will result in the death of a person. Therefore welding work should be performed only, if risk and benefits have been analyzed carefully and no other alternative is given.

Burning and welding under compressed air is different to than the same work done under atmospheric pressure. Acetylene cannot be used. For the given pressure hydrogen has to be used.

Due to the exhaust gases during welding and burning (toxic gases), a special procedure had to be implemented for ventilation and/or the supply of breathing gas. Additionally the risk of fire is higher under compressed air than under atmospheric conditions. During the performance of welding work, one spark would be sufficient to inflame synthetic clothes. Workers were required to wear flame resistant protective suits and throughout the welding a worker with a water hose was standing by at all times.

When extensive wear of the cutter head rim bars were recognized at an inspection intervention during the longest drive section, KBB decided to perform welding work due to the structure of the cutter head

In preparation for this work, a thorough training for certified welders was performed. Special masks and other tools were provided and an extensive Job Hazard Analysis and Work Plan were prepared. The challenging welding work was smoothly performed, continuously, for two days.

Intervention Summary

For the successful completion of the North and South drive of the ESCSO project, a total of 76 interventions were performed with breakdown of the interventions, as provided in Tables 5 and 6.

TBM TRANSPORTATION FROM PORT CENTER SHAFT TO OPERA SHAFT

Upon completion, the slurry TBM was to be returned from the Port Center Way shaft (Swan Island) to the Opera shaft at the main mining site for the South Tunnel drive. At Port Center Way shaft, the main 565-ton TBM and TBM tail shield were hoisted out of the shaft in two pieces using four 220-ton strand jacks mounted on a 700-ton hydraulic boom gantry system as illustrated in Figure 18. After lifting the TBM components out

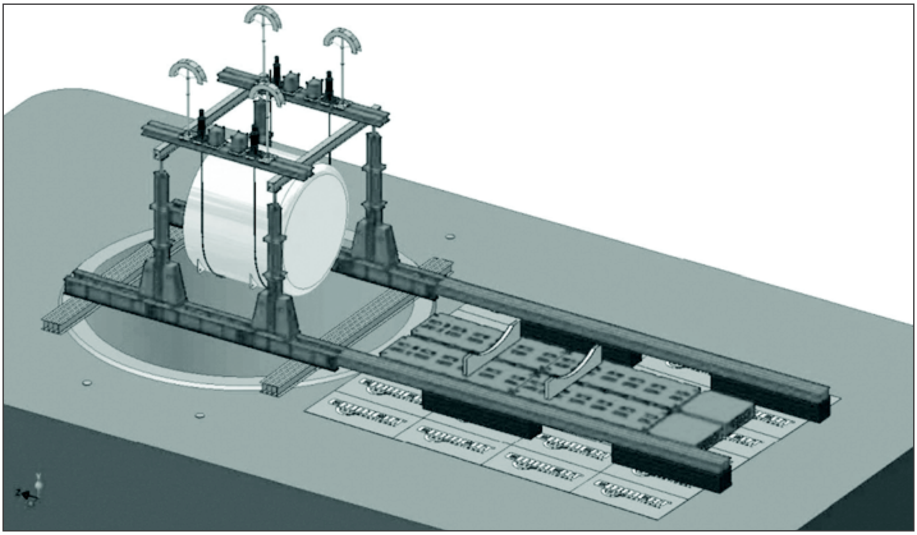

Figure 18. Schematic of strand jack gantry system at Opera shaft

Figure 19. Barge loaded with the TBM and tailshield at the roll-on site

of the shaft, the gantry system loaded the TBM components onto two platform trailers, which then transported the TBM and tail shield to a barge utilizing a roll-on/roll-off method, see Figure 19. The barge transported the TBM and tail shield up the Willamette River, back to the Opera shaft site where the process was repeated in reverse for re-installation of the TBM inside Opera shaft (Kofoed et al. 2011).

SCHEDULE AND PRODUCTION

Table 7 displays the as-built schedule for the tunnel work and Table 8 the performances achieved during the two tunnel drives.

Table 7. Achieved schedule

Activity Description	Start Date	Finish Date
Project NTP	3/21/2006	
TBM Build & Delivery to Site	3/21/2006	3/15/2007
Tunnel Site Setup	10/24/2006	5/11/2007
TBM Assembly	2/20/2007	5/15/2007
Mine North Tunnel	6/2/2007	11/12/2009
TBM Transition - North - South	11/12/2009	3/5/2010
Mine South Tunnel	3/5/2010	10/18/2010
TBM Demob at McLoughlin	10/19/2010	1/26/2011

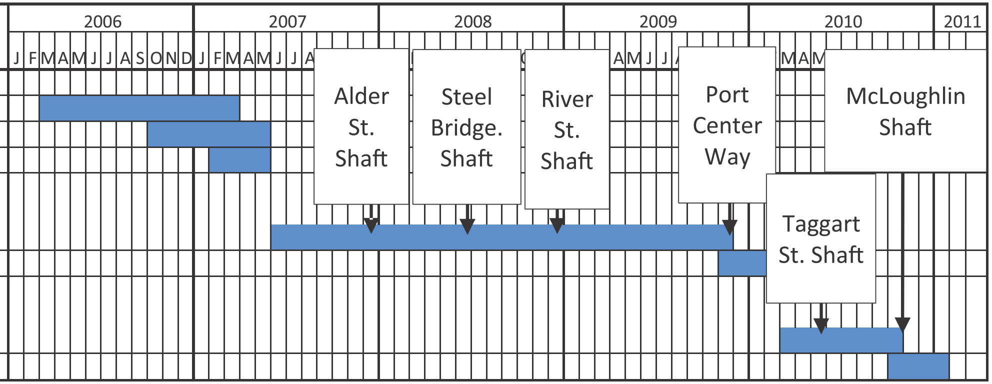

Table 8. Best performances

	North Drive	South Drive
Best Day	25 Rings	24 Rings
Best Calendar Week	110 Rings	93 Rings
Best Month	320 Rings	315 Rings

Considering all challenges addressed above, the construction of the project was successfully constructed on time and within budget. The performance beat all expectations in a team effort between the owner, designer, and contractor.

CONCLUSION

In conclusion, the Portland ESCSO tunnels were successfully completed by thorough up-front planning and disciplined execution in the field and when unforeseen problems arose during the tunnel drives they were successfully dealt with by a team of highly skilled engineers and craft workers working together to develop and execute often innovative solutions to these challenges.

The type of contract allowed the contractor to focus on technical challenges during daily operations. Therefore, all challenges were well-managed in a short amount of time, allowing minimal effects to the operational schedule. All involved parties of the ESCSO Project were satisfied with Safety, Quality, Environmental, and Production related aspects.

ACKNOWLEDGMENT

The authors thank Christof Metzger for contributing to this paper.

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CONSTRUCTION OF THE UNIVERSITY LINK LIGHT RAIL TUNNEL U230 IN SEATTLE, WA

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ABSTRACT

The Sound Transit University Link project is a 5.07 kilometers light rail extension that will run in twin bored tunnels from Downtown Seattle north to the University of Washington, with stations at Capitol Hill and on the University of Washington. Contract U230 of this project includes the installation of the twin-bored tunnels with the length of 1.18 km from Capitol Hill Station to the Pine Street stub tunnel in Downtown Seattle, excavation of five cross passages between the two tunnels, excavation of CHS station, installation of temporary support for CHS, and placement of the bottom slab. This paper outlines the main features of this unique project and highlights the challenges and solutions found during the construction.

INTRODUCTION

This paper describes the work completed for the Sound Transit University Link Light Rail TBM Tunnels (CHS to PSST)–Link Contract U230. This project includes creating the underground space that will be used to extend Seattle's light rail from downtown to the Capital Hill neighborhood, and is part of the overall University Link program which is made up of 10 separate construction contracts, including 2 tunneling contracts (U220 and U230) totaling approximately 5.0 Kilometers (3.15 miles), and is worth an estimated $1.9 Billion. When the University Link program is completed late in 2016, light rail service will be provided from the University of Washington to downtown Seattle. The previously completed Central Link project currently provides service from downtown to Sea Tac airport. This 22.5 kilometer (14 mile) section of light rail started providing service in July of 2009.

PROJECT DESCRIPTION

The U230 project was awarded to JCM U-Link, a joint venture of Jay Dee Contractors–Coluccio Construction Co–Michels Corporation JV (JCM) in September of 2009 and the Effective Notice To Proceed (ENTP) was issued on January 11th, 2010. U230 is comprised of 1.16 km (3,800 ft) of 5.74 m (18 ft–10 in) finished diameter twin tunnels with invert and walkway, 5 cross passages, 2 low point sump alcoves, and a station box excavation approximately 170.7 meters (560 ft) long, 22.6 meters (74 ft) wide, and 22.9 meters (75 ft) deep with approximately 3 meters (9.8 ft) of reinforced concrete invert. The project is expected to be completed in early 2013, with an approximate total value of $155 Million.

The project is in the most densely populated neighborhood in the Northwestern U.S, and the relatively short tunnels crossed under 32 structures, and multiple utilities many of which are 80+ years old. The tunneled portion also included 2 crossings of

Interstate 5 (I-5) which is a key transportation artery carrying approximately 250,000 vehicles per day through downtown Seattle.

The crossing of I-5 by the U230 tunnel was dependent on the completion of the U215 Contract which involved sinking shafts in the on/off ramps on either side of the freeway and clearing the tunnel envelope of the steel piles that make up the retaining wall on either side of the freeway. Once these piles were removed the shafts were backfilled with controlled density fill (CDF) and traffic reinstated on the overlying on/off ramps. The CDF filled shafts provided safe havens for the TBM, as well as locations for the construction of the low point sumps in either tunnel.

Geologic Conditions

The project is located within the Puget Trough, which is a structural basin located between the Olympic and Cascade Mountains, formed by the Juan de Fuca oceanic plate being thrust beneath the North American Continental plate. The bedrock contact is over 305 meter (1,000 feet) below the surface, and is overlain by glacial and non-glacial sediment through which the tunnel will be constructed.

The geologic history of the project site is dominated by at least seven different episodes of advance-retreat cycles of continental glaciers during the Pleistocene era. Each of these glacial advances partially eroded the pre-existing stratigraphy, and deposited a fresh sequence of sediment. Consequently the stratigraphy along the U230 tunnel alignment is quite complex due to the multiple erosion/deposition cycles that have occurred during the time that these materials were at the surface.

During the last glacial period (which ended about 10,000 years ago and included at least 7 distinct advances and retreats in the project area) large quantities of sediments ranging from fine clays to large boulders were deposited in the Puget Trough. Each depositional event was followed by one of erosion during which large and small channels, ravines and valleys were incised into the previously deposited sediments. Subsequent deposition either filled or partly filled those channels, ravines and valleys, then the process was repeated again and again Consequently, many if not all of the formations are only remnants, and refilled channels are common.

The depositional environment for the soils encountered during the excavations of U230 is of two categories, and three basic types. Vashon deposits, which were primarily ice deposited, and Pre-Fraser deposits which were water deposited, either fluvial, or lacustrine.

Vashon deposits are associated with the Vashon glacial advance (ending about 10,000 years ago), and were encountered primarily in top 50 feet of the Capital Hill Station (CHS) box excavation and in the section of the tunnels adjacent to the CHS including Cross Passage 5. The majority of the Vashon deposits encountered were glacial till or till like soils which were very hard erosion resistant soils (such that dozer rippers were required for excavation at the CHS), with low permeability, heterogeneous deposits of varying grain sizes ranging from clay to boulders. A very small portion of the Vashon deposits were very permeable medium sand outwash deposits.

The Pre-Fraser deposits were in the project area were deposited approximately 25,000 years ago, and were encountered in the bottom 30 feet of the CHS excavation and the last 2,000 feet of the tunnels. These deposits were primarily silty sand in the CHS excavation except at the very bottom where they graded to hard cohesive silts and clays. Where encountered in the tunnel excavations these material were primarily hard cohesive silts and clays, with notable occurrences of very dense cohesionless silts, and very silty sands.

The Geotechnical Baseline Report for the project indicates that 74 boulders expected to be encountered by the TBM in each tunnel (although 57 of these were expected to be less than 2 feet in maximum dimension). The results of multiple boulder encounters was evident on the cutterhead at the completion of both drives, however these encounters did not disrupt production. Based on the relative abundance of boulders found during the excavation of the CHS, it is assumed that most of the boulders were in the Vashon deposits, although one fairly large (greater than 4 feet in greatest dimension) boulder was removed from the excavation of Cross Passage 3 which was in the Pre-Fraser deposits.

Groundwater

The glacial advance and retreat coupled with the interglacial periods of fluvial erosion and deposition as well as lacustrine deposition, created a sequence of aquifers and aquitards with varying thicknesses and lateral continuity.

The anticipated groundwater head at the tunnel invert varied from 0 to 65 feet, and the maximum baselined hydrostatic pressure at springline was 1.7 bar. Approximately 1,200 lineal feet of the tunnel was excavated in more or less unsaturated soils including the crossing of I-5, and all of the launching and holing out at either end of the tunnels.

The actual hydrostatic pressure encountered by the TBM varied considerably, with much of the tunnel being in ground that was practically impermeable.

Ground Classification

Nearly 90% of the soils expected to be encountered in the main running tunnels was classified as Firm to Slow Raveling in the Geotechnical Baseline Report (GBR), and every indication is that this was what was encountered. One attempt was made to enter the chamber at atmospheric pressures at the first cross passage location (Cross Passage #5), but the ground was not stable enough to support itself and the face inspection was aborted.

Of the 5 cross passages, 2 (Cross Passage #3 and #4) were expected to have flowing soils. It was originally anticipated that the installation of gravity drainage points around these excavation would suffice to lower the pore pressure to acceptable levels. Due to the low permeability of the soil gravity drainage turned out to be insufficient for producing stable conditions, and extensive vacuum depressurization systems were required at Cross Passages, #3, #4, and #5 in order to stabilize the ground.

Figure 1 is a geologic profile based on the information provided in the GBR and Geotechnical Data Report (GDR). The only location where the ground was found to be materially different in regards to the tunneling was at Cross Passage #5 where the overlying outwash was within 2 feet of the top of the cross passage. To ensure the stability of the crown during excavation of Cross Passage #5 it was determined that the overlying outwash needed to be completely depressurized.

Pre-Tunneling Work

Once the initial safety, quality, and pollution prevention plans were developed, submitted, and approved, the early work was started. The early work included a the construction of a wall around the CHS site, which was between 8 feet and 24 feet high, improving the ground at the tunnel eyes through jet grouting, installation of the soldier piles for the support of excavation of the CHS box excavation, and a significant amount of utility relocation. During the jet grouting work under E. John Street on the north side of the site, pressurized grout and air found its way to a pre-existing monitoring well resulting in an explosive discharge into the street. Thankfully no one was injured and no personnel property was damaged. A small amount of high ph water did flow into the storm drain resulting in a non-compliance event in regards to JCM's pollution prevention program.

As part of the site development the City of Seattle agreed to abandon a section of East Denny Way through the Captiol Hill Station site. In its final configuration the abandoned right of way (ROW) will be converted to a 'festival street' where the neighborhood

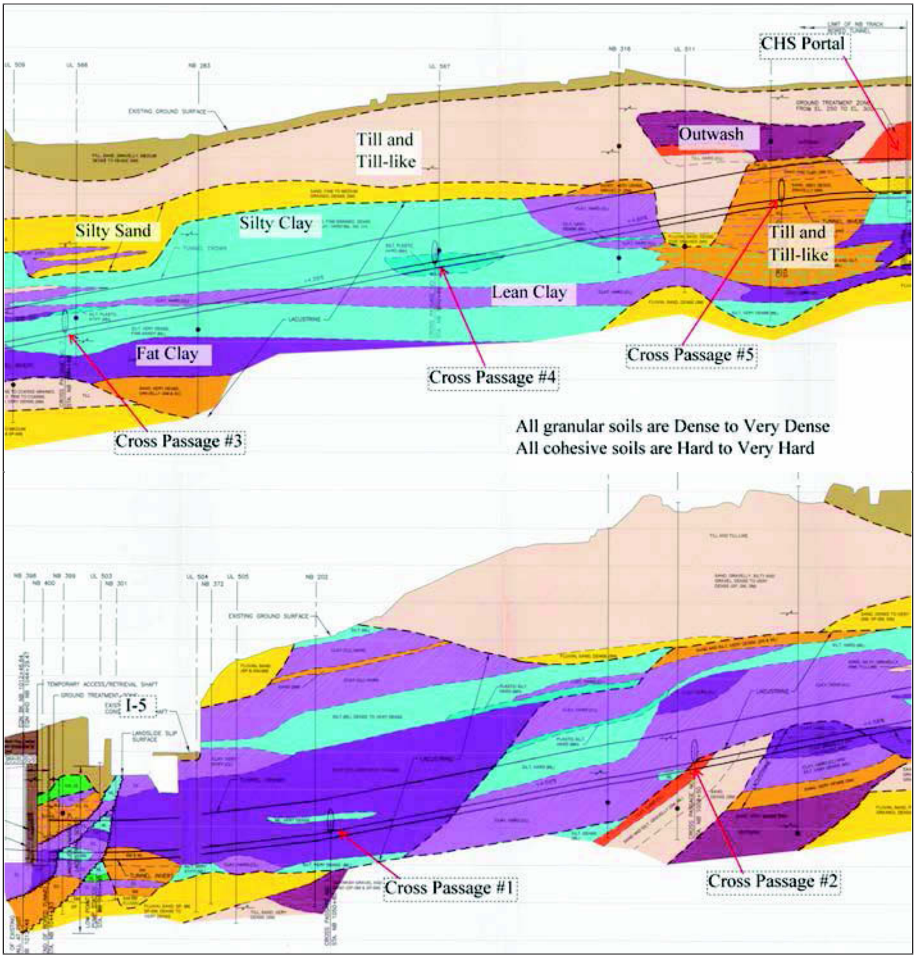

Figure 1. U230 geologic profile for tunnel excavation

Farmers Market and other community events will be held. The ROW splits the site with about ¾ of the box excavation was north of E Denny Way and the remainder was on the south side. The ROW of Denny contained several underground utilities, including an 36" egg shaped brick sewer, a 36" concrete storm drain, and a 16" high pressure gas line among others. Once all of the utilities were relocated and supported such they could span the 65' excavation, a bridge was installed to replace the Denny Way roadway to allow JCM to have access across the project site.

The excavation of the CHS box and the placement of the station concrete invert slab, required the removal of 100,000 bank cyds of soil, the placement of 1,300 tendon tiebacks, 350 soil nails, 4 million lbs of reinforcing steel, and 11,500 cyds of cast in place concrete. During the course of the box excavation a differing geologic condition was encountered that resulted in a significant drop in the efficiency of the excavation method. This change resulted in the box excavation finishing more than 2 months behind schedule. This delay was successfully mitigated by making nearly the entire invert slab construction concurrent with either the completion of the excavation, or the assembly of the Tunnel Boring Machine (TBM). This required re-sequencing the

Figure 2. CHS Site Just Prior to TBM arrival

original invert pour schedule and combining some of the individual placements into larger ones. This led to 5,500 cyds of concrete being placed in a single day in order to make room for the arrival of the TBM.

As the soil nails were being installed in the north headwall of the CHS box excavation, it was determined that the jet grout block was not complete and some remedial work would be needed to ensure that the entrance eye for the U220 TBMs was completely intact. This remedial work was performed after the final concrete slab had been poured. During the course of this work an inclinometer was struck by a drill and high pressure air and water was explosively discharged at the surface. This discharge occurred inside a covered pedestrian walkway that was installed at the northern edge of the CHS site. This walkway was very popular with the local public as it had been fitted with several windows that could be used to view the worksite, including full view of the location where the TBM was assembled. Thankfully, the pedestrian walkway was empty at the time of the mishap and no one was injured.

As the CHS location was being prepared for the arrival of the TBM, work was ongoing at the Pine Street Stub Tunnel (PSST) site for the eventual connection of the U230 tunnels to the current Downtown Seattle Transit Tunnel and the operating portion of Sound Transits Link Light Rail l system (Figure 2). The work at the PSST site included the excavation of a 75 feet deep reception shaft for the Northbound (NB) tunnel with an adit excavation laterally to connect to the terminus of the Southbound SB tunnel. Also included, was the installation of the facilities required to isolate the U230 construction project from the active transit tunnels carrying light rail, and bus traffic under downtown Seattle.

In addition to the major civil work that was required prior to the start of tunneling, a significant amount of instrumentation needed to be installed in order to monitor any ground movement resulting from the underground excavations. The instrumentation utilized for the excavation of the CHS box and the PSST reception shaft included extensometers, inclinometers, monitoring points (surface, structural, and utility), and vibrating wire piezometers totaling over 150 in all. For the tunnel (excluding the crossing of I-5)*, the same types of instruments were used with a total of 250 for tunnel and cross passages.

* The majority of the instrumentation utilized to monitor the U230 crossing of I-5 was installed by a previous contract (U215), and is being described in a separate paper.

Tunnel Construction

The overall plan for tunnel construction was to design and manufacture one Earth Pressure Balance (EPB) TBM that would be utilized for the excavation of both the NB and SB tunnels. The best proposal received for the design and manufacture of the TBM was from Hitachi-Zosen whose manufacturing facility is in Sakai City Japan (Figure 3).

The critical factors that needed to be addressed in the TBM design were the expected geologic conditions, the very tight radius curves in the alignment, the number of times that the TBM needed to be assembled, and the very urban location of the project.

The geologic conditions resulted in the TBM having a very robust cutterhead designed to break any boulder up into digestible sized chunks, and a ribbon screw designed to make the digestible size as large as possible. The cutterhead design was intended to be robust enough to allow the TBM to excavate the entire 3,800 feet of each tunnel without requiring replacement of the cutters, including extended length gage cutters, and copy cutters. The screw conveyor was made up of 2 separate screws and was over 80 feet in length in order to allow for redundant guillotine gates to prevent ground loss that could result in damage to the overlying infrastructure. The soil conditioning system was manufactured for the extremely variable soil conditions expected, and was equipped to deliver multiple types of conditioning agents including water, foam, bentonite, and polyacrylamide. The overall project schedule had more time allotted for putting the TBM together, taking it apart, and moving it around, than the time allotted for actually excavating the tunnel. Therefore, the TBM had a modular design to facilitate the efficiency of the multiple assembly and disassemblies that were required on the U230 project.

The TBM was delivered to the jobsite in April 2011. This delivery was 2 weeks late due to the tragic earthquake and tsunami which occurred in Japan on March 11 2011. Christened Brenda, it weighed 1,000 tons. The total length 342 feet long including the 8 Hitachi Zosen designed and manufactured backup gantries combined with 3 modified gantries from the TBM that had been utilized on the Beacon Hill project. Total thrust was 4,000 tons, and the installed cutterhead power was 720 kw, which was delivered by 8 electric drive motors.

Tunnel Planning

The majority of the planning for the project was documented in the Final Tunnel Excavation Plan (FTEP). This document ended up being quite voluminous with well over 500 pages of plans, appendices, and comments. In addition to the FTEP, JCM prepared approximately 20 other written plans specifically for the U230 tunneling activities including safety plans, quality plans, work plans, training plans, and contingency plans.

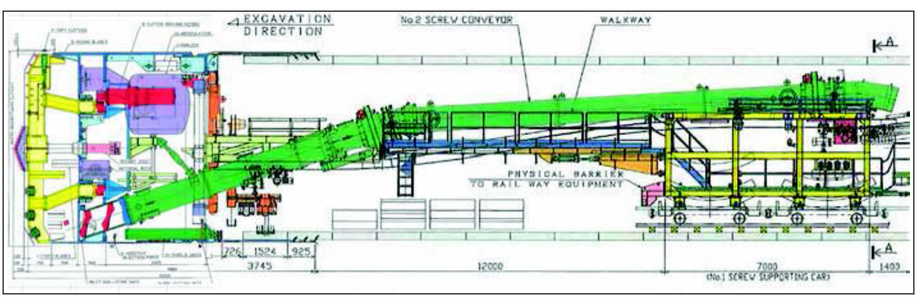

Figure 3. Hitachi Zosen 6.44 M (21 ft 1.5 in) TBM

The two primary issues that needed to be addressed during the planning for the tunneling by TBM were the downhill grade and the curved alignment.

Only 0.18% of the tunnel was not on a curve, and 29% of the tunneling was on both a horizontal and a vertical curve. Most of the tunnel was on a 1,500 ft radius curve, however both tunnels had a short stretch of 550 ft radius curve. The tight radius curve impacted both the TBM design and the segment design, and was the single most important factor in the decision to use a TBM specifically designed for the U230 project. Three separate gyroscopic surveys were planned and carried out for each tunnel as they were being excavated.

The downhill grade (the tunnel dropped over 150 feet over the course of less than ¾ of a mile) primarily created a need for redundant pumping capacity in the TBM in order to prevent flooding of the machine.

The segments were of the Ups and Downs variety, and the U220 and U230 segments were made using the same molds. In addition to the standard Ups and Downs, which had a 2.5 inch taper over the 60 inch ring lengths, Specials (with a 4.5 inch taper) were made in order to negotiate the tight radius curves. The use of the Special rings greatly increased the complexity of ring-building and required additional planning and engineering support before and during the tunneling through the tight radius curve locals.

The target mining pressure was developed by using 3 different methods resulting in target pressures varying between 0.1 bar and 4.3 bar. The anticipated pressure was based on an average of the results of the 3 methods with a safety factor of 1.3, and the final target pressures varied between 0.6 bar and 3.7 bar. These pressures were understood to be very conservative given the urban nature of the project and the high risk associated with settlement damage. In comparison, the TBM manufacturer indicated that the maximum mining pressure was expected to be 1.99 bar.

NB Tunnel Excavation

The final load of TBM components arrived on site on April 16, 2011. The assembly was completed and the launch of the TBM on the NB drive occurred on July 07, 2011, and was completed on November 21, 2011. The assembly, training, and startup of the TBM was challenging due to the language barrier that existed between the Hitachi Zosen and JCM field personnel. This was somewhat overcome through the use of email correspondence, and the use of several translators on-site.

As part of JCM focus on minimizing the risk of settlement damage, long gauge cutters were proposed. The use of long gauge cutters greatly decreases the need for cutterhead maintenance, which requires that the cutterhead chamber be partially emptied of muck, greatly increasing the possibility of instability at the face of the excavation. (see DiPonio et al., 2011 and Frank et al., 2011). The use of the extended overcut allowed JCM to complete the entire 3,800 lineal feet of tunnel without needing to replace any cutting tools.

JCM attempted a free air face inspection at the location of Cross Passage #5, which was about 250 feet from the launch portal. The ground at this location was baselined to be very hard till that extended to approximately 5 feet above the top of the main tunnel excavation, and was overlain by a saturated outwash formation. Based on information obtained during the installation of the extensometers in this location there was concern that the outwash extended down into the main tunnel, and possibly down to the elevation of the top of the cross passage excavation. During the attempted face inspection it was confirmed that the outwash did extend into the tunnel face making the face inspection impossible under free-air, and the attempt was aborted.

After the trailing gear was completely inside of the tunnel the continuous conveyor and belt storage cartridge was installed to transport the tunnel muck out of the tunnel.

Figure 4. Muck pit and truck loading on the NB Drive

A wall was built across the shaft to form a muck bin, and a material handler was placed on the wall from which it loaded trucks on the surface (Figure 4).

Approximately halfway through the NB drive a monitoring well remaining from the geotechnical investigation was located in the tunnel excavation envelope. The water level in this well was being actively monitored as part of the instrumentation monitoring program and it was scheduled to be abandoned prior to the time when the TBM arrived, unfortunately this did not occur. The oversight was recognized just prior to the TBM arrival, but since the depth of cover was fairly high in relation to the mining pressure, it did not appear to be any danger in mining through the well. However, due to the fact that there was a significant amount of foam (compressed air) in the cutterhead chamber at the time that the TBM encountered the well, there was a explosive discharge of air, water, and silt at the surface. Thankfully no one was injured, although several parked cars needed to be cleaned, and both the fire department and the police department responded to the scene.

The last third of the alignment was primarily in firm clay and much of it was above the water table, however the final section under the I-5 was expected to be disturbed and was characterized and landslide deposits in the GBR. The primary concern for the crossing was maintaining enough pressure in the plenum to prevent settlement due to lack of face support, while keeping the pressure low enough such that no fluids escaped onto the travel lanes of the freeway. There was ample evidence that multiple high angle faults existed in the ground which would provide fairly resistance free pathways for soil conditioning foam and water to travel from the face of the excavation to the travel lanes of the freeway, only 15 feet away (Figure 5).

The 2 express lanes were closed early on the evening that the TBM started under the freeway so there was no traffic on the lower roadway for the first 8 hours of the crossing, although the other 13 lanes remained active. Based on inspections of the

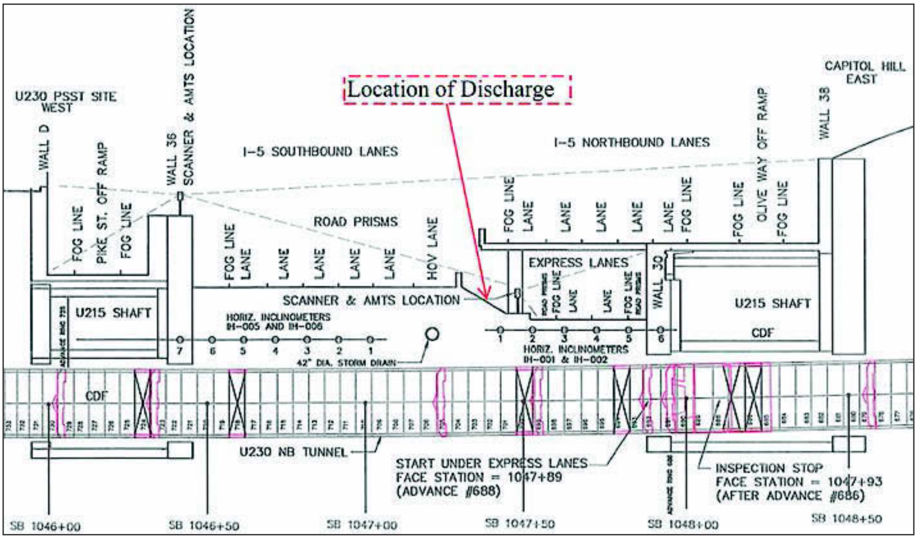

Figure 5. Sketch of the I-5 crossing

travel lanes and instrumentation monitoring data, the excavation was proceeding as planned (disregarding the false alarm due to the malfunction of one of the horizontal inclinometers) with approximately ¼ inch of settlement. This small amount of settlement indicated that the pressure in the plenum was optimal. JCM was tunneling with the minimum amount of soil conditioners needed to maintain muck flow from the plenum and a constant support pressure around the face.

At 4:00 AM the express lanes were cleared of project personnel and were opened up to traffic at the usual time of 5:00 AM There were no reports of any problems until the express lanes were closed again at 11:00 PM at which point the TBM had completed over half of the crossing. At some point during the evening silty foam and water started to flow out of the vegetated slope between the express lanes and the southbound lanes. The material flowe had accumulated along the shoulder of the express lanes, but not had not moved into the travel lanes, so the traffic on the freeway was not impacted. JCM was able to staunch the flow, and clean up the mess in time to allow the express lanes to reopen on time at 5:00 AM.

Shortly after completing the crossing of I-5 the NB tunnel excavation was completed on November 21 when the TBM holed out into the reception shaft at the PSST site. The cutterhead and shielded portions of the TBM were removed and trucked back to the CHS site. The backup gantries and screw conveyors were taken back to the CSH site through the NB tunnel.

SB Tunnel Excavation

As is typical with twin tunnels that are done consecutively the second drive went better than the first. Due to the modular design and now being familiar with the TBM JCM was able to reassemble the TBM much more rapidly than the initial drive. The TBM sections were trucked from the PSST back up to CHS on December 10, 2011. Even with the holiday season, and a significant weather event causing damage to the primary transformer, the TBM was reassembled and launched on January 27, 2012. Despite having to reconfigure JCM's work area to accommodate the arrival of the U220 TBMs onto the CHS site the overall excavation production was improved by approximately 15%, with the work finishing up on May 2, 2012 (Figure 6).

Figure 6. CHS site during the SB tunnel drive

The increased production was partly due to a decrease in the target mining pressures on the SB drive when compared to the NB drive. The data gathered during the SB drive was analyzed, and the apparent ambient pressures in the formation at each point along the alignment was determined. Based on this analysis new target mining pressures were developed for the SB drive, which on average were 33% lower than the target mining pressures on the NB drive.

Along with the improvement in production, an improvement in the quality of the tunnel installation also improved on the second drive. There were no discharges of materials to the surface, and the second crossing of I-5 went without incident. There were also significantly fewer cosmetic repairs required in the SB tunnel, due to minor spalling and cracking of the segments.

The design, and JCM's original plan envisioned the abandonment of the TBM at the end of the SB drive. The shield was to be left in place and all of the TBM components including the cutterhead cut into pieces and removed. However, by modifying the configuration of the adit, and the support of excavation of the reception shaft JCM was able to recover nearly the entire TBM at the conclusion of the SB drive. Only the tail shield was left in place, which was required due to the length of the tail shield and the lack of space in the adit. This modification to the design improved the overall quality of the tunnel installation by minimizing the amount of cast-in-place concrete required.

Cross-Passage Construction

Five cross passages were constructed between the NB and SB tunnels on the U230 project. The construction sequence included an investigation stage, a pre-support stage, an excavation stage, and 3 stages of final lining with cast in place concrete. Both tunnels needed to be supported during the excavation phase of the cross passage construction sequence, and JCM used steel propping rings to keep the deformations inside of the tunnels within acceptable limits.

The expected ground conditions was incorporated into the design by categorizing the excavation into Type 1 and Type 2. These categories referred to the type of temporary support measures that would be required and the sequence of the installation of those support measures, which were described in different drawing sets for Category 1 and Category 2 cross passages. The primary difference between the two types of cross passages was whether the entire cross passage was in cohesive ground (Category 1) or the face was expected to contain portions that was cohesionless ground (Category 2). Additional support measures (including depressurization) and smaller excavation advances or lifts, being required for Category 2 vs. Category 1 (Figure 7).

Cross Passages #3 and #4 were classified as Category 2 in the GBR with Cross Passage #1, #2, and #5 being classified as Category 1. The original plan was to start with Cross Passage #5 proceeding to #4, #3, etc. This sequence had to be modified however when the initial probe drilling performed on February 25, 2012 indicated a significant change in the ground conditions at Cross Passage #5 when compared to the

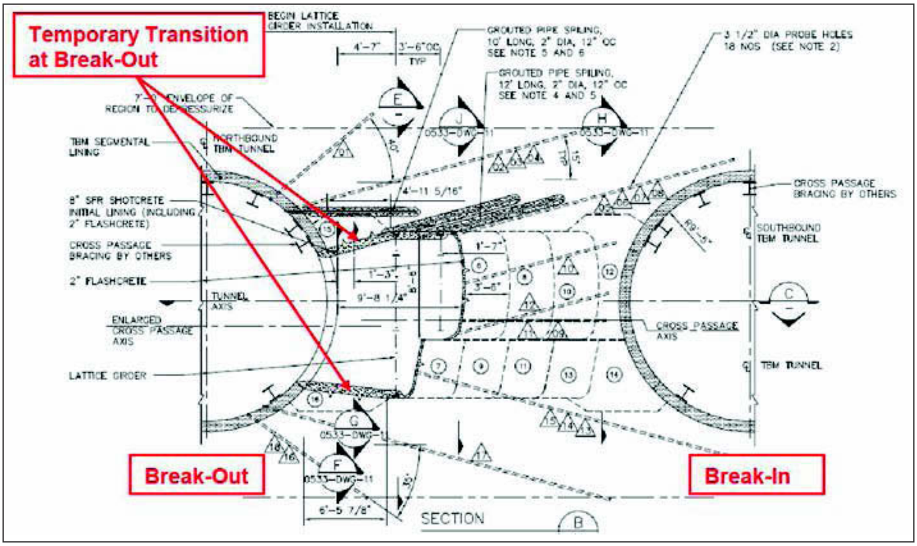

Figure 7. Construction sequence for Type 2 cross passage

GBR. JCM decided to move to Cross Passage #4 in order to get started on the cross passage work while the path forward on Cross Passage #5 was worked out.

JCM started on probe drilling at Cross Passage #4 on March 12, 2012 and encountered very unstable flowing ground, which could not be stabilized by gravity drainage. A vacuum depressurization system was designed and installed, which allowed excavation to commence on April 09, 2012, and it was completed on May 02, 2012.

Concurrent with the excavation of Cross Passage #4 probe holes were installed at Cross Passage #3, and even more challenging conditions were found. Due to the very unstable cohesively flowing/collapsing behavior of the ground at Cross Passage #3, the methods that had been successfully used at Cross Passage #4 were ineffective and JCM had to design a different vacuum depressurization system to use there. The depressurization system used at Cross Passage #3 included the installation of 26 vacuum drain pipes and 6 piezometers that were monitored every 2 hours by field personnel and the pressures were data-logged and analyzed daily by JCM engineering staff. Meanwhile the excavation of Cross Passage #2 was started, and by the time it was finished late in June conditions at Cross Passage #3 were sufficiently stable for the start of excavation. Despite very dicey conditions at times, the excavation of Cross Passage 3 was completed in late July with only a negligible amount of lost ground.

While the other cross passage excavation had been ongoing a significant amount of work had been completed in attempt to define the conditions and develop a plan to move forward for the excavation of Cross Passage #5. The end result was 37 drain pipes, 7 piezometers, and 2 vacuum pumps installed from inside the tunnel in order to depressurize the overlying outwash formation. The excavation of Cross Passages #5 and #1 were completed by September 4, 2012.

Low Point Sump Construction

The low point sumps are constructed inside CDF filled, reinforced concrete structures created by the U215 Contract on the west side of the I-5 crossing. The original design for both of the low point sumps called for removal of approximately 30 feet of the segmental liner and the CDF backfill from the lower cell of the U215 structures. The sump was then constructed on one side of the cell with a cast in place arch poured

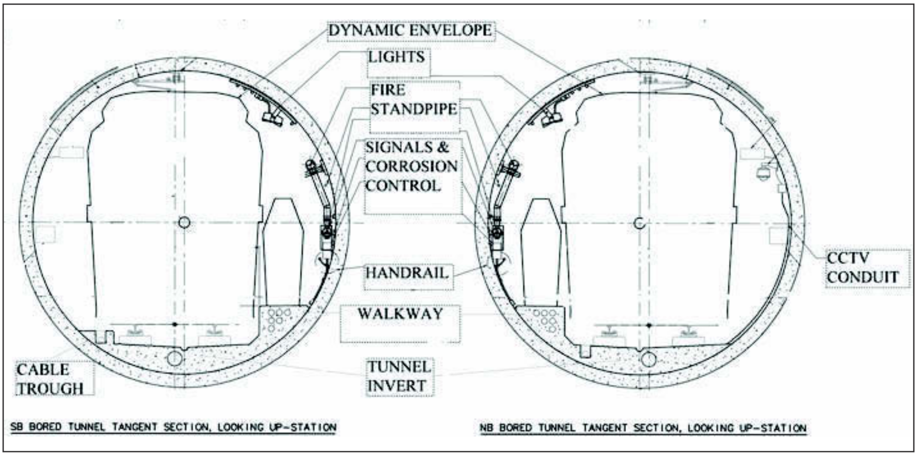

Figure 8. Tunnel finishes

inside of the U215 box to form a roof over the combined area of the running tunnel and sump. Due to several factors including the amount of waste material generated, the volume of new material that needed to be brought in, the disruption to access through the area and the hazards associated with removing the tunnel segments from inside of the tunnel structure, JCM determined that a better solution could be designed and implemented.

JCM initiated a Value Engineering Change Proposal (VECP) that was finalized in cooperation with Sound Transit representatives. The final design developed by JCM was for the sumps to be constructed similar to the cross passages by creating “alcoves” excavated out of the side of each tunnel. These alcoves required that approximately 120 degrees of the segmental lining (from 1 o’clock to 5 o’clock) of the tunnel was removed from 30 feet of each tunnel. Once the tunnel lining was removed, the CDF was excavated back to the concrete wall (approximately 8 feet at springline) of the U215 shaft and a cast in place structure was built to house the low point sumps in each tunnel. The tunnel was supported utilizing the same propping rings that were used to support the tunnel during the construction of the cross passages.

The construction of the low point sumps was among the last activities JCM completed on the project with the excavation of the SB sump alcove starting on September 10, 2012 and the roof concrete for the NB sump being placed on January 15, 2013.

Tunnel Finishing

The finish work required in the running tunnels is shown in Figure 8.

JCM completed this finish work in conjunction with the excavation and primary support of the SB tunnel and cross passages. The invert of the NB tunnel was placed concurrent with the excavation of the SB tunnel and prior to the start of the cross passage excavation. In addition to the finish work in the running tunnels, JCM installed fire standpipe, electrical and signal conduit, and tunnel lighting in the cross passages. The controls and discharge piping for the low point sump was also included in the U230 project.

The finish work started as soon as the NB tunnel was cleaned after the TBM was removed in February 2012, and continued as other work allowed until the end of the contract in March of 2013.

Figure 9. Formwork for final SB tie-in to the pre-existing PSST (the light at the end of the tunnel)

Pine Street Stub Tunnel Connection Work

The southern end of the U230 project needed to tie into the Pine Street Stub Tunnels (PSST), which were short sections of track that were constructed between 2004 and 2007. Since these tunnels were used by Sound Transit as a cross-over track to allow the light rail trains to “clear the track” for bus traffic in the shared downtown tunnels, they were considered “live." This designation required that JCM build a “demising” wall across the stub tunnels in order to completely isolate the U230 project from the active transit tunnels. The demising walls were constructed in March of 2012 and allowed JCM to break out the end walls of the stub tunnels in preparation for the connection work.

The connection work itself involved tying in the discharge line from the low point sump to the stub tunnel plumbing, and connecting all of the power and control conduits. After the embedded lines were connected the SB tunnel was completed by placing the rebar and forms required to backfill the entire adit with structural concrete. The final pour in the SB connection was performed on 12/21/12 (Figure 9).

The NB tunnel is located in the bottom of the PSST shaft and is to be left open to provide access for the follow on contractors to perform rail installation and systems work. JCM only poured the invert and the walls of the NB tunnel to allow this continued access. The final concrete on the U230 project in the NB connection was placed in mid-January 2013. Once all of the concrete was in place, the fire line and electrical installations could be completed and tested in order to make the mid-March milestone for substantial completion.

SUMMARY

At the time of writing this article, JCM has finished all of the structural concrete placement on the contract and is expecting an on-time completion of the project.

This project has been very challenging for all parties from a technical, a community relations, and a safety standpoint, and lessons have been learned on all sides. The technical challenges were many, including the crossing of I-5 with less than a diameter of cover and cross passage excavation in unstable flowing/squeezing soils, both without impacting the overlying infrastructure. The community relations challenges included stringent noise control and multiple third-party stakeholders. The safety challenges included not only the safety of men and women working underground, but the many members of the public who were often completely unaware that the project could impact them. JCM is extremely proud and thankful that no member of the public was hurt during this project that included the removal of over 300,000 cubic yards of material from beneath, and then over public roads that are heavily utilized, by pedestrians, skateboarders, cyclists, and motorists

ACKNOWLEDGMENTS

The authors would like to acknowledge all of the employees of JCM, whose hard work and diligence has resulted in a project that we all can be proud of. We would also like to thank Jay Dee Contractors, Inc., Frank Coluccio Construction Co., and Michels Corp. for their valuable insight and support. Finally thanks to Sound Transit for the opportunity to construct a project of this magnitude.

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EPB TUNNELLING AT HIGH PRESSURES: CUSTOMIZATION OF TUNNELLING SYSTEMS FOR PORT MANN TUNNEL

Keivan Rafie ▪ McNally

Steve Skelhorn ▪ McNally

Allen Mitchell ▪ McNally

ABSTRACT

Metro Vancouver is constructing a new water supply main under the Fraser River, just downstream of the Port Mann Bridge. A joint venture of McNally International and Aecon Constructors was awarded the contract for this highly challenging project. When complete, this new water main will help ensure the continued reliable delivery of clean safe drinking water to the municipalities south of the Fraser River, and will substantially increase the capacity of the existing main.

The water main will be constructed in a tunnel driven through soil, 35 m underneath the riverbed. The 1,000 m (3,280 ft.) long, 3.5 m (11.5 ft.) diameter tunnel will be excavated under the Fraser River at pressures of up to 6 bar using an earth pressure balanced tunnel boring machine (EPB TBM).

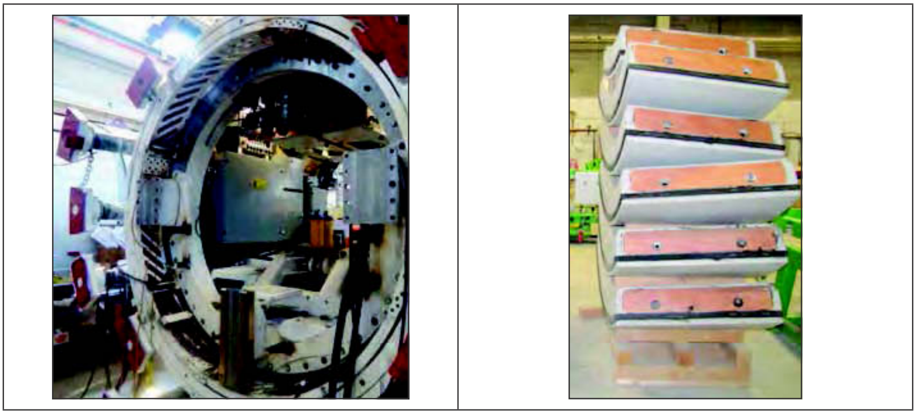
Tunnelling in complex geology, exceptional depths and high hydrostatic pressure involves several technical challenges. This paper describes the approaches used in customization of TBM and tunnelling systems to suit the aforementioned conditions. These modifications were implemented in various areas including but not limited to: design of TBM shield and bulkhead, emergency systems for water ingress, layout of cutting tools, and contractors procedures including provisions for high pressure compressed air interventions, specialised launch procedures and TBM removal provisions at the receiving shaft.

INTRODUCTION

Metro Vancouver is constructing a new water supply main under the Fraser River, just downstream of the Port Mann Bridge. The water main will be constructed in a tunnel driven through soil, underneath the riverbed. The 1,000 m (3,280 ft) long, 3.5 m (11.5 ft) diameter tunnel will be excavated under the Fraser River at pressures of up to 6 bar using an earth pressure balanced tunnel boring machine (EPB TBM). For this purpose, two shafts will be constructed at north and south sides of river.

The south shaft will serve as the launch shaft for the TBM. The shaft will be sunk using slurry diaphragm walls for primary ground support which are 13 m internal diameter and nominally 68 m deep. The shaft will be excavated in partially flooded conditions before placing a Tremie concrete base. The shaft will then be dewatered, cleaned and lined with an 11 m inside diameter, cast in place, concrete lining. A work slab will be installed at a depth of 51 m prior to installation of a TBM launch can. The final lining will then be finished around the launch can to the underside of a buried 7.2 m deep valve chamber which will be constructed integral with the upper shaft lining. Inclinometers will be utilized throughout the process to ensure ground movements do not exceed design tolerances.

Project location (south side aerial view)



TBM stationary shell and precast segments

The north shaft will serve as the receiving shaft and facilitate the removal of the TBM. The shaft will be installed using the same procedures as the south shaft, except it will be keyed into mudstone bedrock and potentially excavated in the dry if the permeability and stability of the ground permits.

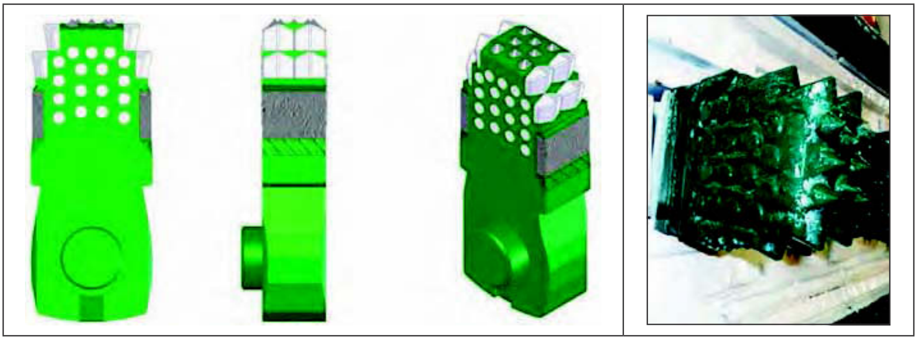
The tunnel will be constructed using a pressurized-face TBM and gasketed precast concrete segmental liner and the annulus will be backfilled with grout. Segmental liner delivery will be facilitated through the south shaft.

To be effective in controlling loss of ground, the pressure applied to the face by the TBM must be sufficient to support the face and limit ground movement and groundwater flow into the excavation chamber. Fluctuations in the pressure applied to the face will occur as the ground conditions and the composition of the spoil within the chamber changes. Continuous monitoring and careful control of the pressure will be applied to the face during tunnel excavation in order to prevent loss of ground and minimize surface settlement.

EPB TUNNELLING AT HIGH PRESSURES

TBM and Cutterhead Design

Because of the risks associated with a head entry under the anticipated conditions (6 bar), the machine has been designed to mitigate or eliminate the need for a



Ripper teeth with state-of-the-art wear protection

compressed air intervention. The following measures have been incorporated into the machine's design:

1. Cutter head design includes a combination of rippers, scrapers and discs to maximize the possibility of performing the drive without having to change tools.
2. Tool wear indicators will be installed and monitored from within the machine and back-loading cutters will be used to enable tool changes from within the plenum if necessary.
3. Robust grizzly-bars will be used to prevent oversized material from entering the head and clogging up the screw.
4. Flood gates will not be used which will minimize the potential for mechanical related failures.
5. Hard wearing plates will be welded to the face of the machine.
6. Elimination of a TBM mounted airlock to maximise clearance in the TBM for a larger screw. A tunnel airlock will be utilised.

Notwithstanding the above, the TBM is designed to allow safe hyperbaric entry should it be necessary. Compressed air workers have worked with the TBM manufacturer through the design phase to assure alignment with all components. The TBM gantry is designed to be easily removable to facilitate installation of an in-tunnel airlock as close to the face as practical.

The TBM has been fitted with sacrificial skin around the stationary shield to facilitate the sealing operation required in the final stages of the reception process at North Shaft.

Face Stability

By definition earth pressure balance provides control of ground movement ahead of the TBM during mining. Initial parameters concerning the selected EPB pressures will be established from the data provided within the GBR and GDR. Ground movement due to the formation of an annulus outside of the segmental lining will be controlled by utilising a pressure and volume balanced system of grouting to maintain a full annulus as the TBM advances.

Under full EPB, areas of highly saturated flowing ground may only be apparent due to the condition of the material within the screw. In such areas, it may be necessary to inject polymer based ground conditioning agents in order to maintain the plug formation within the screw. Polymer will be pre-mixed and injected ahead of the face through injection ports. In extreme cases (ground with minimal fines), it may be necessary to

Modification to cutterhead design to reduce mitigate the need for intervention

add pure polymers to the screw conveyor. As an added precaution, a bentonite holding tank will be maintained on the TBM back up system. This will contain a supply of pre-mixed bentonite and will allow injection to the head to modify the soil and provided sufficient fines for plug formation. The bentonite system consists of a dedicated pump and holding tank which will be recharged from the surface.

Design of Support Pressures

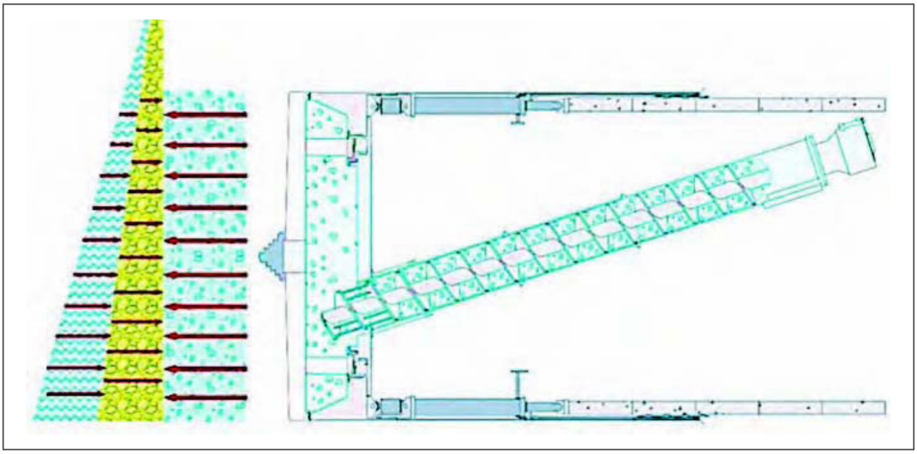
Envisaged EPB settings for intervals throughout the tunnel will be set based upon an evaluation of the known conditions. A calculation reference sheet, together with a graphical interpretation will accompany each sheet of settings. The aim during the drive is to maintain the actual EPB readings, taken from the machine sensors, between a set of upper and lower limits.

Pressure drops across the head vary with different materials; some areas are mined easily with target pressures maintained, other areas require excessive thrust to maintain target pressures and material tends to block resulting in slow mining. It is assumed that in these latter areas a high-pressure drop exists through the material. This is a difficult variable to assess and the actual EPB settings will need modification though the drive.

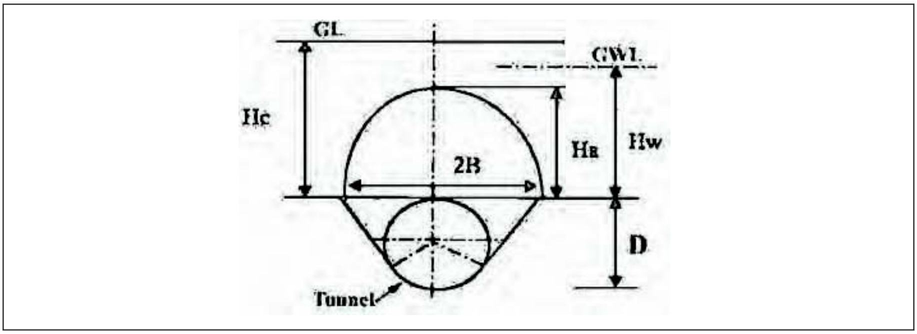
Theoretical calculations for soil pressures are based on “Karl Terzaghi” theory which suggests that the design load head from earth pressure in tunnels with depth of more than 5 times a diameter in running ground would be based on arching at a certain distance above the tunnel, For the purpose of this calculation this distance is taken as 0.4 D.

Controlled Discharge and Water Ingress

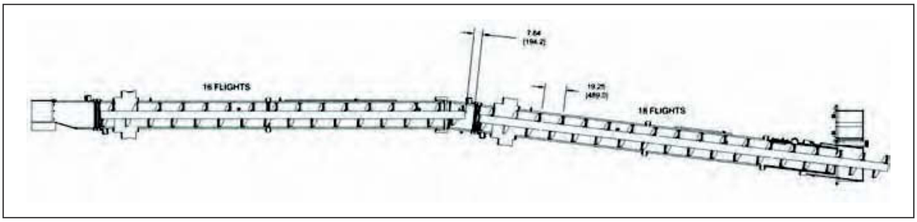
To properly dissipate the EPB pressures from 6 bar to atmospheric to allow for safe and efficient extraction of muck from the working chamber, the total length of screw conveyor has been extended and a double screw system has been utilized on the Port Mann TBM. Several ports along screw have been provided for injection of soil



EPB pressures will be set based on: Static Pressure (as measured at the TBM) + Safety factor



Soil arching over tunnels (Terzaghi's silo theory used to estimate the effective soil column that imposes a load on the TBM as a result of soil arching effect)—*Earth Tunneling with Steel Supports* by Robert V. Proctor and Thomas L. White, published 1977.



Double screw system for safe muck removal from high pressure chamber conditioning material to control the excavated material and to facilitate consistent muck removal.

The full length of drive at Port Mann will be excavated downhill at grades of up to 1.48%, and it will be necessary to control and collect water ingress at the face during tunnelling. In case of an emergency, both screw conveyor sections (front & rear) are equipped with an independent guillotine door. The estimated time to close the guillotine doors using the main bank power is 10 seconds and while under emergency power is 20 seconds.

Water ingress through a breach in the tail seals will be managed by injection of additional tail seal grease and grout. Should water flow not diminish, thicker emergency grease will be pumped through the tail tubes. If this fails to stem the water, a decision to inflate the emergency seal or to inject polyurethanes will be made.

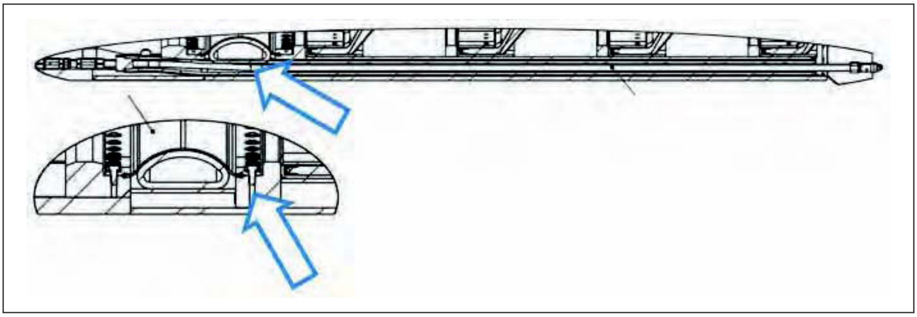
HYPERBARIC INTERVENTION

Airlock Concept

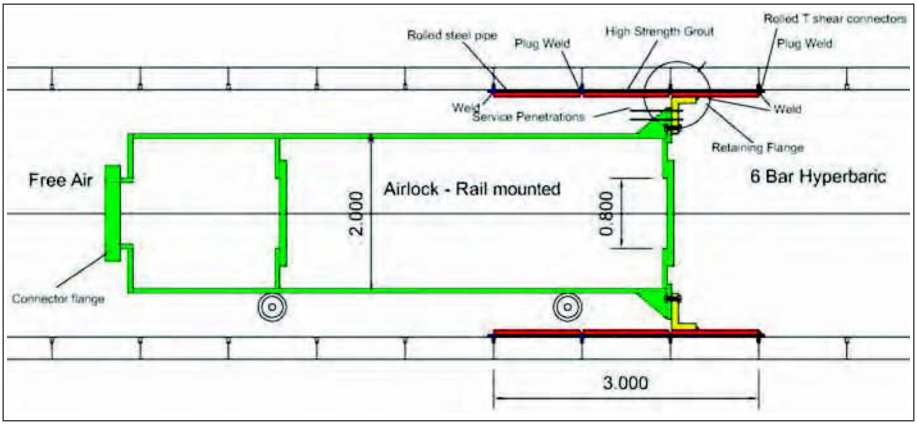
For the purpose of applying compressed air to the tunnel, an in-tunnel airlock will be installed. Setting up the system in this way eliminates compromises associated with TBM mounted airlocks and also improves safety. With a TBM mounted airlock, only around 6 m^3 of air is compressed which could be insufficient in a blow-out. By utilising a tunnel mounted lock the buffer of air is far greater and the logistics of intervention are substantially improved. However, there are risks introduced in applying high pressure air to the tunnel such as fire hazards and detrimental effects to TBM equipment which requires specific procedures to mitigate.

To facilitate installation of the tunnel air lock, the segment design has incorporates a deep caulking groove around the circumferential joint. A series of rolled tees will be provided that will fit into the caulking groove. The tees will be full

Guillotine doors to control discharge along the screw



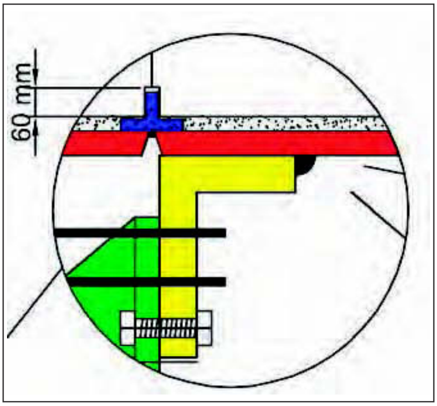
Emergency inflatable seal after tail seal brushes to control the water ingress



Tunnel airlock arrangement

circle and will be cut and welded in situ. Four sets of these tees will be installed at four adjacent ring joints.

A 3 m long steel pipe will be installed in three sections and will be welded together to produce a perfect circular pipe, 2.7 m in diameter. Slots in the pipes will be provided at 1 m centres to allow plug welding of the pipe to the rolled tees. On completion, a rolled flange assembly will be installed inside the pipe. This will be a Computer Numerical Control (CNC) rolled flange with gussets and will be the connection point for the airlock.



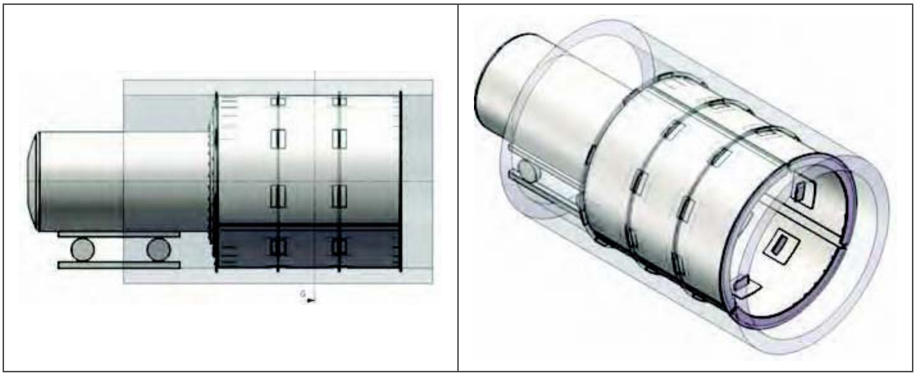
Caulking groove and rolled tee welded to steel pipes

Design Considerations for Tunnel Bulkhead System

The bulkhead has been designed to withstand the maximum anticipated internal pressure developed by the compressed air plant. Specifically, the bulkhead and airlock will pressurized to a maximum of 8.8 bar corresponding to 7 bar maximum working pressure with a temporary overpressure of 1.8 bar.

Design of Rolled Tees

The bulkhead system will be anchored to the existing Port Mann segmental tunnel lining via the circumferential caulking grooves. Several rolled structural pieces will be inserted into the groove to form a ring and subsequently welded to circular steel pipe sections. Since the series of four rolled tees may not be loaded evenly (tees closest to the pressurized hyperbaric condition will experience higher forces than the tees located behind), the design conservatively considered only the first two rolled tees actively resisting the maximum thrust with the remaining two rolled tees installed only for redundancy.



Steel pipe sections and airlock chamber

Design of Welded Steel Pipe

To house the compressed air bulkhead, three steel pipe sections with each section corresponding to a 1.0 m tunnel lining ring will be welded in place. Only one of these pipe sections will actually house the compressed air bulkhead, with the remaining two sections to enable redundancy in the rolled tee anchorage system and also increase the barrier to pressure leakage through the segmental lining.

The steel pipe sections (or cans) are designed according to ASME Pressure Vessel Code design methodology. The grouted annular space between the segmental lining and steel pipe, as well as the presence of the tunnel lining and external soil and groundwater pressure will all further restrain the steel lining.

In addition to internal pressure, the steel pipe sections must have adequate external pressure capacity. An external pressure of 8.8 bar was conservatively selected to ensure that the steel casing on the non-pressurized side of the bulkhead will not be damaged during the initial compressed air proof testing exercise. A calculation was performed to confirm that the steel pipe has ample capacity to withstand the axial thrust from the bulkhead. All of the pipe sections will be rolled to the same outside diameter to match the internal profile of the rolled tee sections. A calculation was also performed to check the adequacy of the pipe thickness.

Design of Steel Bulkhead

A steel bulkhead (flange) with a large central circular opening is required to secure the rail mounted airlock in place. The bulkhead will bolt to the airlock flange and will be welded to the interior of the steel pipe.

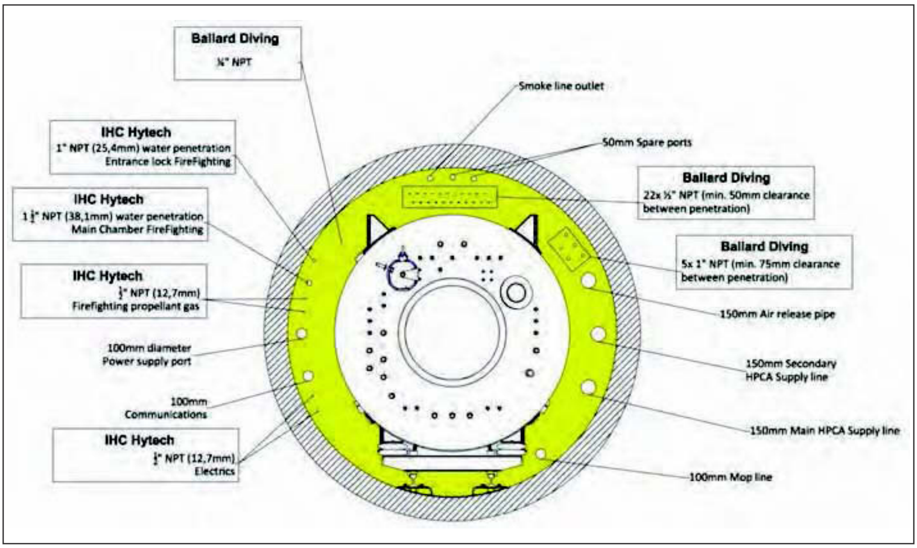
Due to the larger bulkhead area, initially a bulkhead consisting of thick pressure grade steel plate was designed. The bulkhead will essentially act as a ring cantilevering away from the interior of the steel pipe with a concentrated ring load being applied along the circular centerline of the airlock mounting bolts. This results in a large steel plate thickness bulkhead. A second design consisting of a thinner steel plate reinforced bulkhead with steel brackets spaced at 30° centers along the bulkhead perimeter was evaluated which proved to possess adequate capacity.

As an independent check of the capacity of the steel bulkhead, circular steel pipe section, and brackets have been numerically modeled, utilizing 3D structural analysis software (RISA-3D). This numerical model allows for the interaction between the three structural components (bulkhead, pipe, and brackets) to be simulated and the resulting stresses in the steel can be checked to ensure that localized yielding of the members will not occur.

For simplicity, the bulkhead was initially modeled without pipe penetrations. Separate calculations were then performed to assess reinforcement requirements with pipe penetrations through the bulkhead. Based on the ASME Pressure Vessel Code design approach for flat heads with a large central opening and multiple rim openings, the 50 mm head thickness provides adequate additional steel area around the penetrations to satisfy the ASME requirements. This is based on a required head thickness of approximately 1.5 inch (37.1 mm) to span between the brackets, approximately 12.9 mm of the head thickness is surplus material, which serves as penetration reinforcement. Spacing criteria and maximum pipe sizes for the penetrations were also checked and the current penetration layout satisfies the ASME criteria.

Calculation of Quantity of Air Required and Compressed Air Pressure

Compressed air will be used during tunnelling to stabilize the face and to allow for TBM maintenance, such as changing cutters. Traditionally, empirical methods have been employed to estimate the air pressure and air quantity required to stabilize the face. These empirical methods attempt to account for the complex interaction of air, water, and soil mixing within a zone in and around the face of the TBM. Complicating the estimates for compressed air demand is the non-uniformity of the geologic materials, the irregularity in the size and continuity of the annulus between the excavated tunnel and the tunnel liner, and the unpredictability of leakage through the tunnel liner. The calculated air requirements with three different empirical methods are shown in Table 1.



Steel bulkhead penetrations

Table 1.

Empirical Method	Estimated Air Flow (Free Air) (m^3/min)		
	Low	High	Average
Hewett & Johannesson	39.6	79.3	59.5
Kirkland	NA	NA	49.8
Mayo	NA	NA	52.1

As an alternative to the three empirical methods presented above, an appropriate analytical method for estimating compressed air flow in a tunnel heading is Darcy's law for fluid flow through a porous media, which is:

$$Q = K \times I \times A$$

where:

K = permeability of the porous media
i = hydraulic gradient
A = area through which the fluid passes into the porous media

For the Port Mann Tunnel, based on the maximum expected groundwater permeability (Kw) for worst expected ground conditions with permeability of [1×10^{-5} m/s], an expected hydraulic gradient (i) of 1, an area (A) equal to the face area [9 m^2] plus un-grouted and hydraulically-connected annulus area of 52 m^2 and an air permeability equal to 70 times groundwater permeability (Kramer and Semprich, 1989), the estimated air flow is:

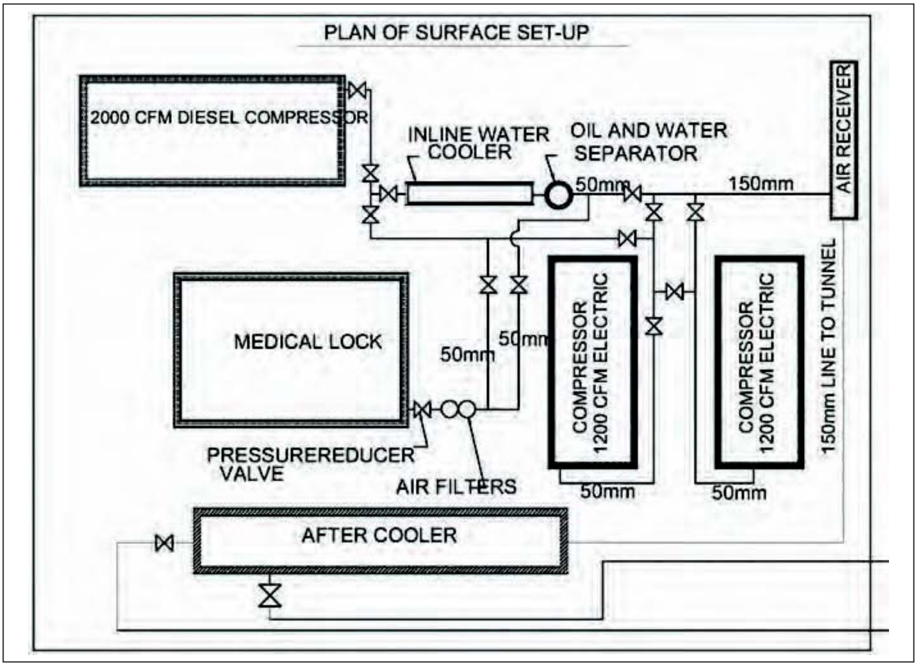
$$Q = 70 \times 1 \times 10^{-5} \times 1 \times (9 + 52) \times 60 \text{ (seconds)} = 2.6 \text{ m}^3\text{/min (compressed air)}$$

Converting the compressed air flow rate to free air flow rate, using the multiplier of 7 for the compressed air pressure to atmospheric air pressure ratio (approximately 700 kPa/100 kPa), yields an estimated air flow of 18.2 m^3/min (free air).

$$Q \text{ Free Air Required} = Q \text{ Comp Air} \times 7 = 18.2 \text{ m}^3\text{/min}$$

EQUIPMENT AND APPURTENANCES

Compressors and Air Receivers



Surface arrangement for hyperbaric equipment

For selection of equipment, diving and hyperbaric specialists were contacted to coordinate capacity of compressors and working pressures. The main components of the compressed air system (air compressor, controller, and air receiver) are detailed below.

Main Compressor

The compressor recommendation provided by specialists given the free air volume requirement would be a 250HP compressor which delivers 35 m^3/min of air to the tunnel. The compressors are oil-injected, air-cooled, direct driven, heavy-duty rotary screw air compressors completely wired and equipped with all interconnecting pipe work and fittings.

Standby Compressor

A portable diesel drive compressor is recommended which delivers 50 m^3/min at 7.4 bar.

Controller

A central controller will be used to optimize system operations which would be wired with the compressors in a LAN (local area network). The central controller optimizes the system operation as well as rotates the operational hours of the compressors and ensures the stand-by machine starts if one of the operating machines has shutdown.

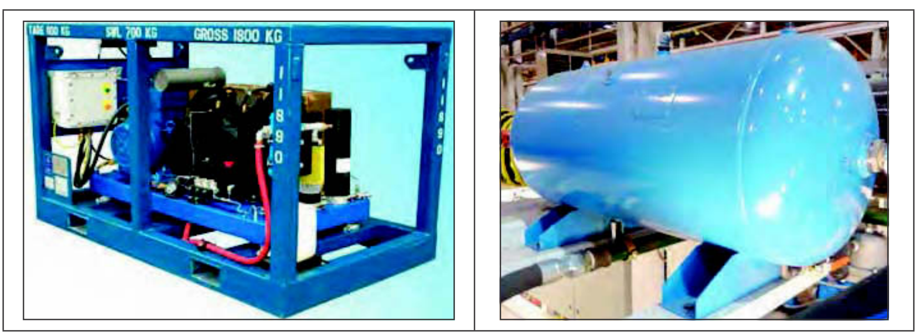
Air Receiver

For the required capacity of the system a 3,000 US Gallon Vertical Air Receiver will be utilized.

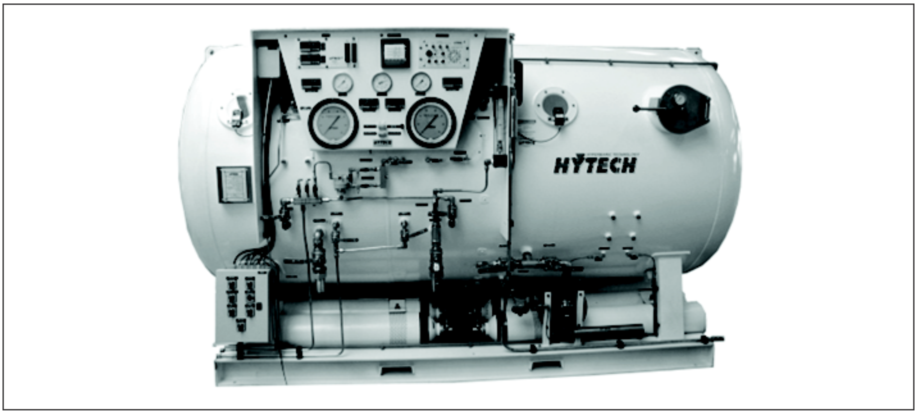
Hyperbaric Chamber (Tunnel Airlock)

A portable double compartment Airlock chamber with a diameter of 2000 mm and an overall length of 4000 mm will be utilized. The unit is manufactured in the Netherlands (By Hytec) and is built in accordance with Canadian standards CSA Z275–Class A hyperbaric chamber. This chamber will be brought in to the tunnel on rail and will connect to airlock bulkhead to allow preparation and passage of the diving team from the atmospheric side to compressed air side of tunnel and vice versa.

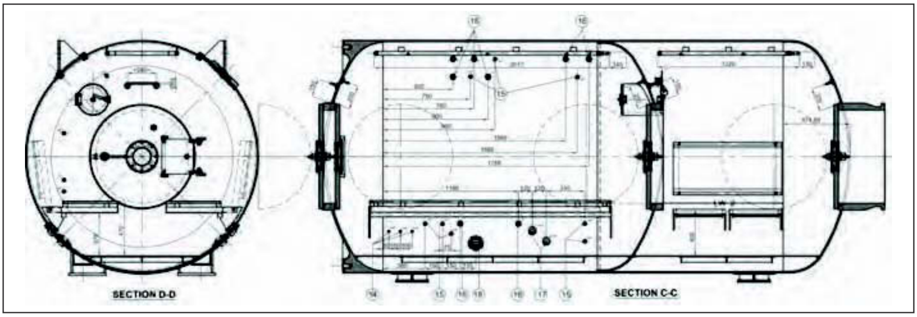
This chamber has an entrance compartment to facilitate compression or decompression of individuals while other members of the team stay in the main section under compressed air. This section will also facilitate the passage of tools to the working team in the compressed side of tunnel. The entrance chamber compartment is outfitted with seats on both sides to provide seating for 4 persons.



Typical air compressor and air receiver



Compressed air chamber delivered to site (Courtesy of Hytech)

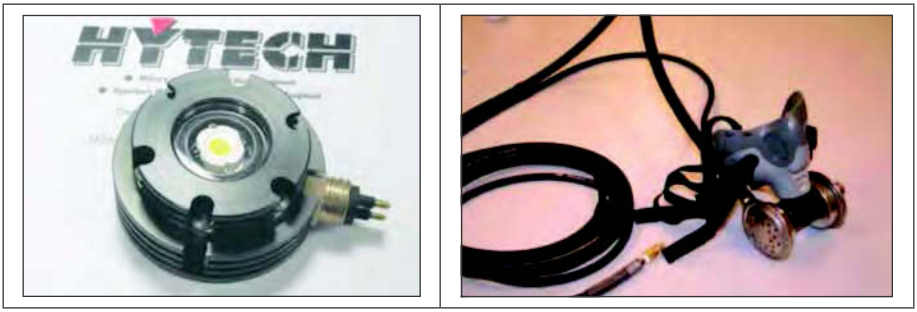


Tunnel hyperbaric chamber schematics

Inside compressed air chamber

The chamber is outfitted with all the necessary equipment and components necessary to pressurize and control the chamber system, such as:

- Built-in breathing masks with overboard dump system with necessary controls to feed oxygen and mixed gas to the breathing masks in both compartments
- O_2, CO_2 and Helium analyser (can be connected with main chamber or entrance chamber)
- Lighting system in both compartments



Internal LED lighting device and oxygen breathing mask

Shuttle car by Hytech

- Drain valves in both compartments
- Bunks in both compartments with fire retardant mattresses, blankets, etc.
- Water deluge fire suppression system / sprinkler system for both compartments independently
- Portable firefighting extinguishers for each chamber compartment
- Heating and cooling equipment

Transfer Shuttle

A smaller compressed air chamber will be built for transportation of crew under compressed air between the surface and tunnel. This shuttle could be connected to the main airlock chamber in the tunnel or medical lock on the surface to ensure safe and rapid transfer of compressed air workers for saturation diving or medical reasons.

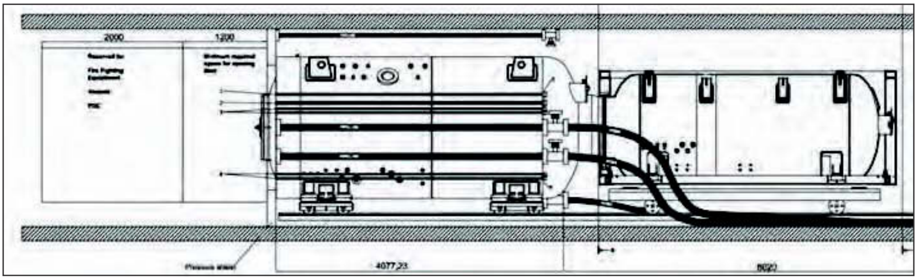
This shuttle is designed with a diameter of 1,500 mm with circular entrance doors and assembled in compliance with the ASME guidelines. The layout of the shuttle allows for the treatment of 4 workers in sitting position. This unit is made from carbon steel and weighs around 6,000 kgs. The transfer shuttle can also be used for 2 patients laying down on the bunks and an attendant in sitting position.

Surface Habitation/Medical Lock Unit

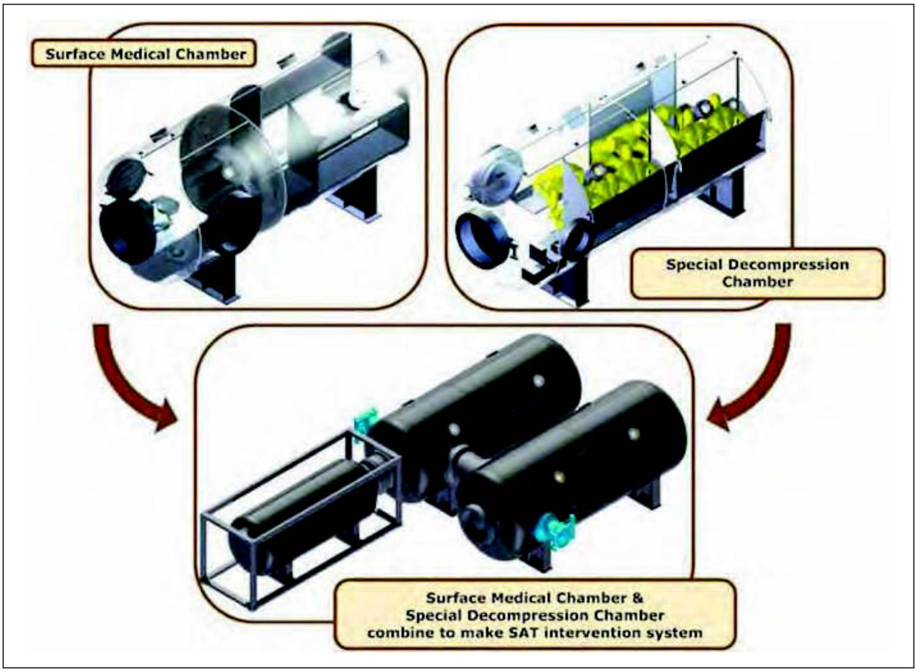
This unit will be sized for 15 people and will be compatible with the shuttle car. The habitation unit will be built in accordance with Canadian standards CSA Z275–Class A hyperbaric chamber and will be used for saturation diving or in case of emergency, to facilitate the treatment of compressed air workers by appointed physicians specialized in compressed air interventions.

TBM LAUNCH AND RECEPTION

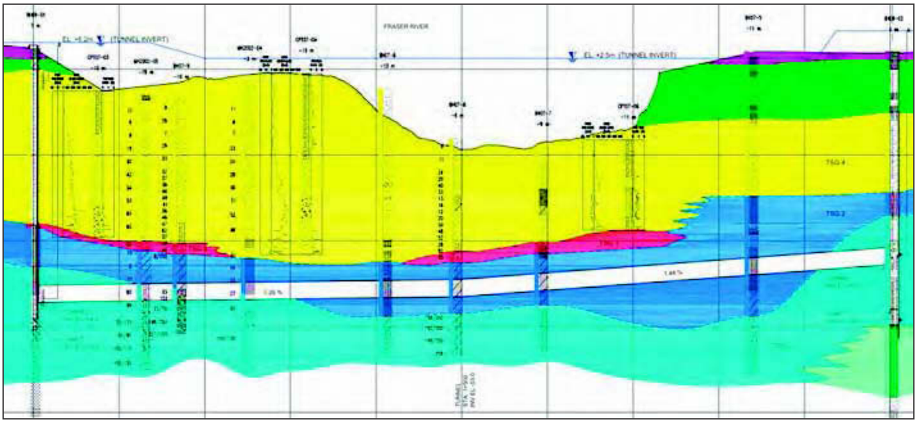
Due to high hydrostatic pressures at the bottom of launch and reception shafts, the TBM must break-in and out of the ground through specialized procedures. The intent of these methods are to balance the 6 bar pressure outside the shaft slurry walls to ensure the integrity of the shaft and tunnel are not be compromised.



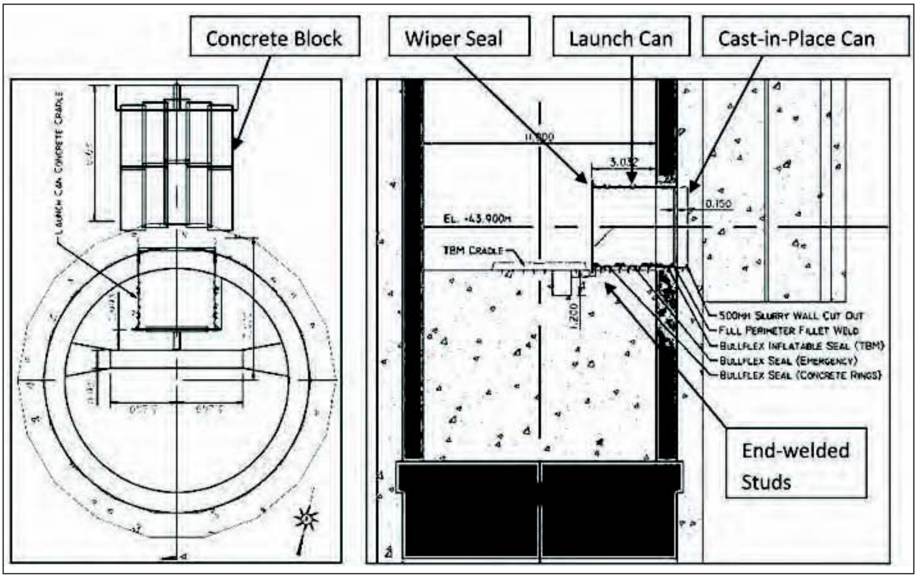
Shuttle car connected to main airlock chamber



Surface habitation/medical lock unit



Tunnel and shafts geology

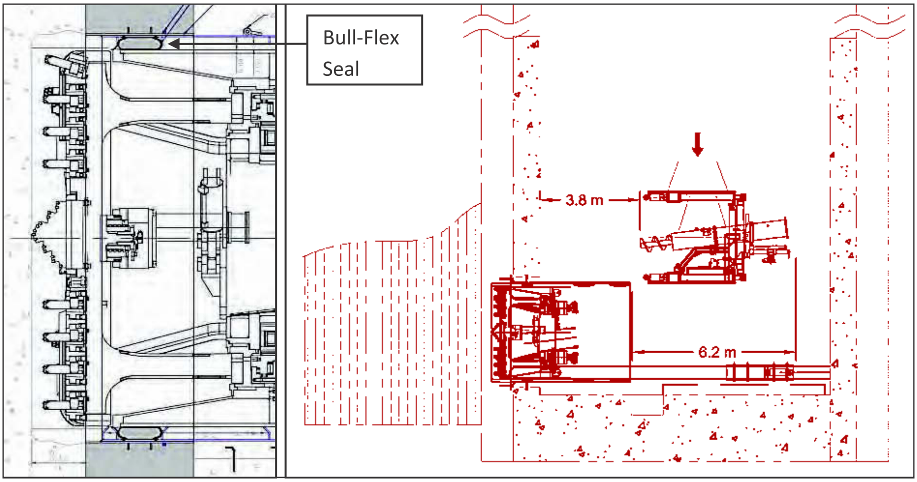


TBM launch shaft and tunnel eye

Hydrogeological conditions at the project area are controlled by combined influences of local topography, a complex geological setting, and the Fraser River tidal cycles. The water table below the upland areas is elevated approximately 40 m to 50 m above the river level and ground water generally flows toward the River. Artesian piezometric levels measured at the south shaft are on the order of 5.1 m above ground surface.

TBM Launch at South Shaft

The TBM will be launched from the south shaft which is located about 220 m south of the south bank of the Fraser River. The existing ground surface in the vicinity of the shaft is relatively level at about elevation +3.7.



TBM maneuvered into the launch can

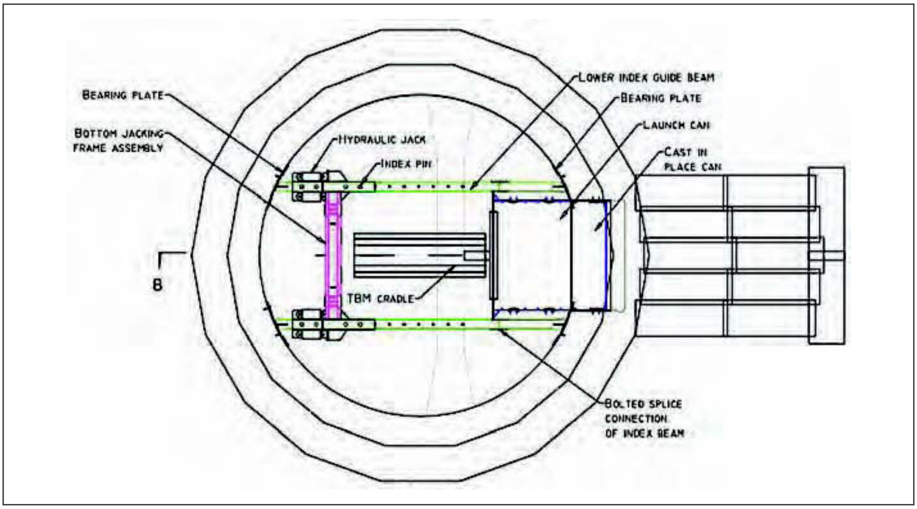
To successfully launch the TBM into the challenging ground conditions behind the slurry walls, several provisions have been made. Firstly, a 7×6 m ground replacement zone (Concrete block) has been constructed at the tunnel horizon using slurry wall installation techniques to provide protection for the first few meters of tunnel excavation. Further, a launch can has been designed to encapsulate the TBM and allow for manual pressurization of the TBM working chamber up to 6 bar to balance the hydrostatic pressures expected behind the slurry wall. This pressure will be provided to the TBM working chamber by introducing bentonite to a "launch can" sealed using Bullflex system.

Due to structural constraints, the installation of the launch can requires a two stages process. In the first stage, a slightly oversized can will be cast into the reinforced inner shaft wall while the internal shaft lining is being poured. Next, the 'launch can' will be lowered into the shaft and set inside the cast in place can. The launch can contains three Bullflex inflatable seals: one for sealing against the TBM, one to act as an emergency backup, and one to seal against the concrete rings. These seals will be inflated with grout and lubricated if necessary prior to insertion of the TBM into the launch can. The can will also be fitted with a wiper seal and keeper plates should inflation of Bullflex seals fail. Lifting lugs and internal bracing have been designed to facilitate the lifting while maintaining small tolerances for circularity.

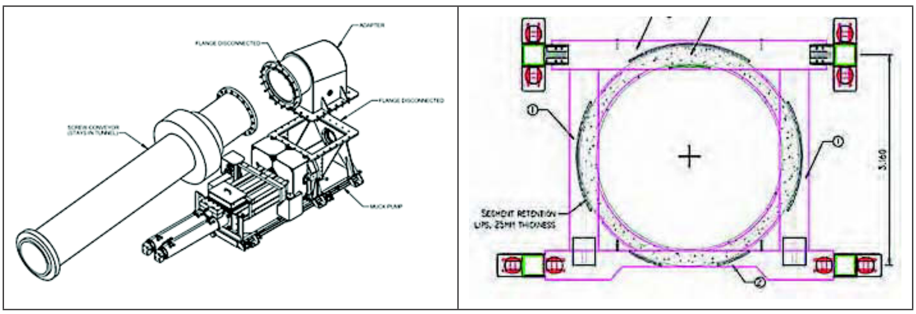
Inside the can, the TBM will be supported by cast-in-place concrete panels poured into steel troughs. The lower half of the can will be furnished with end-welded studs to provide increased interface shear resistance between the can and its support concrete. Hydrophilic gaskets and polyurethane foam will be used around and in between the cans to ensure water tightness of the system.

The TBM is to be lowered into the shaft and maneuvered into the launch can as it is being assembled. Prior to launching the TBM, the forward most Bullflex seal will be inflated with grout creating a seal between the launch can and TBM.

The TBM will require three pre-cast concrete tunnel liner rings in the tail shield while mining through the shaft wall and break-out block until the tail shield enters the launch can. During this launch process the TBM is expected to be pushed forward through the rings in the tail shield by hydraulic propulsion jacks located on the jacking frame outside the launch can.



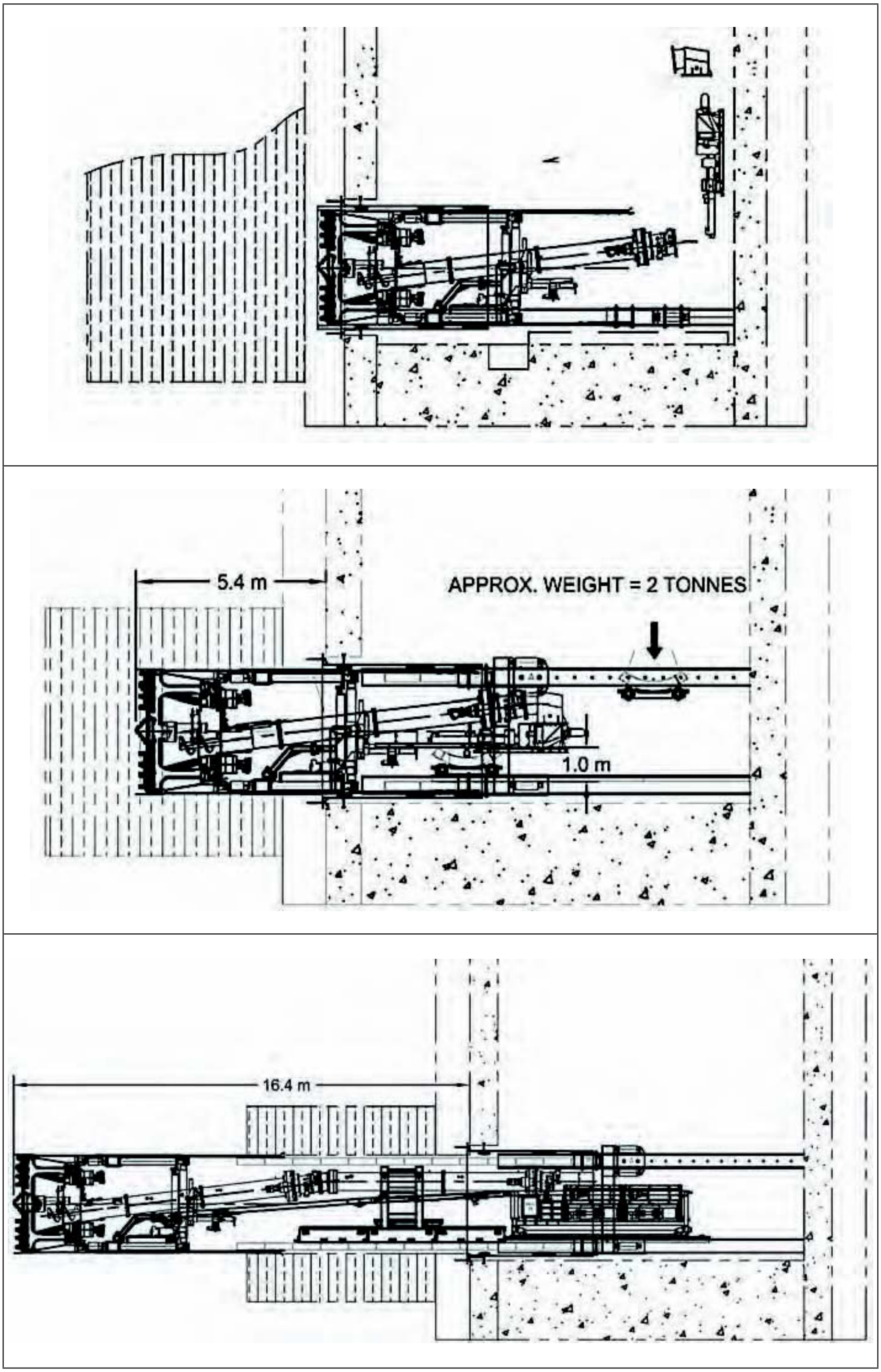
TBM launch frame



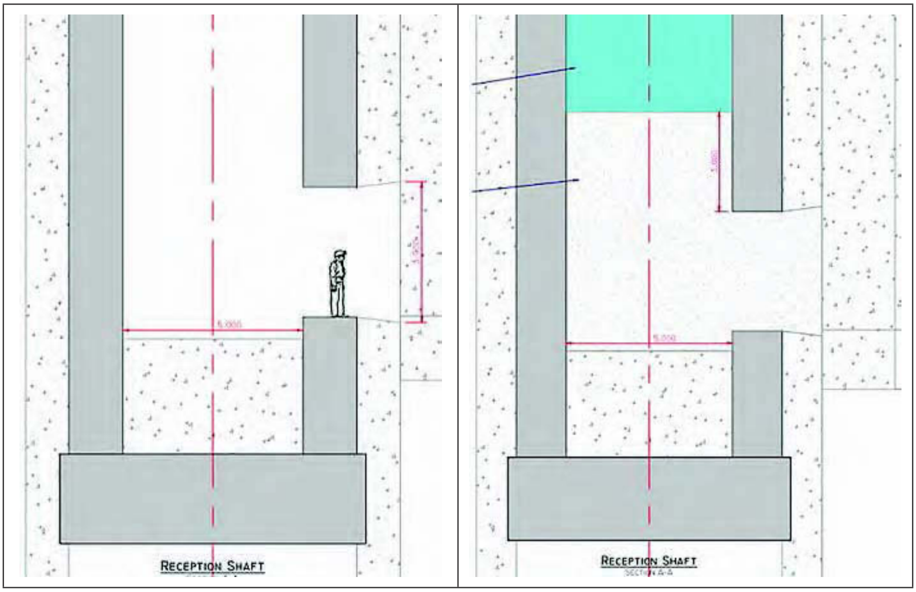
Muck pump at launch phase (left), and launch frame segment retention lips (right)

The jacking frame consists of lower and upper index guide beams and hydraulic jacks. Due to space restrictions in shaft, the index beams are formed from two units. Both units have one bearing plate welded at the end to be anchored to shaft wall. The lower jacking frame assembly will be utilized prior to TBM excavation to install the forward shell, stationary shell and other necessary components. At this stage 3 precast rings will be lowered into the shaft and will be placed one by one within the tail can. A muck pump will be installed before assembly of upper index beams and hydraulic jack to facilitate the extraction of material from the partially assembled TBM and screw conveyor.

The jacking frame will remain in constant contact with the rearmost ring at all times and is considered to have reached its limit of travel once it reaches the launch can opening. At this point, the tail shield will have entered the launch can and the TBM is expected to begin pushing forward using its own hydraulic thrust cylinders. The rearmost Bullflex seal will be inflated against the tunnel liners once the tail shield has cleared the seal contact area. A total of 54 rings are required to be pushed before the jacking frame can be released, however to fully bury the TBM and for complete installation of trailing gear, a total of 72 rings are required.



Launch sequence



TBM reception shaft

TBM Reception at North Shaft

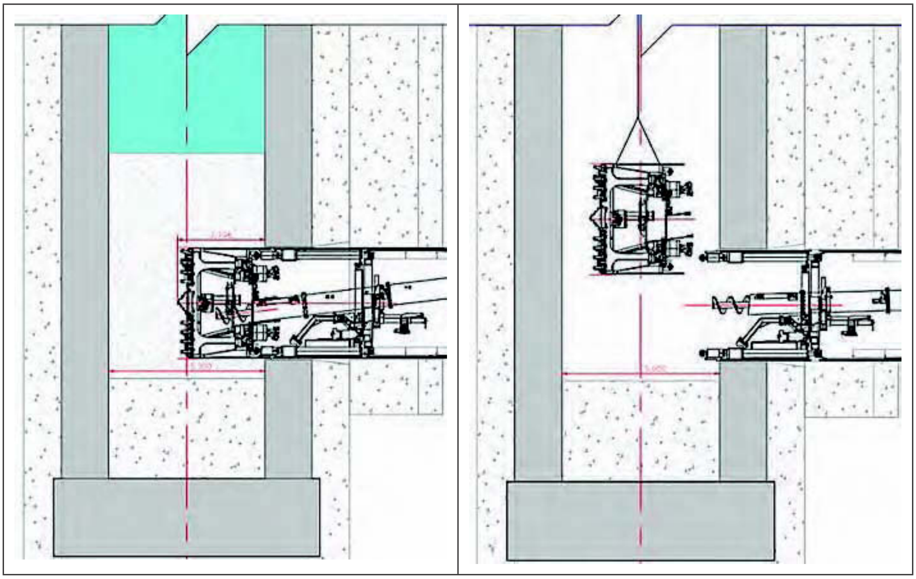
The TBM will be received at the north shaft located about 35 m north of the north bank of the Fraser River. Initial support of the shaft was accomplished with slurry diaphragm walls with a nominal internal diameter of 8 m and a depth to bottom of tremie slab of 65.7 m. A 5.0 m ID, reinforced, cast-in-place concrete structural shaft, will be constructed within the slurry wall shaft. Similar to the south shaft, a 3×6 m ground replacement zone (break-in block) was constructed at the tunnel horizon using slurry wall installation techniques.

TBM reception involves casting a hole through the permanent lining with no encapsulating can around TBM. The shape of this through-wall hole will be conical to produce an ID of 3,600 mm at the slurry wall and cast in place interface tapering out to 3900 mm at the rear of the slurry wall panel. This may be removed by hand or pressure water jets following a series of stitch drilling. Probe drilling will be done through the portal to confirm the integrity of the concrete block on the other side of slurry wall prior to concrete removal.

After the slurry wall has been broken out and the debris removed, the shaft will be backfilled to 3 m above tunnel. Water will then be added to balance the exterior hydrostatic pressure. At breakthrough the TBM will be driven through the eye. The cone through the slurry wall section will act as a guidance cone and will centralise the TBM through the eye.

The TBM will bore through the break-in block and into the backfilled shaft. Once the TBM reaches stopping position, the tail shield and stationary shell will be grouted in place in the breakthrough block and shaft wall.

Once the TBM tail shield is grouted in place, the EPB pressure within the head can be reduced and the effectiveness of the seals and grout can be determined. Grouting will be done via holes drilled through the TBM shell and its effectiveness will be gauged by reducing face pressure and observing the recovery to confirm the integrity of the seal created by the grout. Additional grouting might be needed to adequately seal the area.



TBM disassembly

Once an adequate seal has been confirmed, the shaft will be drained, the backfill removed and the TBM cleaned. The TBM cutter head and forward shell will be disconnected from the stationary shell and pushed forward into the center of the shaft. Lift Lugs will be welded to the forward shell and both cutter head and forward shell will be lifted from the shaft. The inner stationary shell and TBM components (screw conveyor, erector, etc.) will be removed from the stationary shell and lifted out of the reception shaft to complete the operation.

CONCLUSION

This paper is intended to outline some of the practical engineering solutions developed to tunnel through complex geology at exceptional depths and high hydrostatic pressure.

Innovative approaches in customization of TBM and tunnelling systems to suit the aforementioned conditions including the design of the TBM, emergency systems, procedures for high pressure compressed air interventions, and specialised launch and removal procedures are just some of the details that have been analysed. It is expected that the diligence paid to these technical aspects will pay dividends with respect to providing Metro Vancouver with a successful project.

UNIVERSITY LINK LIGHT RAIL TBM TUNNEL UWS TO CHS CONTRACT U220: CASE HISTORY

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Michael Krulc ▪ Traylor Bros., Inc.

Richard McLane ▪ Traylor Bros., Inc.

Jeffrey Brandt ▪ Traylor Bros., Inc.

ABSTRACT

This paper reviews the underground work performed on Sound Transit's University Link Light Rail TBM Tunnel UWS to CHS Contract U220. The tunnel system consists of twin 11,400 feet long segmentally lined tunnels with excavated diameters of 21.5 feet using earth pressure balance TBMs. The twin tunnels are connected by sixteen cross passages excavated using the sequential excavation method. Topics included in this paper include TBM selection and design, ground conditioning, preparations for hyperbaric interventions above 4.5 bar, review of conveyor muck scale data, segmental lining design and challenges overcome during TBM mining. Additionally, the challenges and advantages of performing SEM excavation on multiple cross passages from within an active TBM tunnel are discussed.

INTRODUCTION

University Link Extension is a 3.15 mile extension of the existing Central Puget Sound Regional Transit Authority (Sound Transit) light rail system that runs in twin-bored tunnels from Downtown Seattle north to the University of Washington, with stations at Capitol Hill and on the University of Washington campus near Husky Stadium which is expected to add 70,000 daily boardings to the system by 2030. The complete alignment of the University Link program is shown in Figure 1.

Based on experience gained during construction of the Central Link segment, Sound Transit elected to separate the University Link extension work into eight discipline specific contracts, with two contracts performing the vast majority of the civil work and all of the tunneling work. This paper focuses on the U220 TBM Tunnels UWS to CHS Contract which included UWS (University of Washington Station) site preparation, UWS station shoring, crossover box excavation, 11,400 feet of twin light rail tunnels from UWS to Capitol Hill Station (CHS), excavation and final lining of sixteen (16) cross passages, permanent electrical, permanent mechanical and tunnel concreting.

The same joint venture of Traylor Frontier-Kemper (TFK JV) that successfully completed LA MTA's Gold Line Eastside Extension was awarded the contract on June 5, 2009 and granted Partial Notice to Proceed on the same day. The project garnered two bids, both of which were well below the published Engineer's Estimate:

Traylor/Frontier-Kemper	$309,174,277
Jay Dee/Collucio/Michels	$360,775,000
Engineer's Estimate	$395,354,000

Construction management services for the University Link program were provided by the Seattle Tunnel and Rail Team (START) a joint venture of CH2M Hill and Jacobs Engineering Group with additional support provided by internal Sound Transit staff and the design team, Northlink Transit Partners (NTP), itself a joint venture of Jacobs Associates, HNTB and Earth Tech.

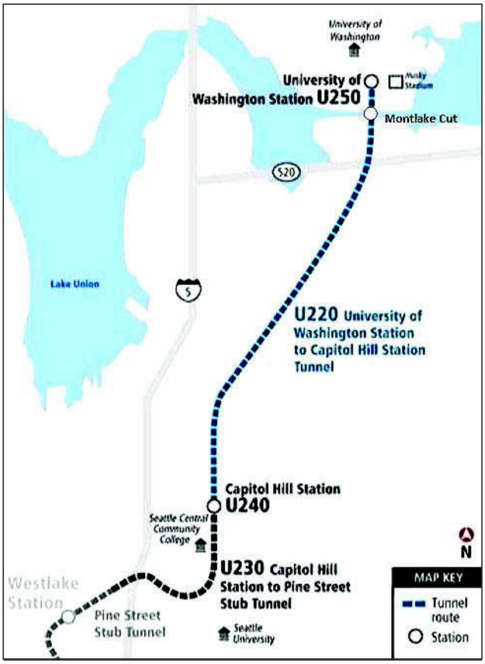

Figure 1. U220 University Link alignment

GEOLOGY

The U220 tunnel alignment lies completely in soft-ground deposits below the groundwater table. The project area has been subject to a number of glaciations, with ice thicknesses at times exceeding 3000 ft, and due to these glaciations and corresponding thick ice sheets, the basic soil strata at tunnel level is overconsolidated.

Soil Groups

The Geotechnical Baseline Report (GBR) grouped the geologic units expected to be encountered in the tunnel excavations into Soil Groups in an effort to better describe and characterize the wide variability present within each of the geologic units. The Soil Groups were established using geologic units expected to exhibit similar engineering characteristics. A brief description of the Soil Groups expected during tunneling are listed below:

- Blue Soil Group—fine grained plastic clays and silts, generally firm, intensely fractured in places,contains interbeds of fine sand and silt in thicknesses up to 1 foot.
- Turquoise Soil Group—nonplastic silt and fine sand, generally firm but can deteriorate if disturbed.
- Yellow Soil Group—variable silty sand to clean sand, overconsolidated, cohesionless though lenses of cohesive material can be present.
- Red Soil Group—sandy gravel to silty sandy gravel, overconsolidated, cohesionless though lenses of cohesive material can be present.

The GBR reports ranges of the Soil Groups expected in the tunnel face, with the breakdown expected to be about 62% full face Blue Soil Group, 13% Turquoise Soil Group, 8% Yellow Soil Group, and 17% some combination of the four groups (Red Soil Group is never expected to be full face, but is expected to be present in the tunnel alignment).

From a TBM operating perspective, the geology implied by the GBR is that long stretches of Blue Soil Group will be encountered, but will be interrupted by several “buried valleys” that will be made up of the other Soil Groups.

Groundwater

Water head above the tunnel invert ranges from a minimum of 64 ft (1.9 bar) near the Lake Washington Ship Canal to a maximum of 210 ft (6.2 bar) at Crosspassage 9 under Volunteer Park. The water head along the alignment is commonly in the range of 110 ft to 160 ft (3.5 bar to 5 bar), which is generally quite high compared to other soft ground tunnels that have been completed in the United States to date.

Other Significant Characteristics

Boulders

The GBR expected that boulders would be encountered during tunnel excavation and implied that they are most likely to be found at the interfaces of the “buried valleys” described above. The GBR states that 12 boulders of greater than three feet will be encountered. Boulders did not turn out to be a significant issue during tunnel excavation.

Soil pH

The GBR expected that the natural pH of the soil excavated during tunneling would fall in the range of 5.5 to 9.8, and that 20% of the soil would be greater than 8.5. Elevated pH levels affect disposal options and cost. It turned out that approximately 90% of the tunnel muck exhibited high pH levels and had to be disposed of using special procedures.

Stickiness

The GBR expected 77% of the Blue Soil Group to exhibit moderate to high stickiness potential based on plasticity characteristics. Sticky muck can create significant problems with Earth Pressure Balance TBMs by clogging cutterhead openings, plugging screw conveyors, affecting control of face pressures, and clogging conveyor transfer points.

Abrasion

The GBR utilized the Soil Abrasion Test (SAT) method to describe the abrasivity of the Soil Groups. The Blue Soil Group is expected to exhibit an SAT between 0 and 4 (extremely low to very low abrasivity), the Turquoise Soil Group is expected to exhibit an SAT between 5 and 7 (low abrasivity), and the Yellow Soil Group is expected to exhibit an SAT between 12 and 24 (medium abrasivity). When considered in conjunction with the expected occurrence of the various Soil Groups, it can be seen that a majority of the tunnel alignment was expected to have low to extremely low abrasivity. Even so, significant efforts were undertaken to armor the cutterheads and select the appropriate cutting tools for the TBMs based on experiences of other contractors in the Seattle area (i.e., VPFK at Brightwater). Abrasion did not turn out to be a significant issue during tunnel excavation, and in fact, there was still paint remaining on the cutterheads after the completion of tunneling.

INSTRUMENTATION

The U220 Contract tunnel alignment is located in a densely populated urban area that requires tunnel excavation under more than two hundred residential homes, commercial buildings, utilities, historic structures, and a subaqueous crossing with low cover. As a result, a significant instrumentation program was specified in the contract documents.

The program consisted of structure settlement points on 82 different structures along the tunnel alignment, multiple point borehole extensometers (MPBX) at each of the crosspassage locations, inclinometers around the shored excavations and slurry wall at the University of Washington site and near the Lake Washington Ship Canal crossing, utility settlement points on the major utilities that pass over or along the tunnel, vibrating wire piezometers (VWP) at various points along the alignment, and strain gauges on the steel slurry wall bracing.

In all, the project installed and monitored 352 structure settlement points, 40 MPBXs, 18 inclinometers, 55 utility settlement points, 33 Vibrating Wire Piezometers, 12 observation wells, 26 near surface settlement points and 19 strain gauges.

GROUND IMPROVEMENT

The ground immediately north of the northern slurry wall that forms the UW Station box was treated with jet grout under the U220 contract in anticipation of receiving two TBMs as part of the Northgate Link Extension program. The treated zone extended 50 feet north of the slurry wall, included a minimum treatment width of ten feet beyond the outside of the future twin tunnels, and was performed over a 50 foot vertical section centered on the future tunnel centerline. Hayward Baker performed the jet grout treatment using twelve foot diameter columns spaced at roughly 9.5 foot centers. The jet grout work was performed in an area adjacent to the University of Washington's football stadium (Husky Stadium) known as the West Plaza, which is one of the main access points for the stadium. The working grade from which the station box slurry walls were excavated was created by a roughly 20 foot deep excavation adjacent to the West Plaza and shored with a soldier pile and lagging wall with a single row of tiebacks. To avoid conflicts between jet grout treatment, shoring installation, tieback drilling and slurry wall construction the jet grout treatment was separated into two distinct phases.

During the first phase of jet grout, all columns were installed except for the row (Row A) immediately adjacent to the station box north slurry wall with the second mobilization installing this final row of jet grout columns. Between the two mobilizations the station box area was excavated to the slurry wall working grade, lagging and tiebacks were installed and the north slurry wall was constructed. Further complicating matters was the contract requirement to vacate the West Plaza 72 hours prior to any football game and prior to the annual commencement ceremony held in June of each year.

The U230 contract, executed by JayDee Coluccio Michaels (JCM) Joint Venture, included jet grout treatment of the sand and silt just north of the Capitol Hill Station (CHS) excavation where the U220 TBMs were intended to hole through. This jet grout block extended 40 feet north of the CHS excavation which carried the treatment zone under John Street which is a congested and heavily travelled urban thoroughfare. In order to minimize the impact on local traffic, a number of the jet grout columns were installed at a significant angle from the vertical with mixed results. A remedial grouting program was undertaken from within the station excavation to ensure overall integrity of the jet grout block.

The 40 foot length of the jet grout block was worrisome to TFK as the U220 TBM shields were 37 feet long, meaning that at hole through less than a single tunnel ring would be grouted into the jet grout block whereas a minimum of two is preferred. Despite our concerns, the jet grout block in combination with the deep dewatering system installed around the station excavation performed well and both machines holed through without incident.

STATION EXCAVATION

Slurry Wall

The University of Washington Station is an underground station constructed under the parking lot of the University's football stadium (Husky Stadium) that relied upon nearly 230,000 square feet of reinforced slurry diaphragm walls for both temporary support of excavation and as the permanent structural station walls. The overall station box is

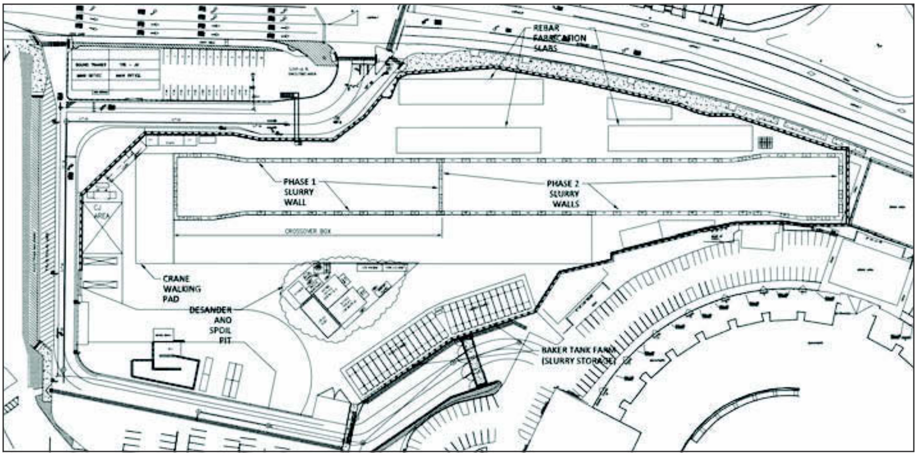

Figure 2. Slurry wall layout plan

approximately 700 feet long, with a width varying from 66 feet in the central portion to 80 feet at each end and depth varying from 120 feet at the south end to 163 feet at the north. Geology consisted of heavily overconsolidated clays overlain by glacial till with some sandier material at the north end of the site.

Base contract design of the slurry diaphragm walls called for a four foot thick wall with double layers of vertical reinforcing steel on both the inside and outside faces combined with evenly distributed horizontal stirrups and seismic ties between the inner and outer face cages. The overall reinforcing steel density resulting from the base design was approximately 65 pounds per square foot of wall. Schematic layout of the slurry wall cages in the contract drawings called for 23 foot wide primary panels and nine foot wide secondary panels with a single reinforcing steel cage in each panel (Figure 2).

The joint venture of Condon Johnson/Nicholson (CJN) that was subcontracted to excavate and install the slurry diaphragm walls quickly recognized that a primary panel with such a heavy rebar pattern in a single cage would create significant constructability problems and they set about changing things. CJN made the following three primary changes to circumvent the reinforcing steel issues:

1. Value Engineering proposal that revised the primary reinforcing steel from Grade 60 to Grade 75 which increased the spacing between the vertical reinforcing steel from six to seven and a half inches. This reduced the total amount of steel required and resulted in a significant credit for Sound Transit.
2. Split the reinforcing steel cages in the primary panels into two separate cages that were lighter and thus easier to install with the lift cranes available. This change added a minor amount of steel back into the wall but not enough to offset the VE work.
3. Introduced a splice in cages for panels that were more than 140 feet deep which maximized the available lift cranes.

CJN started slurry wall work in June 2010 with excavation for guide walls in the southern half of the station box. In lieu of traditional formwork for the guide walls, CJN elected to install the required reinforcing steel then place wet mix shotcrete followed by a quick pass by cement finishers to complete the wall. The end product was of very high quality and was performed well under budget.

CJN mobilized a significant equipment fleet to complete slurry wall construction ahead of the contract milestones. Their fleet of major equipment included the following:

- Two hydro fraises (two were required by provisions in the prime contract)
- One crane mounted hydraulic clamshell
- Two desanding units
- One 30 ton off road haul truck
- One 30 ton excavator
- Three support cranes
- Fifteen baker tanks for slurry storage.

CJN also required three reinforcing steel cage assembly pads each measuring 30 ft × 180 ft and a 400 cubic yard spoil pit. Once everything was on site and working, there was little room to move around and the potential excavation sequence was limited.

Completing slurry walls for the south half of the station (known as the crossover box) was the critical path as this is the area TFK had to excavate in order to assemble and launch the TBMs. CJN was allowed a total of nine weeks to complete this work and they did so on time. Further complicating matters was the presence of a soldier pile lagging wall offset three feet from the back side of the slurry wall to allow future demolition of the top fourteen feet of the cross over box slurry wall.

Construction of a typical primary panel followed this basic script:

1. Excavate top 40 feet with hydraulic clamshell. Tram spoil to pit with off road truck.
2. Excavate balance of panel with hydro fraise in three bites (left, right, center), pumping spoil laden slurry to the desanding plant.
3. Cleanout bottom of excavation and confirm geometry via Koden inspection.
4. Change out excavation slurry with clean "concreting" slurry.
5. Hoist and install reinforcing steel cages. This was always done at night.
6. Place tremie concrete and remove concreting slurry.

Secondary panel excavation followed a similar pattern except that it was performed between two previously excavated primary panels and required the removal of seven inches of cured concrete from each of these panels. Subsequent concrete placement against this milled face created the water tight barrier required for the permanent structure.

Progress throughout slurry wall construction was excellent with the exception of two issues which hampered production and caused overall difficulty. The first issue related to the contract requirement to mobilize a minimum of two hydrofraises and appropriate separation plant facilities to support their operation. Having been forced to mobilize this equipment, CJN was left with no choice but to utilize it given the limited area available on the site. This resulted in CJN relying almost exclusively upon hydrofraise excavation to complete the work which placed a heavy burden on the separation plants. The separation plants struggled to keep up with the excavation rate and were generally inefficient at removing the clay spoils from the bentonite in the slurry. The end result was extremely wet muck that quickly filled the available muck bin and often delayed additional excavation. In addition to creating a mess, the wet muck had a negative impact on both spoil disposal and bentonite costs.

The second major problem encountered was one that the slurry wall community had not seen before so it became a learning experience for all of us. Several of the early secondary panel excavations encountered horizontal and vertical reinforcing steel in the adjacent primary panels. In order for the secondary panel excavation to encounter rebar from the primary panel, the primary panel cage had to move horizontally at least one foot toward the end of the primary panel excavation. At first, this

Figure 3. Fully mobilized site in action

Figure 4. Typical reinforcing steel cage installation

was hard to believe as the gap between the cages and the ends of the excavation was maintained with a series of corrugated plastic pipe spacers strapped to the cage at even intervals. The spacer detail had worked on every slurry wall project to date but in certain instances on this job it failed (Figures 3 and 4).

Excavating through rebar-laden concrete dramatically slowed secondary panel excavation and required the installation of additional reinforcing steel in the secondary panel to replace any reinforcing steel that might have been damaged in the primary panel. As could be expected, hydrofraise tool wear was very high as well, with one panel consuming more than 800 teeth—more than four times the average secondary panel.

Once the problem was discovered, CJN began an investigatory process to determine the cause and was ultimately able to prevent future occurrences by significantly increasing the robustness of the spacer design.

Bracing Redesign

The base contract design called for up to four levels of temporary cross lot bracing in the crossover box to support the slurry walls during excavation of the shaft and subsequent construction of the station. Two bracing levels also contained permanent structural elements that would be incorporated into the station floor system. Constructability of the original bracing system design was a significant concern for multiple reasons:

- Specified shapes were not available
- Significant field welding was required

- Limited ability to adjust bracing for slurry wall installation tolerance without field fabrication

TFK recognized the issues with the contract bracing design during the bid phase and set out to improve constructibility by simply reverse engineering the existing design so that it relied upon more readily available sections (wide flange and pipe) and converting as many field welded connections to bolted connections as possible. With the engineering work completed, the estimators did their work and then on bid day forgot to include the purchase of about half of the required steel, which left more than a million dollar hole in the budget. Oops.

The first order of business after being read as low bidder was to perform a complete redesign of the bracing system to save as much steel as possible and try to get back under budget. With the help of two design consultants, the joint venture redesigned all facets of the temporary bracing system while leaving details of the permanent components untouched. Five primary improvements were made during the redesign process:

1. Commercially available rolled sections were utilized
2. Total steel weight was reduced by 1,500 tons
3. More than 2,000 pounds of field welding was eliminated
4. Torsion restraint requirement was eliminated
5. Need for all field fabrication was eliminated

Bracing installation was performed in conjunction with excavation of the shaft and proceeded generally according to plan, with only minor difficulties along the way. Shaft excavation was accomplished using a combination of hydraulic excavators, tracked loader and crane-hoisted muck boxes in a pre-defined sequence that provided access for installation of one bracing level at a time. Smaller excavators were used to provide initial access under a newly installed section of bracing and feed spoil to a track loader that was used to tram material to a centrally located load out location. The track loader would then fill the 18 cubic yard muck box which was hoisted by one of the two lift cranes supporting the bracing and excavation work. Working around the clock, six days per week, excavation was completed and all four levels of bracing were installed between early November 2010 and the middle of February 2011 with a typical 24 hour period seeing 1,500 cubic yards of material leave the site. Bracing members were hoisted into place by one of the lift cranes with the track loader and excavators providing assistance as needed to line things up.

The only significant difficulty arose when several of the attachment plates embedded in the slurry wall concrete shifted horizontally during the concrete placement. This created a fit up problem where the packing members welded to the back side of the wales would no longer line up with the plates embedded in the slurry wall. The typical solution was to as-built the embedded plates and have the packing members on the wales relocated in the fabrication shop prior to delivery.

TUNNEL

Earth Pressure Balance Tunnel Boring Machines

TBM Selection

The U220 contract had specified two new Earth Pressure Balance TBMs to excavate the running tunnels on the project. TFK reviewed four proposals from the following suppliers for the project:

- Robbins, Solon, Ohio—Two new machines
- Lovat, Toronto, Canada—Two new machines
- Herrenknecht AG, Germany—Two new machines

- Herrenknecht AG, Germany—Two remanufactured machines

TFK had used two Herrenknecht EPB TBMs on a previous project on the Los Angeles County MTA Goldline Eastside Extension (MGLEE). Conveniently, the internal diameter of the tunnels for the U220 project was identical to that of the Los Angeles project and although the pressures in Los Angeles were lower, there was potential for a remanufacture.

After a detailed analysis, the best value for the machine supply was the remanufacture of the two TBMs from the MGLEE project. TFK, along with Herrenknecht, held several meetings with the client and the design team to negotiate the use of remanufactured machines versus the specified new machines. Because the pressures on the U220 project were expected to be much higher than those on the MGLEE project, new shields would have to be used, but several components from the MGLEE machine could be utilized, including the trailing gear.

The remanufacturing process had several other advantages over purchasing new machines, as the remanufacture would be carried out at Jesse Engineering Company, of Tacoma, WA, a local manufacturing facility. Performing the bulk of TBM assembly in close proximity to the project site would:

- Facilitate better supervision of machine construction
- Facilitate earlier training for both TFK and Sound Transit
- Reduce industrial waste
- Generate local jobs

For these reasons, Sound Transit agreed with the remanufacture of the TBMs and Herrenknecht received the order for two remanufactured TBMs.

Mucking

Although nearly identical in design to the MGLEE TBMs, the U220 TBMs were required to use continuous tunnel conveyor for spoil removal, versus the muck boxes that were used on MGLEE.

Screw conveyors were used to extract muck from the excavation chamber. Since the screws act as a sealed mechanism, the muck extraction rate, and thus the pressure in the excavation chamber can be precisely controlled. The machines for U220 utilized three screw conveyors. The first screw extends from the bottom of the excavation chamber up to the top of the first gantry. The number one screw has the ability to extend and retract 1000mm into the excavation chamber to help facilitate muck pickup and repairs to the screw if required. The second screw is connected to the first screw via universal joint and conveys muck to the third screw. Utilizing multiple screws, the earth pressure is stepped down so that upon exit of the third screw conveyor the pressure energy of the material matches atmospheric conditions.

Once the muck exits the screw conveyor, it is transferred to the continuous tunnel conveyor via a transfer conveyor. Excavated material, measured by two belt scales installed on the transfer conveyor, gives direct feedback to the operator of the weight of material excavated. It is imperative to the proper tunnel excavation operation that the scales are calibrated and measuring properly. Because the scales measure a weight, and the speed of the belt is known, a mass flow rate is calculated and displayed to the operator. The operator can then compare actual mass flow rate versus the theoretical mass flow rate. The theoretical value is based upon machine advance rate (the amount of muck being excavated) and the amount of ground conditioners being pumped into the excavation chamber.

Backfill Grout

The backfill grout is injected into the annulus between the excavated ground and the extrados of the segmental lining. The accelerated grout is made of two components; A-Liquid and B-Liquid. The B-Liquid is sodium silicate in liquid form and is the accelerator. Batched at the surface, the A-Liquid is a mixture of water, cement, bentonite, flyash, and stabilizer. Once batched, the A-Liquid is pumped to the TBM using a peristaltic pump through a 2" line. Once on the machine, the A-Liquid is stored in a tank and agitated to keep the mixture from separating.

During machine advance, both the A and B liquids are pumped from their respective tanks on the backup gantries to mixing packers in the shield and out through grout lines that are built within the Tail Shield. Although equipped with four each twin injection ports, only two of the injection ports, located at 10 o'clock and 2 o'clock positions were used.

TFK has proven that grouting from the lower ports provides false feedback for grout injection pressure. When the volume is filled from the bottom, a false pressure is sensed by the pressure transducers because the dynamic viscosity of the accelerated grout is changing with respect to time as well as the static head pressure that continues to increase during filling process. When filling from the top ports, the injection pressure is equal to the earth pressure in the void until the grout reaches the injection point. Once the grout reaches the upper injection point, a back pressure is then sensed by the system pressure transducers. Refer to Figure 5.

A PLC controls the flow and the pressure of the grout. The grout, like the TBM, is limited by earth pressure and the grouting pressure is not to exceed the calculated earth pressure at a given station along the alignment.

Soil Conditioning

Unique soil conditioning was developed during the U220 tunnel project. As with most EPM TBM tunnels soil conditioning is used for a number of reasons:

- Reduce cutterhead torque
- Reduce cutterhead wear
- Reduce ground permeability
- Provide uniform face support
- Reduce stickiness
- Increase flow characteristics of the muck

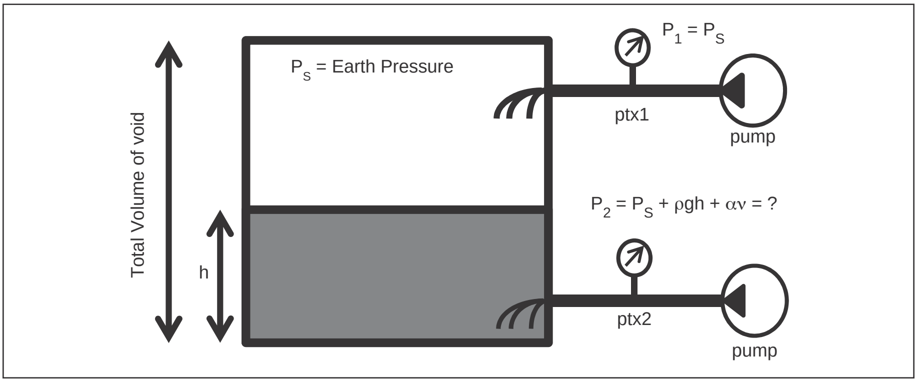

Figure 5. Upper ports versus lower ports

The TBM drives initiated from the UW station, where muck was initially removed using 18 cubic yard muck boxes. Once the TBMs had tunneled far enough (nearly 900 feet), the muck removal system switched over to a 26 inch wide continuous conveyor belt.

From the very start of the drive, TFK encountered the overconsolidated lacustrine clay (Blue SG). It was clear that the material was difficult to handle and as feared, it easily plugged the cutterhead and the screws. Early attempts to condition the material with anti-clay foaming agents proved difficult because the Blue SG ground had such low permeability and high unit weights that it would collapse the foam bubbles. The collapse of the foam bubbles at 2.3 bar (Earth pressure at launch), meant that difficulties certainly would occur at the highest anticipated EPB pressure of 6.1 bar. Furthermore, the collapsed foam bubbles would collect at the top of the excavation chamber, and several times this air had to be “burped” by opening a two inch ball valve in the crown of the bulkhead.

Because the initial portion of the drive utilized muck boxes, controlling the behavior or consistency of the material, although difficult, was not hindering TBM advance. TBM operators made the material as wet as possible to facilitate it in moving through the cutterhead and the three screws into the boxes. In basic terms, they took the material past the liquid limit on the water content continuum. Because the material flowed as a liquid, boxes were easily filled but because the operators were using so much surfactant to get the material flowing, the muck was too wet and consumption of the concentrate was nearly 20 times normal use. As a result, TFK decided to suspend the use of surfactant and allowed only water for the TBM conditioner.

As mentioned above, once the advance had reached about 900 feet, the mucking was switched over to the continuous conveyor belt. At first, the TBM operators were given full reign over the soil conditioning. Their thought process was the same as with the boxes, i.e., get the material wet enough to pass through the screws. Because the material was taken past the liquid limit, this proved disastrous since the material would not stay on the belt to be transferred out of the tunnel to the muck bin. Machine advance went from 20 minutes per push to two to three hours per push. The material was just too wet and slippery.

To keep material on the belt it had to be dry, yet plastic enough to move through the cutterhead and screw conveyors. Through several iterations, a fully automatic conditioning system was developed that controlled the amount of soil conditioner, both liquid and air, that would be injected into the excavation chamber. The system used feedback from the cutterhead drive and the first screw conveyor drive torques as well as advance speed. Based on the feedback information, the PLC would adjust up or down the amount of conditioner required. The results were an earth paste that had the consistency of dry crumbly cheese, and traveled along the conveyor system with minimal issue.

Production Rates

The U220 TBM tunnels progressed very well with the first machine finishing three months ahead of schedule and the second machine finishing four months ahead of schedule. Average production rate was 55.7 ft/day including all stops for crosspassage investigations, conveyor transitions, and some crosspassage work. The best production was 175 feet/24 hours for one machine, while the other machine advanced 80 feet in the same 24 hours.

Limiting factors of production were:

- Backfill grout batch plant production
- Trucking restrictions
- Segment delivery
- Water delivery to TBM for conditioning

Conveyors

A continuous tunnel conveyor system was required by the Contract. Supplied by The Robbins Company of Solon, OH, TFK used a 26" wide tunnel conveyor belt to move material from the TBM to the surface. The belt traveled at 625 feet/minute and had a capacity of 500 tons/hour. The maximum instantaneous material mass flow rate is approximately 502 tons/hour with an average flow rate of 300 tons/hour.

Because of the tight excavation schedule, the installation of the conveyor system on the surface started immediately after the TBMs were assembled. To do so, a decking system was installed over the shaft, which supported not only the belt storage cassettes, but also the main drives and a bend pulley structure. The bend pulley structure facilitated the direction of the belt to the belt storage cassette located over the shaft.

Each tunnel belt was driven by a 250 horsepower main drive and controlled by a Variable Frequency Drive (VFD). A booster drive was installed approximately half way through the tunnel drive. The conveyor PLC, located on the surface controls both drives for each tunnel. The PLC monitored electrical loads on the VFDs as well as all emergency stops. Communications to the booster drive were via fiber optic link to the TBM, with communications to the E-Stops handled over a Dupeline system. The PLC was programmed in such a way that the motor torque between the main drive and booster drives were equalized.

The muck from the tunnel conveyor was transferred to a 30 inch wide overland stacker belt, which then dumps the muck into the muck bin. A separate overland stacker is provided for each tunnel. Even though the overland stacker conveyor had an equal theoretical capacity as the tunnel belt at a slower speed, TFK encountered numerous plugs at the transfer point. The solution was to speed up the stacker belt faster than the tunnel conveyor through the VFD.

Because there is a communications link between the conveyor PLC and the TBM, the TBM operator controlled the conveyors from the TBM and monitored all of the emergency stops.

Concrete Tunnel Lining

The Precast Segmental Tunnel Lining serves two functions. Its primary function is ground support as the tunnel advances. Its secondary function is to provide a reaction for the TBM thrust jacks to advance the TBM. It is therefore imperative that the construction of the tunnel liner be of the highest quality, not only for aesthetic purposes, but also for safety purposes, as the quality of the built ring affects the performance characteristics of the liner.

The tunnel lining consists of a series of nominally five foot long rings each of which consist of six segment pieces in a 5+1 configuration (five large size segments and a small keystone). Segments arrive to the jobsite from the Tacoma plant on a daily basis. The segments arrive in two stacks of three. From the trucks, the segments are offloaded to the storage area utilizing a Taylor 30,000 lb forklift fitted with adapter forks to mitigate surface damage to the segments.

The segments are lowered down the shaft using the site crane, a Liebherr HS 895, to the awaiting segment cars using two each four inch straps with softeners. The segments are transported into the tunnel on two specially designed segment cars pushed by a 35-ton locomotive. These segment stacks are rotated on the segment cars by hand and then are lifted several inches by the segment stack lifters located on Deck 1 of the TBM. The segment lifters allow for the segment cars to leave the heading once the segments are unloaded.

Once a TBM advance is complete, each segment is transported via segment hoist with a vacuum head to the segment feeder. The segment feeder, as the name implies, feeds segments to the segment erector. In order to maintain active face support during the ring build, only the thrust jacks where the segment is being placed are retracted while the remaining jacks continue to apply force.

Using the TBM erector, the segments are installed one at a time with the Key Stone being the last installed. The segments are bolted on the radial joints with special galvanized segment bolts utilizing air impact wrenches. The circumferential joints are connected using Sofrasar Sof-Fix dowels.

The Contract calls for a tapered ring to be used in the tunnel, either a left and right or universal ring. The trapezoidal taper allows for the rings to be placed in such a way that they follow the excavated tunnel. TFK has chosen to use the two different types of rings, referred to as an “Up Ring” and a “Down Ring.” The two ring styles give the ability to install key segments above spring line, which is generally accepted as being a safer method of ring erection. An “Up Ring” has a taper that is designed such that when the key is installed at the 12 o’clock position, the narrowest section of the ring is in the crown and the widest section is at the invert. Conversely, a “Down Ring” has a taper that is designed with the widest section at the crown and narrowest section at the invert when the key is installed at the 12 o’clock position. Ring type (Up or Down) and orientation are calculated specifically for each ring with ring-building software. The software takes into account a number of parameters including thrust jack extension and tail shield gap clearances (collected manually) to determine the best fit ring. With the telemetry data from the thrust jacks, tail gap clearances, TBM orientation with respect to the DTA, and the previous ring type built, the ring build software calculates a best fit ring and a variety of alternatives that will best fit the tunnel without creating cruciform joints.

In order to minimize the risk of the precast concrete tunnel lining coming into contact with the tail shield of the TBM, a condition often referred to as becoming “Iron Bound,” the TBM has been specially designed such that the thrust jacks continuously load the rings axially; as such, the tail shield axis remains parallel with the built tunnel axis. This same design was used on the MGLEE tunnels in Los Angeles where the TBM negotiated an 800 foot radius horizontal curve without becoming “Iron Bound.”

Hyperbaric Preparation

Due to the expected high water heads along the tunnel alignment, the potential for unstable ground during interventions, and the recent difficulties at the nearby Brightwater Central tunneling contract, TFK JV put significant effort into preparing for hyperbaric interventions. TFK JV partnered with several consultants and industry experts to help in planning and training for the potential hyperbaric work. These included Dr. Tommy Love and Kevan Corsan as medical advisors, Georges Gourdon of Hyperbarie SARL as a technical advisor and trainer, Ballard Diving as Trimix technical advisors, and Steven Reimers of Reimers Systems as technical advisor for ASME code issues.

Planning

TFK JV produced a comprehensive Hyperbaric Operations Manual covering the full range of pressures that were expected along the alignment. This plan was used not only as the operational guideline for the potential hyperbaric work, but also as a submittal to the State of Washington Department of Labor and Industries to secure variances necessary to perform the high pressure intervention work. The variances that were applied for and granted can be summarized as follows:

- Variance to use French decompression tables
- Variance to use manual decompression controls
- Variance to perform work above 50 psi
- Variance for manlock headroom less than 6 feet

- Variance to decompress for longer than 75 minutes without a special decompression chamber
- Variance to use female medical attendants

While the highest pressure expected along the alignment was 88.5 psi (6.1 bar), the contract documents limited hyperbaric interventions to 75 psi (5.17 bar). Following this guideline and taking into account specific project conditions, TFK JV prepared for four primary intervention types, and a fifth contingency mode:

1. Intervention breathing air with air decompression between 0 and 26.1 psi (0 to 1.8 bar)
2. Intervention breathing air with oxygen decompression between 21.8 and 58 psi (1.5 to 4.0 bar)
3. Intervention breathing air with oxygen decompression between 58.0 and 69.6 psi (4.0 to 4.8 bar)
4. Intervention breathing Trimix with oxygen decompression between 58.0 and 75 psi (4.0 to 5.2 bar)
5. Intervention breathing Trimix in saturation mode for long prolonged repairs in high pressures.

Each intervention type required different decompression tables, training, and equipment. A rigorous training program for all compressed air workers, manlock tenders, supervisors, and engineers was provided by TFK JV and their consultants. Overall, 30 individuals were trained to perform hyperbaric interventions.

TBM Equipment

Because of the high pressures and potential for multiple long interventions, the TBMs were specified by TFK JV to be equipped with state of the art hyperbaric facilities and equipment, with each TBM providing seven air chambers:

1. Excavation Chamber (working chamber): the area between the cutterhead and the TBM pressure bulkhead. This is the area where the intervention work will take place.
2. Staging Chamber: The Staging Chamber provides access from the Man Locks to the Excavation Chamber, as well as storage and staging for equipment and tools.
3. Man Locks: There are twin sets of transfer air locks connected to the Staging Chamber. Each Man Lock is composed of two compartments; the Main Chamber and the Pre Chamber (for a total of four Man Locks.)
4. Material Lock: The Material Lock is connected to the Staging Chamber and allows heavy tools, equipment, and materials to be passed into the Excavation Chamber efficiently.

The two independent sets of Man Locks connected to the Staging Chamber allows a compressed air worker team to be operating in the Excavation Chamber while a second compressed air worker team is in decompression. The Man Locks, which provide access from atmospheric pressure to the compressed air work area, are permanent features of the TBMs and are fully equipped to compress or decompress the compressed air workers. The Man Locks are configured with a two-person Pre Chamber and a three-person Main Chamber, with both sets of chambers equipped with Trimix and oxygen breathing apparatuses.

The addition of the Staging Chamber to the TBM provides several important safety features. First, it is large enough that compressed air workers can enter it during decompression and stand fully upright. This is beneficial since it is believed that providing space to stretch-out decreases the likelihood of decompression illness by preventing the buildup of nitrogen bubbles in a person's joints. Since the headroom in the Man Locks is approximately five feet, it is difficult for a three person compressed air worker team to fully stretch out if only the Man Locks were available for doing so. Second, the Staging Chamber provides a "buffer zone" between the dangers of the Excavation Chamber and the relative safety of the Man Locks. Instead of the heavy, bolted access door to the Excavation Chamber being located in the Man Locks, it is located in the Staging Chamber. This way, if there is an uncontrolled inrush of water or ground when the access door is opened, the compressed air worker team can abandon the door and retreat to the Man Locks, where they simply need to close a relatively light inward-closing self-sealing door rather than a heavy outward-closing door that can only be sealed by fastening it shut with multiple heavy bolts and air tools.

The TBMs were not only prepared for bounce diving operations, but TFK JV also worked with Herrenknecht AG to make preparations for saturation diving in the event sustained high pressure work needed to be performed. The TBMs were designed to allow the Material Lock to be removed and have a saturation "access pipe" installed in its place. This access pipe would allow saturation divers to be transferred under pressure from the Excavation Chamber to the Shuttle Lock and then to the surface Habitat Lock. While the access pipe was never produced, TFK JV asked Herrenknecht AG to produce a full design package for the pipe including all manufacturing drawings, an installation plan, and all necessary certifications so that the pipe could be manufactured in less than three months.

Saturation diving operations never became necessary and the access pipe was never manufactured.

Surface Equipment and Locks

In addition to the facilities on the TBMs, preparations needed to be made outside the tunnel as well. These preparations included designing and assembling a compressed air supply plant and procuring specialized Medical and Shuttle Locks.

The compressor plant consisted of six oil-free compressors located on the surface. These compressors could generate a total of 3,942 ft^3/min at 145 psi, which was about double what TFK JV calculated would be necessary to account for air loss through the ground and the air necessary for ventilation. The compressors were plumbed to a header pipe that fed five refrigerated dryers which not only removed moisture from the air (by reducing dew point) but also cooled the air. After the dryers, the air passed through a bank of activated carbon and particulate filters set in series to ensure the air met breathing quality standards. The compressed air was then delivered to the heading via a single 10" steel pipe. A secondary 10" pipe was also installed in parallel as a 100% backup to the first in case of an emergency.

State and Federal regulations require a Medical Lock to be present on site when hyperbaric operations are taking place. The Medical Lock has four main functions:

1. Emergency recompression and decompression
2. Treatment of decompression illness
3. Pressure trials for medical screening
4. Pre-intervention training under pressure

TFK JV worked with Reimers Systems to design a containerized Medical Lock that could be easily moved and setup from project to project. It was designed for a working pressure of 75 psi and can accommodate six people seated. It is also equipped with a therapeutic gas supply system and back up air supply systems so that it can continue to be operated in the case of power loss. One of the most important features of the Medical Lock however is that it is fitted with a Tube Turns flange so that the Shuttle

Lock can be directly connected to it and an afflicted compressed air worker can be transferred under pressure, thus eliminating the risks associated with decanting.

While not required by regulation for normal hyperbaric operations, a Hyperbaric Shuttle Lock was required by the State of Washington Department of Labor and Industries in order to secure a variance to perform interventions in pressures over 50 psi. The Shuttle Lock is a rail mounted, portable recompression chamber that is staged in the tunnel close to the Man Locks during hyperbaric activities above 50 psi. It has the capability of connecting, under treatment pressure, to the Medical Lock. Its primary purpose would be to, in an emergency, transfer a compressed air worker with an injury, medical condition, or decompression illness, plus an attendant, from the tunnel to the Medical Lock. The Shuttle was designed for a pressure of 100 psi, can operate independently for up to three hours, and is capable of transporting six people seated.

The Man Locks, Medical Lock, and Shuttle Lock were all certified according to ASME PVHO-1 2007 codes. Additionally, the Medical Lock is FDA compliant.

While it can be seen that significant preparations were made for hyperbaric interventions, it turned out that none were performed on the U220 Contract. TFK JV was able to stop the TBMs and perform interventions in areas of Blue Soil Group with stable ground conditions and little to no water inflow.

Tunnel Schedule

The progress of the two tunnels was able to be accelerated using a few key scheduling advantages, allowing the TBM production mining to be completed nearly four months ahead of schedule. During TBM mining, the specifications called for an inspection stop at each cross passage location (16 per tunnel) to determine ground conditions prior to SEM excavation and before proceeding underneath the Lake Washington Ship Canal. Using controlled decompression of the excavation chamber by monitoring several EPB sensors within the chamber and watching a video feed from a camera mounted inside a window to the excavation chamber, ground conditions could be evaluated, allowing for a simple decision of entering the chamber or continuing with excavation. This led to fast 'stop-and-go" interventions, often only taking only two to six hours for an entire ground condition inspection and eliminating the necessity of entering the chamber. Addition of the excavation chamber camera also permitted TFK JV to plan cutterhead inspections in ground where hyperbaric interventions would not be necessary, saving both time and cost. Maximum advance for a single 24 hour period with one machine was 175 feet while the best combined day for both machines was 257 feet.

Settlement Data

Settlement data was collected along the entire TBM alignment and at both starting and ending shafts, with additional instruments situated on both sides of the Montlake Cut and near a historic water tower directly under which the TBMs will pass. Instruments used for this data collection were strain gauges, multi-point bore hole extensometers, inclinometers, utility settlement points, surface settlement points and structure settlement points. Baselines were recorded at these points before excavating the University of Washington Shaft and before the TBM began excavating. During the shaft excavation these instruments were monitored on a daily basis and then reduced to a monthly reading after excavation was completed. For monitoring the settlement along the TBM alignment, daily readings were taken whenever the TBM was within 200 feet of the instrument. If conditions were satisfactory as the TBM reached each instrument, the readings were reduced to weekly and then monthly, and will gradually taper down to even less frequent observations until decommissioning. Settlement along the tunnel alignment was well controlled and was typically far less than the 13 mm trigger level outlined in the contract.

Ground Borne Noise

Ground borne noise proved to be a challenge for the contractor, client and members of the community surrounding the shaft sites and those along the alignment. Sound Transit received multiple complaints from residents that could hear and feel locomotive traffic in the tunnels located up to 300 feet below their property. There were two primary complaints: (1) constant "bowling ball" noise—created by heavy train traffic running on steel rails and (2) intermittent "thump-thump"—created by train wheels running over joints in the rail.

In response, Sound Transit initiated a significant mitigation effort that included replacing steel track ties with timber, gluing rubber pads under the steel ties, welding and grinding rail joints and reducing locomotive speeds within certain reaches of the tunnel during night hours. None of the mitigation measures completely eliminated the vibration issues or community complaints, however they did improve the situation to the point that project could be completed.

Additional night time restrictions were imposed on demolition operations for cross passage segment break out and propping demolition within specific sections of the tunnel based on the complaints received during TBM mining.

Permanent Invert and Walkway Tunnel Concrete

The main areas of permanent cast-in-place concrete in the tunnel were the tunnel invert, the cross passage final linings, a permanent walkway that runs the length of the tunnels for emergency and maintenance access, and a 1,240 foot duct bank that houses conduit for the traction electric system. The concrete used was a 4000 psi mix with both polypropylene and steel fibers and several admixtures to delay set time and maintain sufficient slump. Due to the difficulty of getting concrete in location through the full length of each tunnel, TFK used four front-discharge 12 cubic yard capacity concrete mixer trucks to transport concrete from the shaft through the tunnels to the placement location. Concrete arrived on site from a local batch plant and was loaded into the mixer trucks via a down-pipe from the surface to the shaft bottom. The concrete was then driven to its final design location and either placed directly with the concrete truck's discharge chute or into a pump for the cross passage final linings and tunnel inverts. Using this method, TFK was able to pour 400–600 feet of tunnel invert daily and 700 feet of sidewalk every other day.

CROSS PASSAGES

Overview

There are 16 cross passages along the route at approximately 800 ft centers. Typical cross passages have a maximum width of excavation of 12 ft 4 in, while two specialty passages, for a sump and interconnection of Traction Electric conduit go up to a width of 16 ft 8 in. All of the cross passages require an opening in the segmental lining two rings wide, within which a permanent concrete support is cast. The opening requires temporary support from break-out until the permanent concrete has achieved its specified strength, and the contract specifies this to be contractor's design (Figure 6).

The contract defined two potential ground support categories (Category I and Category II) and provided detailed prescriptive design for each situation with a toolbox of additional support items that could be utilized if ground conditions warranted. Excavation sequence identified in the contract documents called for removing alternating rounds of top heading and bench with lattice girders and steel fiber reinforced shotcrete providing temporary support.

Final lining for permanent structural support could be provided in one of two ways: either entirely out of reinforced cast in place concrete or through a combination of

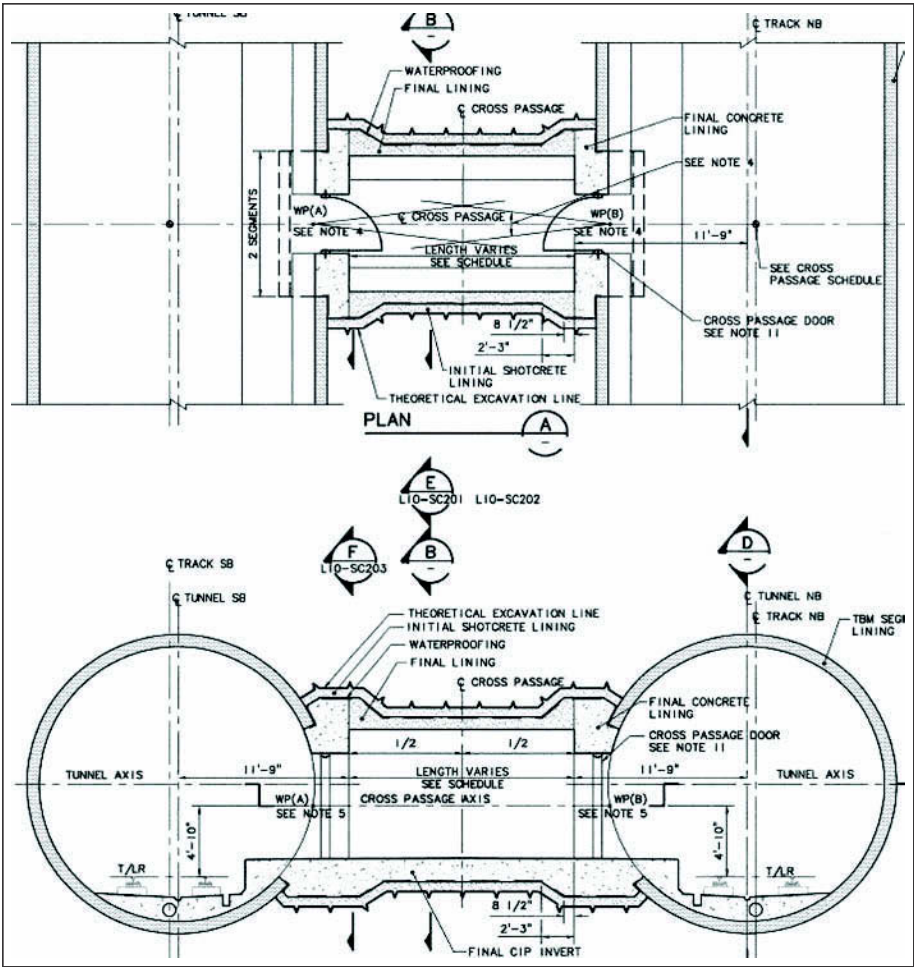

Figure 6. Typical cross passage layout

shotcrete and cast in place concrete. The work associated with the cast in place concrete option that was selected for the final lining will be discussed later in this paper.

Schedule

After allowing for site preparation, shaft excavation and TBM mining, the contract schedule allotted only 10 months to construct the cross passes, place tunnel invert and walkway concrete, complete electrical and mechanical work and demobilize. The large number of cross passages (16) made it fairly obvious that the project could not be completed within the allotted time if cross passage work did not commence until after the completion of TBM mining. TFK began planning how to simultaneously progress cross passage excavation and TBM mining in the same tunnel to achieve the required substantial completion deadline. As an added bonus, it was determined that a minimum of two cross passages would have to be excavated simultaneously while several others would be undergoing waterproofing installation and final lining construction.

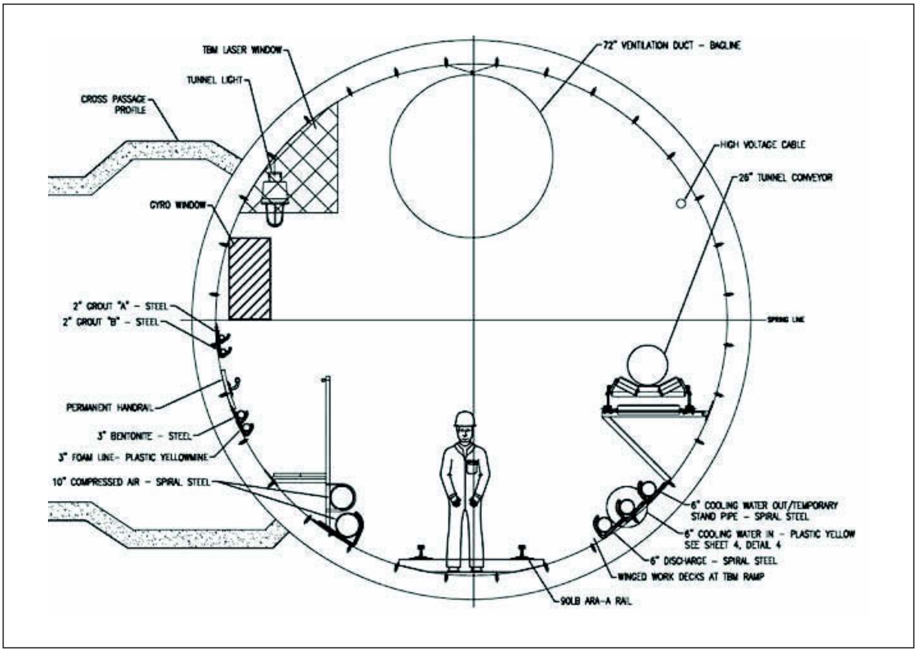

Figure 7. Typical tunnel cross section

The principal benefit of progressing cross passage excavation concurrently with TBM mining was to reduce overall schedule risk and exposure to significant liquidated damages. Secondarily, reduction of the overall schedule would help to avoid additional overtime and other acceleration costs in addition to the elevated safety risks inherent with the rush to finish a project on time.

A number of temporary services are installed along the length of the TBM tunnel (Figure 7) and need to remain in operation during cross passage excavation. Installation of the temporary support system required momentary interruption of these services to provide access to the cross passage work. Some interruptions lasted only a few minutes as hard line pipe clamps were removed and soft hoses were routed around the work area. This was the case for all of the utilities except for the three water lines (cooling in, discharge out and cooling out/temporary standpipe). These utilities remained in service at all times and were gradually blocked away from the tunnel way to provide adequate clearance for installation of the shotcrete shell. Belt conveyor and structure was also left in place for the duration of the works with the drive motors locked out when access around the belt itself was required. The ventilation bag line created the biggest utility challenge as it occupied a significant amount of space and severing it prevented any work from occurring further down the tunnel. As mentioned previously, maintaining train traffic through the cross passage work area was of primary importance during TBM mining shifts. Work on temporary support installation had to be coordinated with the ongoing TBM work and frequently shifted to non-mining maintenance shifts or weekend days.

This was a new approach for the TFK team and the problems that needed to be overcome to make it happen were numerous and challenging. There are not many pieces of equipment more expensive to purchase, operate, supply and maintain than an earth pressure balance TBM. This quickly determined the driving tenet during cross

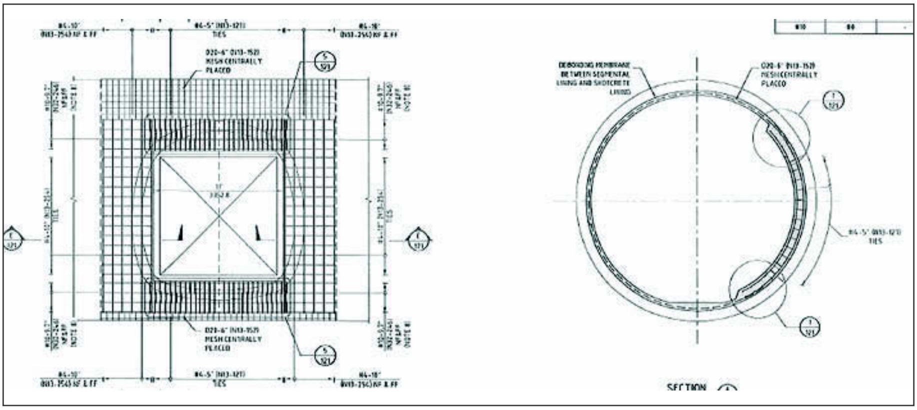

Figure 8. Propping design detail

passage planning "Do not stop the TBM" and created many of the challenges that had to be overcome.

Temporary Propping System

Satisfying the requirement to maintain TBM functionality meant leaving all temporary utilities that run the length of each tunnel, including continuous conveyor belt, in service at all times. Unobstructed access for train traffic must also be maintained. These conditions when combined with the large number of cross passages that needed to be supported at one time precluded using the traditional hamster cage system (rolled steel beams blocked to the intrados of the segmental lining) for temporary segmental lining support. TFK sought out an alternative system that would be more cost effective and reduce tunnel obstructions and contracted Halcrow with design of the temporary propping system.

Halcrow initially considered a steel door frame system that relied on heavy fabricated members for the lintel, sill and jambs. The lintel and sill beams would be rigidly connected to the segmental lining by a large number of high capacity drilled and epoxied anchors. Overall fabrication cost and the large quantity of anchors made this option unattractive.

Fortunately a Halcrow representative who was visiting the job on an unrelated issue came up with another option that fit the situation quite well—a thin shotcrete shell placed directly against the segmental lining around the cross passage opening. With little debate, Halcrow proceeded with design of the shotcrete shell and rapidly produced a complete design package (Figure 8).

Work Schedule

TBM mining was conducted 24 hours per day, five days per week with maintenance performed on the weekends while cross passage excavation and final lining was bid with a 24 hour per day/ six day per week schedule.

TFK's base plan was to commence cross passage excavation from the Southbound tunnel after the trailing gear of the Northbound TBM (which was trailing Southbound by one month) had cleared the third cross passage along the alignment. This approach would have cross passage excavation starting roughly four months after the start or TBM mining. In reality, planning and procurement for cross passage excavation took much longer than anticipated and thus true excavation did not begin until seven months after the start of TBM mining.

Work Preparation

Very early in the job planning process, it was determined that the Southbound TBM would be launched first and that cross passage excavation would be supported from this tunnel. To this end, the ventilation system for the Southbound tunnel was much larger than that of the Northbound tunnel to account for the additional locomotive traffic and personnel required to perform the work.

In addition to a unique engineering solution to the temporary propping system, several pieces of custom equipment were required to make the overall cross passage plan work. The most significant piece of which was the folding work decks that would support all cross passage operations and still permit locomotive traffic to pass through them and the dual scissor car required to install and remove them. Design and fabrication of the work decks and scissor car was a lengthy process that evolved a seeming simple concept into an elaborate and detailed system of intertwined components. The need to maintain appropriate clearances for locomotive traffic (including the scissor car loaded with a folded up work deck) drove the overall geometry of the work decks and limited the overhead room available on top of them. Once the elevation of the bottom of the work deck was established at 100 inches above top of temporary rail, all the components and pieces of equipment that had to pass through the deck could be finalized. The work decks and scissor car were designed by Kelley Engineered Equipment (KEE) of Omaha, NE. KEE also fabricated the scissor car while Traylor Bros., Inc. equipment shop in Evansville, IN fabricated the work decks (Figures 9 and 10).

Propping Construction

Propping installation methodology evolved as TFK's crew became more familiar with the process and were able to refine it. Propping installation boiled down to three basic steps:

1. Segment preparation
2. Reinforcing steel and formwork installation
3. Shotcrete placement

Segment preparation entailed installing small plywood covers over the bolt pockets, cleaning off muck and placement of two coats of bond breaker. A series of tests alleviated our concerns about shotcrete sticking excessively to the segmental lining, which would make removal difficult.

Throughout the propping installation process the basic reinforcing steel detail remained unchanged—the radial and longitudinal steel in the thinner shell section was shipped loose and installed piece by piece while the reinforcement for the thickened section around the cross passage opening was separated into four cages that were pre-assembled off site and set in place. A series of hangers and slab bolsters were utilized to provide appropriate clear cover against the segmental lining and adequate support to hang the cages. Sections of plywood were cut to the appropriate shape and mounted to the segmental lining to provide control and containment during shotcrete placement (Figure 11).

Dry mix shotcrete was placed by hand to the required profile. Shotcrete was delivered to the job site in one cubic yard super sacks and transported to the cross passage work via flat car and locomotive. A custom shotcrete delivery trolley was mounted to a flat car and provided storage for up to 10 cubic yards of dry mix material while hoisting super sacks high enough to clear the hopper on the pre-dampener. The shotcrete delivery trolley was also capable of laying itself down so that it could pass underneath the previously mentioned cross passage work decks. Initially a stand-alone scissor car mounted work platform was utilized to access the upper level reinforcing steel, formwork installation and shotcrete placement. This system was functional but required

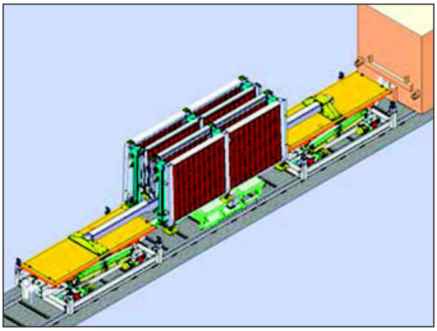

Figure 9. Scissor car and work deck in transport

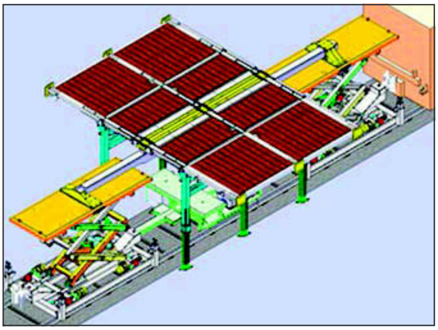

Figure 10. Work deck deployed

Figure 11. Completed propping reinforcing and formwork with work deck in place

blocking of the TBM tunnel and severing of the ventilation bag line to allow significant work to place, limiting work to weekends and other non-mining shifts. In short order, TFK figured out a work sequence whereby the cross passage work decks could be installed and utilized to provide access for propping installation without blocking the tunnel. Shotcrete placement still shut down the balance of the tunnel. Additionally, an alternative means of severing and locally adjusting the ventilation bagline location permitted work to occur on the propping during TBM mining. These arrangements worked very well and greatly increased the efficiency of propping installation.

Pre-Support

Pre-support at each cross passage was provided by a series of spiles (either #10 rebar or two inch diameter pipe) over the top of the cross passage excavation profile. Each subsequent round of top heading excavation also called for spiles arrayed around the crown until the spiles reached to opposite tunnel lining, irrespective of ground conditions.

After installing 10 foot long pre-support spiles at the first two cross passages, TFK elected to install pre-support spiles that reached the lining of the opposite tunnel and eliminate the spiles called for in the subsequent top heading excavation rounds. The competency of the ground allowed the full length spiles to be inserted and grouted into pre-drilled holes and predominantly eliminated the need for additional top heading spiles.

Dewatering

Based on probe hole drilling data and information collected during TBM stops at cross passage locations, it was determined that one of the Category II cross passages (CP 6) would require some amount of vacuum dewatering prior to commencing excavation. A series of fourteen three inch diameter cores were drilled around the periphery of the cross passage profile and 1.25"Ø commercially available well screen was drilled and driven through the core hole up to eight feet deep. Each vacuum lance was grouted into the segmental lining and attached to one inch discharge hose. All such hoses were gathered and tied into a single dewatering header that was hooked up to a four inch centrifugal pump. After nearly a week of operation a piezeometer installed in one of the previously drilled probe holes showed the ground water level had dropped nearly ten feet. While there was concern about the ground in this cross passage, the small dewatering effort made it the driest and safest excavation on the project.

Cross Passage Excavation and Support

The need to maintain train traffic for TBM mining resulted in the use of work decks at each cross passage which greatly hampered access to perform cross passage excavation. TFK's base plan was to perform the vast majority of the excavation work with a Brokk 180 and assorted attachments. In reality, the work decks set too high in the tunnel to permit the Brokk 180 to access the cross passage excavation from on top of the deck. Accessing the excavation from below the deck would block the tunnel and halt train traffic supplying the TBM.

This left the only available option as hand excavation of the first several rounds until sufficient room was created for the Brokk to walk into the cross passage from beneath the work deck. Two rounds of top heading and one bench round were hand excavated at the first cross passage prior to trying to work with the Brokk 180. Attempts to continue excavation with the Brokk proved unsuccessful as access was limited and the cross passage excavation was not wide enough to allow the Brokk to swing 180 degrees for spoil removal. After two cross passage of trying, all mechanical excavation was abandoned in favor of hand excavation. At the peak of cross passage excavation, each crew could hand excavate and support a complete cross passage in twelve working days.

After the Southbound TBM had completed mining, excavation methodology transitioned to a flat car mounted mini excavator that loaded muck boxes and walked into the cross passage as necessary. While the use of rail mounted equipment and muck disposal greatly eased the excavate work, it required careful coordination of muck box removal and delivery of other materials as multiple cross passages were always being excavated (Figure 12).

Typically, the top heading was excavated all the way across and braced with a temporary shotcrete invert prior to starting bench excavation. This sequence changed slightly when machine excavation became practical to include bench excavation rounds as necessary to permit the exactor to continue mining top heading rounds. All top heading rounds were excavated in a minimum of four smaller pockets to reduce employee exposure to unsupported ground. Typical bench rounds were excavated in two pockets.

Initial ground support was provided by a two inch layer of steel fiber reinforced shotcrete with a subsequent lattice girder supported steel fiber reinforced shotcrete layer of varying thickness depending on the cross passage location. Similar to the propping shotcrete system, dry mix shotcrete was delivered in one cubic yard super sacks and hoisted onto the work decks at the cross passage. An overhead monorail with an electric hoist maneuvered the shotcrete bag over the hopper of the combined pre-dampener/shotcrete pot which produced material that was placed by hand held nozzles.

Figure 12. Hand excavation at cross passage 20

Propping Demolition

The project schedule that required temporary propping to be in place in tunnels and at all sixteen cross passages also drove the need to leave all the propping in place until very late in the job. Delaying propping removal made it imperative that an efficient demolition methodology be developed and implemented. We spent countless hours discussing potential options for propping demolition and the manner in which the demolition debris would be removed from the tunnel.

After one rather unsuccessful attempt, an effective methodology and sequence were arrived upon. A rubber tired Gradall armed with a hydraulic demolition hammer working in conjunction with a smaller excavator equipped with a hydraulic hammer, bucket and thumb proved to be the proper combination. With an excavator situated on either side of the cross passage and appropriate protective devices in place around the utilities, the demolition procedure begins by breaking out a swath of the shell section along the full length of the crown. The excavators next turn their attention to the upper portion of the shell section on the non-cross passage side of the tunnel to break the bond between the shell and segmental lining allowing the shell section to be pulled off the wall in a few large pieces. The large pieces are processed, sorted and loaded out before additional demolition occurs. With a clean invert the excavators begin breaking a relief cut across the columns on the cross passage side roughly two feet below the lintel cage. With the relief cut complete, the large machine starts breaking the lintel cage free from the segmental lining and it falls off in one large section. The two columns are then broken down traditionally to the top of the sill beam (Figure 13).

Demolition halts at this point and moves to the next cross passage. The sill beam and invert portion of the propping are removed in a follow on operation when there is good access for rapid completion of the invert concrete across the propping width.

Lower level propping demolition is performed in a similar manner with the same equipment spread. The sill beam is removed in several large sections while the remainder of the invert is broken up traditionally and removed.

Demolition debris was removed from the tunnel using concrete recycling boxes mounted on a tilt bed truck.

Cross Passage 17

The inspection stop from the Southbound TBM for cross passage 17 was performed on September 19, 2011 and the inspection indicated unstable ground conditions in

Figure 13. Conveyor side shell demolition

addition to continuous in-flowing water to the excavation chamber. A second inspection stop was performed 40 feet south of the first location showing improved ground conditions that appeared to be suitable for a safe cross passage excavation. The Northbound TBM performed an inspection stop at the second location and confirmed the desired conditions. The cross passage was thus moved to this second location in favor of more stable ground conditions for excavation.

Prior to excavating a cross passage, three probe holes were drilled through the segmental lining and into the ground that would be excavated for cross passage to confirm ground conditions. With the probe holes at cross passage 17 showing no signs for concern, excavation of the cross passage commenced on March 19, 2012. Similarly to the previously excavated cross passages the sequential excavation method was utilized at cross passage 17 with excavation of the complete top heading (from the southbound toward the northbound tunnel) being the first order of business. The material encountered in the top heading was wet and soft but stable clay. As each top heading girder was installed, spiles were grouted in place to provide additional overhead protection. As top heading excavation progressed, ground water seepage into the excavation adjacent to the southbound tunnel lining steadily increased up to a maximum flow rate of 35 gallons per minute. It was decided that after completing the top heading excavation and placing the temporary shotcrete invert a number of probe holes would be drilled through the temporary invert to explore ground conditions in upcoming bench excavation. During this probe hole investigation, a significant stream of groundwater and sand began to flow up the hole and into the cross passage. By the time a packer was installed, nearly five cubic yards of sand were deposited in the cross passage and the running tunnel. Four other probe holes were drilled and contained with packers to further monitor the water table.

After losing the material and detecting flowing water in the bottom invert of the excavation, it was determined that further dewatering measures would be needed to ensure the bench excavation could be performed safely. Much discussion and debate on the path forward ensued. The first dewatering plan enacted called for 10 vacuum well points to be installed from within the twin TBM tunnels (five per tunnel) under the cross passage excavation to depths varying between 20 and 40 feet. Each well was comprised of a full-length two inch PVC pipe with an eight foot long slotted screen section at the bottom of the well. Well heads were grouting into the segmental lining and reduced to one inch discharge. Discharge lines within each tunnel were connected to a single dewatering header and hooked up to the suction side of a screw sucker centrifugal pump. After commissioning of the in-tunnel system it was allowed to run for several weeks while the earlier probe holes were used to evaluate the impact on the local ground water level. While the in-tunnel wells did lower the ground water level they did not lower it to a level that would allow the bench excavation to be performed, which meant that a secondary dewatering system was needed. The system decided upon called for up to three deep dewatering wells to be drilled from the surface to an elevation below the bottom of the cross passage excavation (max depth of 240 feet). Surface well installation brought with it its own difficulties including significant well heave and sand and water infiltration into the cross passage of more than 500 gallons per minute. Ultimately the combination of the surface dewatering system and the in-tunnel system were successful in lowering the water level to a point where bench excavation could proceed.

With the full system running for several days and evidence of the water table falling several feet below the limits of the cross passage excavation profile, excavation resumed on September 17, 2012. The dewatering systems were kept on throughout the excavation which encountered only one issue. About halfway into the cross passage, a large void on the northern side was encountered. The end of the hole could not be visually inspected, but a screen was installed and the hole was filled with grout and then shotcrete was placed to seal the hole. Excavation continued to complete the full profile without any further problems arising. After the initial lining was completed, the waterproofing lining was installed in typical manner with high priority of getting the final lining completed as soon as possible.

After completion of the final lining, the wells were decommissioned in accordance with state regulations. Several probe holes were drilled from within the tunnel to inject grout around the cross passage with the intent of filling any voids that were created by the flowing water and lost ground during excavation.

Final Lining

After the initial shotcrete lining has been placed, a two inch smoothing layer of plain shotcrete was applied to cover any of the exposed steel fibers and to remove any extruding corners to allow for proper placement of the waterproofing membrane. Following installation of the waterproofing membrane and placement of a protective concrete layer, reinforcing steel was installed and secured to create an invert from which the arched walls and doorway bulkheads could be erected. The final lining was cast-in-place using a pump that is positioned in the tunnel and fed by the tunnel designated concrete trucks. The doorway bulkheads were erected first using modular steel panels and provide structural replacement for the segments removed from the tunnel ring for access into the cross passage. After the bulkheads achieved their design strength and were stripped, wooden arch forms were erected to achieve the necessary curved interior walls. Concrete was pumped through pour ports in the forms, filling from the bottom up until the crown was full. After the forms were stripped, any voids behind the new walls were contact grouted using a neat cement grout pumped to a pressure of 50 psi. The final step was to inject a water stop grout to fill any remaining voids within the cross passage walls and cold joints.

The most difficult aspect of the final lining was water intrusion. After the initial lining was placed and the water proofing membrane was installed, there were instances where water would still build up behind the membrane. The water would create bulges in the membrane causing it to be ineffective and add difficulty in setting the rebar for the final lining. Additional water stop measures had to be used, including polyurethane grout injection and swell sealants at the joints.

CONCLUSION

From the very beginning, it was envisioned by many of the project stakeholders that the U220 Contract was going to be a challenging project, especially from a scheduling and community impact point of view. As of January 2013, we are pleased to report that the project is projected to finish on time, within the envisioned budgets, and with minimal overall impact to residential neighborhoods and the University of Washington. This was accomplished by an outstanding partnership between the general contractor Traylor Frontier-Kemper and their subcontractors, the owner Sound Transit, and the University of Washington. We would like to thank all of the individuals who expended such a great effort over the last four years to make the project a resounding success.

EAST SIDE ACCESS—QUEENS BORED TUNNELS CASE STUDY

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Jean-Marc Wehrli ▪ Granite-Traylor-Frontier Joint Venture

ABSTRACT

East Side Access Contract CQ031, in Queens, New York, involves the construction of 10,500 feet of pressurized soft ground tunnels, beneath rail yards and mainline tracks in the Sunnyside Yards. The project was awarded to GTF, the Joint Venture of Granite Construction Northeast, Inc., Traylor Bros., Inc. and Frontier-Kemper Constructors, Inc. (hereafter referred to as 'Contractor') and was managed by New York's MTA Capital Construction Company (hereafter referred to collectively as 'Owner'). The Designers of the project were (a tri-venture of Parsons Brinckerhoff, STV and Parsons Transportation). The four 22-ft diameter tunnels were constructed through soft ground under numerous active rail tracks in the Queens Sunnyside Yard. The following paper is a Case Study of the Queens Bored Tunnels Project. Please also refer to an additional paper which details some of the projects 'Engineering Challenges.'

INTRODUCTION

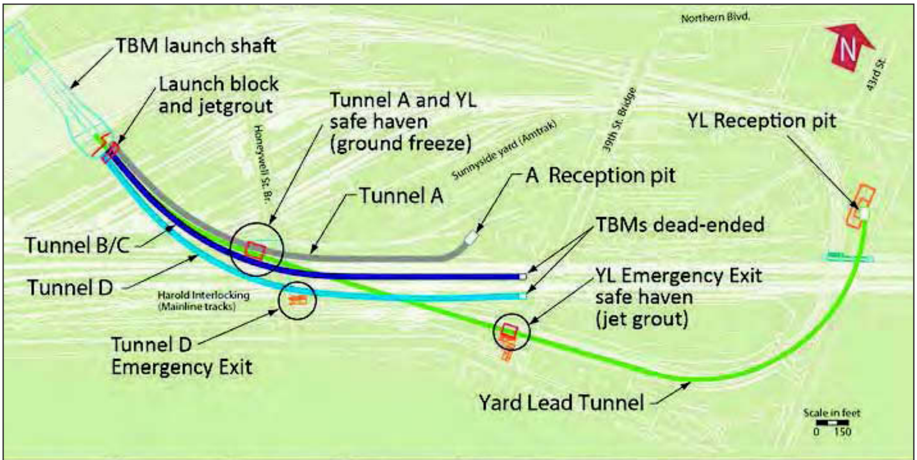
Contract CQ031 included the excavation of four, single track tunnels (Tunnels A, BC, D, and the Yard Lead Tunnel) using two TBMs launched from an open-cut excavation located in the northwest corner of Sunnyside Yard. The tunnel alignments run in a southeasterly direction diagonally across Sunnyside Yard and then turn east, parallel to the existing Long Island Railroad mainline tracks (refer to Figure 1).

GEOLOGY AND ALIGNMENT

The Yard Lead Tunnel lies below tunnel drives A, BC and D, and runs beneath the mainline tracks after passing under Sunnyside Yard. It then continues under the Amtrak loop tracks before turning back towards Sunnyside Yard and terminating at the Yard Lead Approach Structure. The tunnel was driven on an uphill gradient throughout, varying between 0.25 and 3.25%, with overburden decreasing from maximum 21m (68ft) to approximately 4m (14ft) at the break-in location.

The mainline tunnels A, B/C, and D run beneath Sunnyside Yard then make a turn to follow the existing LIRR mainline tracks before terminating at a reception pit (A Tunnel), or dead-ending (B/C and D Tunnels). The tunnels rise at a maximum gradient of 3.25% throughout their length, and the soil cover diminishes from a maximum of 9m (28ft) below the embankment of the mainline tracks to 4m (12ft) at the end of the drives. The B/C Tunnel was extended by approximately 130m (420ft) after contract award by maintaining the same rising gradient, which resulted in cover of as little as 1.8m (6ft) at the end of the drive.

It was not possible to construct the Tunnel A reception pit at the location originally planned, because the location was blocked by utilities, and these could not be relocated until a large systems upgrade program was completed by others. As a result, the Tunnel A reception pit was relocated to the north, offset from the future permanent tunnel alignment. This occurred after the TBM had been launched and resulted in a 73m



Tunnel drive	Soil	Rock/mixed	Total
A	585 m 1,919 ft		585 m 1,919 ft
B/C	666 m 2,185 ft		666 m 2,185 ft
D	652 m 2,139 ft	18 m 59 ft	670 m 2,198 ft
Yard lead	1,185 m 3,888 ft	131 m 430 ft	1,316 m 4,318 ft
Total	3,088 m 10,131 ft	149 m 489 ft	3,237 m 10,620 ft

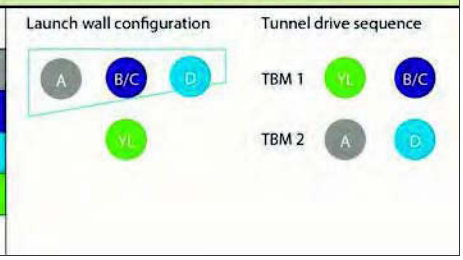


Figure 1. Tunnel alignment

(238ft) long left turn with an uphill grade of 7.5% and a radius of 224m (734ft) that had to be introduced at the end of the tunnel to steer the TBM north towards the relocated reception pit.

The intent at bid time was to recover the Tunnel D TBM at the completion of the drive via a reception pit. Due to challenges with utility relocations in this area the Pit could not be constructed in time for the arrival of the TBM and it was therefore decided to dead-end the TBM.

The tunnels were driven through glacial deposits that were divided into three groups, mixed glacial deposits, glacial till, and outwash/reworked till deposits. These layers are highly variable with a fines content of 5 to 30%, and their classification according to the Unified Soil Classification System (USCS) ranges from well-graded gravel and sand (GW, SW), to silty sands (SM), all the way to low plasticity silts/clays (ML, CL) and even high plasticity clays (CH). At tunnel horizon, the glacial deposits are generally dense with Standard Penetration (SPT) blow counts of mostly over fifty. Overlaying the glacial deposits is a layer of fill material consisting of very loose to very dense sands with silts/clays, gravel and miscellaneous debris. Fill material was thought to have been encountered only twice in the tunnel horizon, both occurring in Tunnel A. The first potential encounter was as the TBM approached the Honeywell Bridge foundation in the first half of the drive, and the other encounter was towards the end of the drive.

Excavation of the lowest tunnel, the Yard Lead, began with a full face of bedrock comprised of strong to very strong gneiss and schistose gneiss with an unconfined compressive strength (UCS) of up to 186MPa (27,000psi). The rock mass ranged from slightly weathered to un-weathered, with Rock Quality Designation (RQD) values varying from 17 to 100%. Full-face rock conditions lasted for approximately 110m (350ft) before transitioning through mixed face conditions for about 25m (80ft) into glacial till.

Tunnel D started with approximately 24m (60ft) of mixed face while Tunnel A and B/C encountered small amounts of rock in the invert during the first 6m (20ft).

Based on excavated fragments, cutter performance, and excavation for approach structures, boulders of up to several feet in diameter were likely encountered throughout the drives. Nested Boulders and cobble clusters were typically found just above the top of the bedrock. Tests performed on cores taken from boulders revealed an unconfined compressive strength (UCS) of up to approximately 372MPa (54,000psi). A boulder the size of a small car was found in the Yard Lead approach structure.

The ground water table is typically 1.5 to 3m (5 to 10ft) below the surface of Sunnyside Yard, and 8 to 9m (25 to 30ft) below the surface of the mainline track embankment. The tunnels intersected plumes of contaminated ground water containing VOCs and hydrocarbons. As a result of this, maximum allowable ground water drawdown was limited by the contract to avoid migration of the plumes to other areas. The contaminated ground prompted OSHA to classify the tunnels as “potentially gassy.”

GEOTECHNICAL INTERPRETATION AND DEVELOPMENT OF TUNNEL OPERATION PARAMETERS

The tunnels were driven in close proximity to sewers, bridge footings, retaining walls, sensitive railroad infrastructure, and beneath rail yards as well as the mainline tracks that comprise Harold Interlocking with, in some places, less than one diameter of cover. Minimizing ground movements and surface settlements due to tunnel excavation were therefore key to the successful completion of the contract. On top of that, the TBMs had to mine below the water table through highly variable glacial till and outwash deposits, mixed face conditions with bedrock in the invert composed of strong to very strong gneiss. As a result of this, performing a thorough analysis of the tunnel operation parameters such as confinement pressures required to support the tunnel face and backfill grout pressures to ensure filling the annular void was of paramount importance.

The tunnel operation parameters were derived from the information on the ground conditions given by the Owner. As part of the contract documents, with the Geotechnical Data Report (GDR), the Owner issued over 300 logs of borings drilled within the project site, along with data obtained from field and laboratory testing. In addition, the contract included a Geotechnical Interpretative Report (GIR), which provided an interpretation of the available geotechnical data, a description of the anticipated subsurface conditions and ground behavior, as well as design parameters.

Confinement and Backfill Grout Pressures

In the past, different models for determining the confinement pressures to be applied to the face of tunnels driven in cohesionless soils, such as the glacial deposits prevailing across the project site, had been developed by various consultants. The Contractor decided therefore to have the confinement pressures analyzed using more than one method, and to compare the results. In total, four analyses were performed, which were based on: a. Leca and Dormieux, b. Ron Heuer using Proctor and White, c. Mohkam and Wong, and d. German Standard DIN 4085. The latter three methods were limited to the analysis of the face stability based on a certain factor of safety, and did not consider ground deformations. The Leca and Dormieux analysis however, took into account surface settlements that were analyzed following the Convergence-Confinement Method by M. Panet using a finite element software. The confinement pressures along with the settlements were calculated iteratively, until the settlements above tunnel axis had reached the target value of 13mm (0.5in). In all of the seventeen sections analyzed, the target settlement value, not the face stability, was the governing factor. The sections

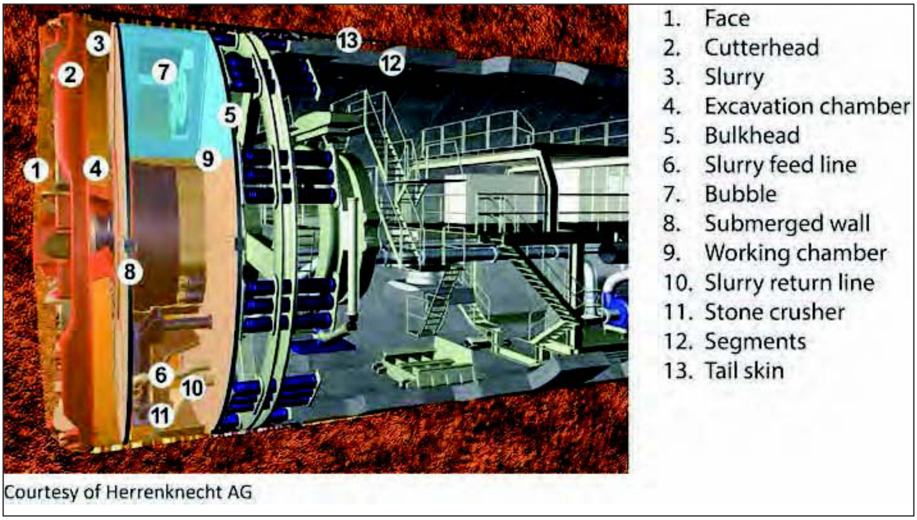


Figure 2. Slurry TBM configuration

were selected based on ground conditions, soil cover, and the structures located above the tunnel. Six sections were analyzed for the Yard Lead (YL) Tunnel, four each for the A and D Tunnels, and three for the BC Tunnel. The face stability was analyzed for collapse and fracturing/heave, which resulted in a minimum and a maximum allowable value for the confinement pressures.

The Slurry TBMs utilized to excavate the tunnels were designed on the principle of having an air bubble in the plenum trapped behind a submerged wall separating the excavation from the working chamber as shown in Figure 2. With such a system, the confinement pressures at the face are controlled by the pressure of the air bubble (bubble pressure). An air regulating system on the TBM maintains the bubble pressure at a pre-set value. It senses the actual pressure of the bubble and instantly initiates the necessary adjustments to match the desired pressure by automatically opening or closing of air supply and exhaust valves.

The confinement pressures were analyzed for tunnel crown elevation, and needed to be converted to bubble pressures for use in the field, with the level of the slurry in the working chamber and the density of the slurry being the variables. The level of the slurry in the working chamber was usually kept around spring-line, and the density of the slurry fluctuated in general between 1.1 and 1.3tons/m^3 (69 to 81 lb/cft). The slurry pressure at the interface with the air bubble in the working chamber corresponds to the pressure in the excavation chamber at the same elevation, and the density of the slurry determines the pressure gradient across the face. As a result of this, for a fixed bubble pressure, the confinement pressure increases in invert and crown when raising the slurry level in the working chamber. Conversely, the confinement pressure decreases in the crown and increases in the invert with increasing density of the slurry if the slurry level in the working chamber is maintained at spring-line level. These phenomena are illustrated in Figure 3.

Backfill grouting was performed through the tail shield, simultaneously with TBM advance, via two ports installed in the upper half, one each per quadrant, at around the 2 and 10 o'clock positions. A two-component grout system was used, which consisted of a cementitious and non-sanded grout, mixed with an accelerator at a nozzle immediately before being injected into the tail void. The backfill grout pressures had

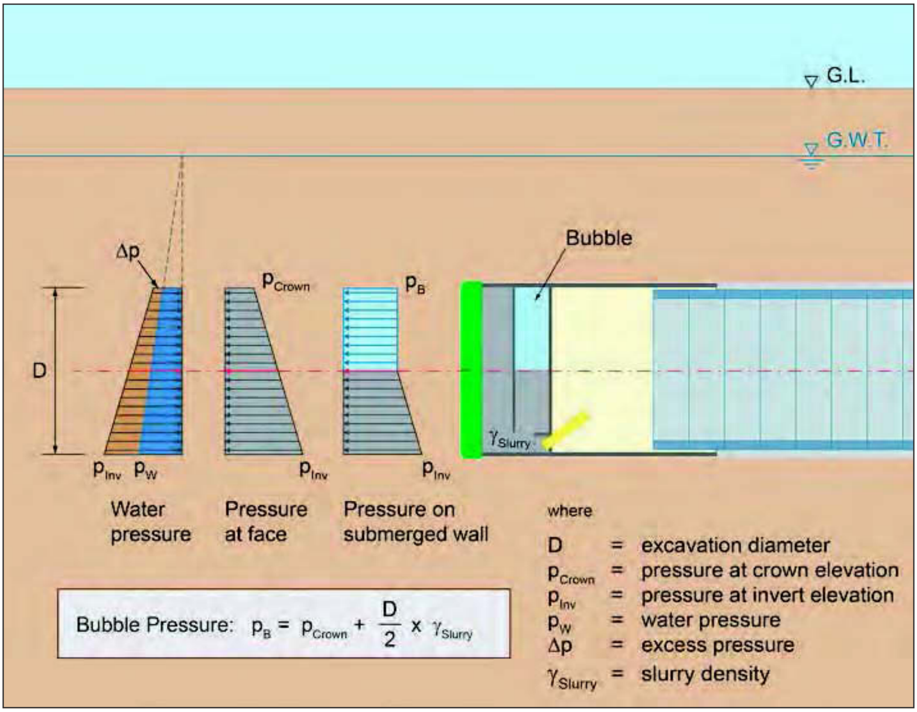


Figure 3. Confinement pressure diagram

to be sufficient for the grout to flow around the annular gap of the lining, and greater than the slurry pressure, to prevent the slurry in the Cutterhead from migrating through the overcut around the shield and into the tail void. The maximum allowable backfill grout pressures were analyzed considering the overburden and the shear strength of the ground, as well as the capacity of the tunnel liner. Based on previous experience, the Contractor set the target grout pressure at 2bar (29psi) above bubble pressure. The results of the backfill grout analysis confirmed that sufficient safety margin against blow-out was available with the exception of a few areas where the cover was less than one diameter. For these isolated cases, the grout pressures were lowered to 1.5bar (22psi) above bubble pressure.

The results of the confinement pressure analysis for all four of the methods used were tabulated in a spreadsheet, and the minimum required face support pressure compared with the hydrostatic head. The difference between the two, defined as Delta P, was then compared among the four methods used in the analysis. The Delta P values varied significantly, with the Leca/Dormieux method being on the high end with numbers of up to 1.1bar (16psi), and Proctor/White as well as Mohkam/Wong on the low end with numbers as low as 0.1bar (1.5psi). The Contractor then determined the minimum confinement pressures for every 50 linear feet (15m) of tunnel selecting a Delta P varying between approximately 40 and 90% of the Delta P obtained with the Leca/Dormieux method. The bubble pressures were then derived from these confinement pressures based on a slurry density of 1.3t/m^3 (81lb/cft) and a slurry level in the working chamber located at spring-line. Figure 4 depicts and sample of the spreadsheet.

The confinement pressures and backfill grout pressures that were applied later in the field proved to be adequate. The surface settlements measured were consistently below predicted, and ranged from 3 to 8mm (0.1 to 0.3in), with an isolated case where

Bubble Pressure for B/C Tunnel TBM

data		
Tunnel S/L to T/R	6.17 ft	74 in
TBM crown to Tunnel S/L	11.26 ft	3432.5 mm
pressure	14.50 psi	1.0 bar

				Start VC							End VC				BC1
station		ft	1182+29	1182+35	1182+50	1183+00	1183+50	1184+00	1184+50	1185+00	1185+35	1185+50	1186+00	1186+50	1187+00
geology															
surface elevation		ft	320.0	320.0	320.0	320.0	320.0	320.0	320.0	320.8	320.6	320.5	320.7	321.7	321.0
ground water elevation	high	ft	315.0	315.0	315.0	315.0	315.0	315.0	315.0	315.0	315.0	315.0	315.0	315.0	315.0
	low	ft	311.0	311.0	311.0	311.0	311.0	311.0	311.0	311.0	311.0	311.0	311.0	311.0	311.0
track elevation		ft	275.4	275.60	276.08	277.57	278.90	280.06	281.05	281.88	282.36	282.55	283.18	283.80	284.43
grade			3.25%	3.25%	3.15%	2.82%	3.30%	2.15%	1.82%	1.49%	1.25%	1.25%	1.25%	1.25%	1.25%
tunnel S/L elevation		ft	281.57	281.77	282.25	283.74	285.07	286.23	287.22	288.05	288.53	288.72	289.35	289.97	290.60
TBM crown elevation		ft	292.83	293.03	293.51	295.00	296.33	297.49	298.48	299.31	299.79	299.98	300.61	301.23	301.86
TBM invert elevation		ft	270.31	270.51	270.99	272.48	273.81	274.97	275.96	276.79	277.27	277.46	278.09	278.71	279.34
tunnel cover		ft	27.17	26.97	26.49	25.00	23.67	22.51	21.52	21.49	20.80	20.52	20.09	20.47	19.14
hyd head (high) - crown		ft	22.17	21.97	21.49	20.00	18.67	17.51	16.52	15.69	15.21	15.02	14.39	13.77	13.14
		m	6.76	6.70	6.55	6.10	5.69	5.34	5.04	4.78	4.64	4.58	4.39	4.20	4.01
		bar	0.66	0.66	0.64	0.60	0.56	0.52	0.49	0.47	0.45	0.45	0.43	0.41	0.39
hyd head (high) - invert		ft	44.69	44.49	44.01	42.52	41.19	40.03	39.04	38.21	37.73	37.54	36.91	36.29	35.66
		m	13.62	13.56	13.42	12.96	12.56	12.20	11.90	11.65	11.50	11.44	11.25	11.06	10.87
		bar	1.34	1.33	1.31	1.27	1.23	1.20	1.17	1.14	1.13	1.12	1.10	1.08	1.07
hyperbaric		psi	20	20	20	19	18	18	17	17	17	17	16	16	16
delta P - chosen		bar	0.60	0.61	0.62	0.67	0.70	0.64	0.67	0.59	0.61	0.61	0.63	0.65	0.67
crown pressure		bar	1.26	1.27	1.26	1.27	1.26	1.16	1.16	1.06	1.06	1.06	1.06	1.06	1.06
slurry density		kg/L	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
bubble pressure		bar	1.70	1.70	1.70	1.70	1.70	1.60	1.60	1.50	1.50	1.50	1.50	1.50	1.50

					BC1	
Expert's Figures	crown pressure	Panet	bar	Panet	1.50	bar
		HK	bar	HK	0.94	bar
		Mokham	bar	Mokham	0.60	bar
		Heuer	bar	Heuer	0.62	bar
	delta P	Panet	bar	Panet	1.11	bar
		HK	bar	HK	0.55	bar
		Mokham	bar	Mokham	0.20	bar
		Heuer	bar	Heuer	0.23	bar
	delta P	max	bar	max	1.11	bar
		min	bar	min	0.20	bar
		% range		% range	52%	
		% Panet		% Panet	60%	

Figure 4. Bubble pressure spreadsheet

the settlements reached 15mm (0.6in). The mainline tracks settled by a maximum 6mm (0.25in), which was well below the allowed maximum of 32mm (1.25in). The actual Volume Loss was in the range of 0.2 to 0.5%.

For interventions into the Cutterhead in compressed air, the bubble pressures in the table were used for setting the air pressure inside the Cutterhead. This resulted in face support pressures significantly higher than required to balance the water pressure, particularly in the crown. However, prior to each intervention a stable and tight filter-cake was formed at the face using fresh bentonite slurry, also called “mother mud,” to reduce air losses and to improve the stability of the face. As the job progressed, this approach proved to be the appropriate one to use in the field, and settlements during compressed air work were kept within the same margin experienced during regular advance. Also, the air losses were manageable, and forming a new filter cake (“re-caking”) when the existing one had dried out (which resulted in increased air loss) was necessary a maximum of only once a shift, even at the most adverse locations in terms of ground permeability and cover. This re-caking was usually carried out at the end of the shift to minimize down-time.

Slurry Parameters

The two consultants, Marc Panet and Michel Mohkam, were tasked with analyzing the ideal parameters for the slurry. In terms of face stability, the Yield Point, Plastic Viscosity, and the Filtrates were the relevant parameters. The target Yield Point and Plastic Viscosity were established by analyzing the slurry penetration into the ground and verified with historical data from previous slurry TBM projects in similar ground conditions. The two experts recommended for the Yield Point a range of 6 to 10Pa, with the exception of areas where a higher percentage of fines were expected. In these more silty and clayey areas, a Yield Point of as low as 4Pa was deemed acceptable. The Plastic Viscosity had to be equal to or less than 10 centipoise (1cP = 1mPa·s) according to the experts. The desired range for the Filtrates was determined numerically based on the permeability of the ground, average pore size, slurry properties, and anticipated flow characteristics of the slurry at the face. The maximum allowable number recommended by the two experts varied, with Panet setting it at 19ml and Mohkam stipulating 30ml. GTF decided to adhere to the lower of the two values.

Soil boring samples, along with several bentonite product samples from various suppliers in the Wyoming area, were sent to the slurry treatment plant manufacturer’s lab in France for testing in order to find suitable bentonite types which would enable us to produce slurry capable of meeting the desired parameters. In addition, cores taken from jet-grout blocks and concrete walls through which the TBMs would later be advancing were sent to the lab for testing so as to determine the resilience of each bentonite type against cement contamination.

Three bentonites were found suitable, with one achieving the best results with regard to cement contamination. The bentonite used ended up being supplied by Laviosa/MPC from a mine located in Wyoming’s Big Horn Basin, and consisted of naturally occurring sodium bentonite treated with additives to meet the specific requirements of the job.

During tunneling, the mother-mud was batched at a concentration of 70kg per cubic meter of slurry and stored in a 1,000m^3 open tank. Concurrently with each advance, mother-mud was added to the slurry as needed to achieve the desired properties. In addition, water was added to regulate the density of the slurry. The target density of the feed slurry was set at 1.15t/m^3 with some exceptions, for instance when mining through ground with a high content of fines, where 1.18t/m^3 was acceptable in order to keep the amount of surplus slurry within a range manageable by the slurry treatment plant. During each advance, a sample was taken from the slurry and tested for rheological properties, filtrates, density, and pH after it had passed through the separation and

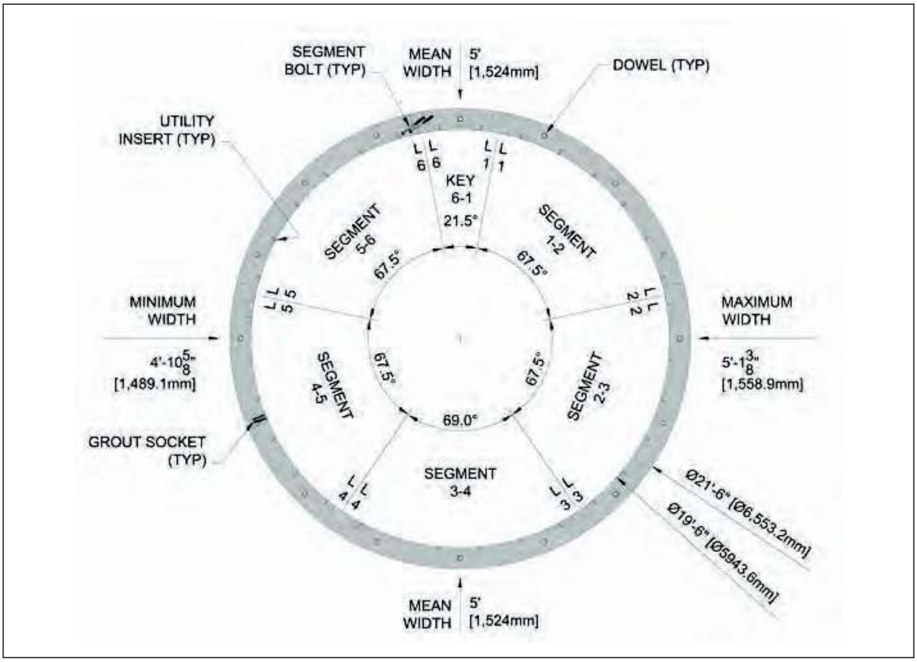


Figure 5. Tunnel segmental liner

treatment process, but before it was fed back to the TBM. Adjustments, such as the adding of mother mud and/or water were made to the slurry, based upon the results of these tests. In addition, the slurry was tested for sand content, which preferably had to be kept below 2% in order to maintain its ability to form a good filter cake at the face.

TBM Operating Parameters

On the TBM, engineers would set the bubble pressure and grouting pressure limits for mining operations in accordance with the analysis. For compressed air Cutterhead interventions, the same analysis was used for setting the Working Chamber Pressure. This analysis proved to be paramount to the success of tunnel operations, and would be later tested at the end of the BC Tunnel, where an extension was added and the TBM successfully mined under mainline rail tracks with as little as 2m (6ft) of cover.

TUNNEL SEGMENTAL LINER

Precast tunnel liner segments were originally designed by the Owner's engineer (Parsons Brinckerhoff, STV and Parsons Transportation Group) and the design was modified jointly by the Contractor and Halcrow. The segments were manufactured in Pennsylvania by Technopref of Quebec, Canada. Upon approval of the modified design, the Owner's engineer adopted the 'designer of record' status of the design. The tunnel liner was 5,944mm (19ft-6in) internal diameter, 6,553mm (21ft-6in) outer diameter, and 1,524mm (5ft) wide, with a ring taper of ±35mm (1.375in) for 244m (800ft) radius tunnel curves. Segment Molds were manufactured by CBE, gaskets were from Phoenix and hardware (bolts, dowels and inserts) were from Sofrasar. Left and Right rings were used to keep the key in the upper half of the tunnel. The segmentation is shown in Figure 5.

Herrenknecht TBM Specifications - S-558 & S-559

Queens Bored Tunnels

Description				Imperial Units	Metric Units
TBM Cutterhead	Diameter			270.3 in	6865 mm
				22.5 ft	
	Drive	Type		Hydraulic	
		Installed Power		2681 HP	2000 kW
		Torque		3,540,345 lb.ft	4800 kNm
		RPM range		0 to 6.5 rpm	
	Cutters	Type		Single Disc Monoblock	
		Size		17 in	432 mm
		Qty	Single	34 no.	
			Quads	4 no.	
			Spare	1 no.	
	No. Openings			8 no.	
	No. Wear Detectors			5 no.	
Slury Circuit	Slurry Pump	Manufacturer		Warman	
		Feed (P1.1)	Type	250 PG PC	
			Motor Size	469 HP	350 kW
		Return (P2.1)	Type	10/8 S GH	
			Motor Size	603 HP	450 kW
		Boost (P2.2) YL	Type	10/8 S GH	
			Motor Size	603 HP	450 kW
	Telescopic Pipe	Travel Length		27 ft	8.23 m
	Tunnel Pipe	Diameter		12 in	305 mm
		Length		20 ft	6.10 m
	Crusher Type			Jaw	
	Grill Plate opening	Original		5.51 in	140 mm
		Modified		4.72 in	120 mm
TBM Shield	Diameter			269.1 in	6835 mm
				22.4 ft	
	Length			34.3 ft	10445 mm
	Weight			500 ton	454 tonne
	Stabilizers	For Rock	Qty	2 no.	
	Thrust	Jack Qty		16 pairs	
		Force per Jack Shoe (Jack Pair)		294 ton	267 tonne
		Total Force (all jacks)		4714 ton	4277 tonne
		Qty Pressure Groups		6 groups	
		Jack Stroke		86.6 mm	2200 mm
		Stroke Measuring Device	Type	Temposonic	
			Qty	4 no.	
	Articulation	Type		Active	
		Jack Qty		12 no.	
		Qty Pressure Groups		3 no.	
		Jack Stroke		9.84 in	250 mm
		Stroke Measuring Device	Type	Temposonic	
			Qty	4 no.	
TBM Trailing Gear	Total Length			361 ft	110 m
Tunnel Liner	Width			60 in	1524 mm
	Erector	Type		Ring	
		Segment Pickup Mechanism		Vacuum	

Figure 6. Slurry TBM general specifications

SLURRY TUNNEL BORING MACHINE

Herrenknecht manufactured two Slurry Tunnel Boring Machines (TBMs), with a cut diameter of 6,865mm (22ft-6in). The supply scope also included the design and procurement of the Slurry Circuit components. During the design phase, Pierre Longchamp (former Technical Director of Bouygues) helped the Contractor with various technical aspects of the TBM and Slurry Circuit design. Some general specifications of the TBM are listed in Figure 6, and the general layout of the shield is shown in Figures 7 and 8. As described previously, Tail Shield Grouting was employed, similar in design to what was used on the Los Angeles Metro Gold Line Eastside Extension Project (refer to 2007 RETC paper by Robinson and Bragard for more details).

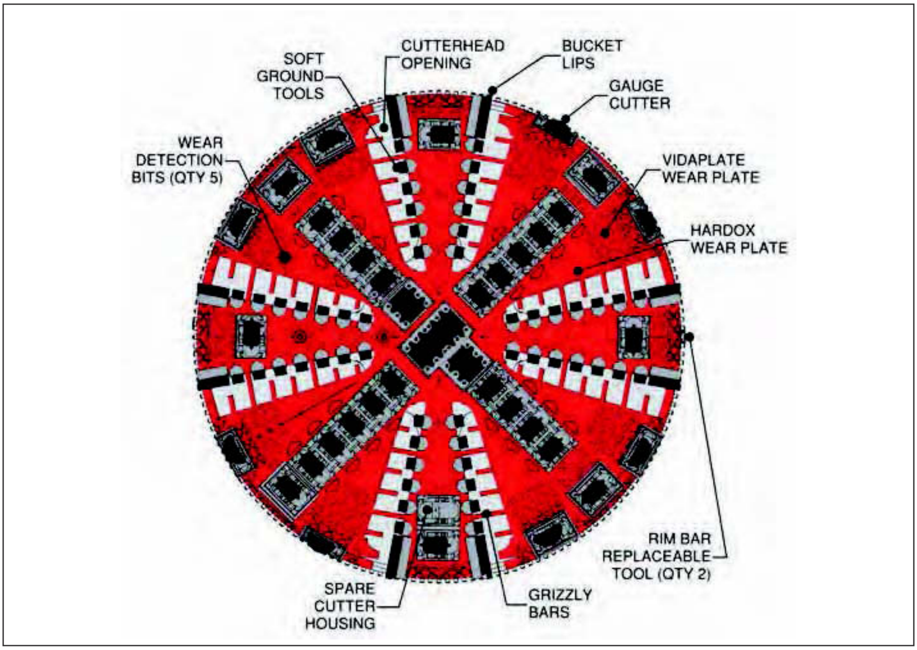


Figure 7. Herrenknecht TBM cutterhead

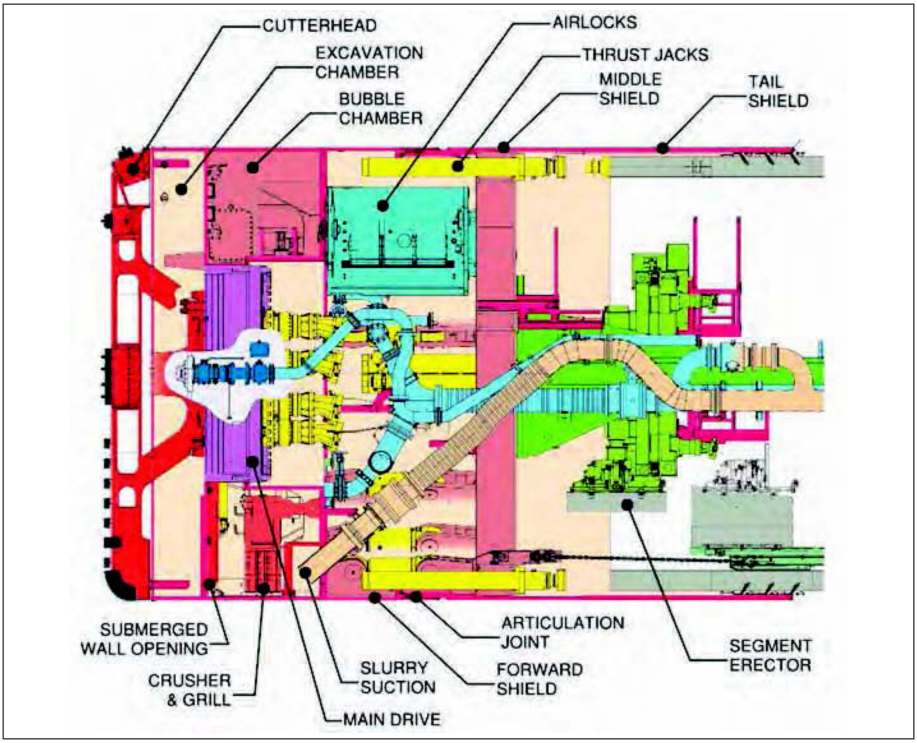


Figure 8. Herrenknecht slurry shield

Figure 9. Crusher grill plate modifications

12in (305mm) nominal diameter schedule 40 pipes were used for conveyance of the slurry. Telescopic pipes on the TBM Trailing Gear allowed the TBM to mine approximately 8.2m (27ft) before a 20-foot pipe required installation. A bypass system, located at the portal, was remotely controlled from the TBM Trailing Gear, and was used to isolate the feed and return slurry lines, and remove slurry from the pipes between the bypass and the TBM, so that when pipe joints were broken for pipe extension, slurry was not lost. The system worked extremely well. On the TBM, there was a return Slurry Pump (P2.1), and for the upper tunnels (A, BC and D) this was capable of transporting the slurry all the way to the Slurry Treatment Plant (STP). The Yard Lead Tunnel required a booster (P2.2) at the portal, and this was installed when the TBM was approximately 50% along in the tunnel alignment. At the STP, there was a feed pump (P1.1) for each TBM.

The TBM was fitted with a larger than normal stone crusher to handle the large strong boulders the TBM was expected to encounter. When the Launch Shaft was excavated, boulders up to 12in diameter were sampled from the muck and sent to the Herrenknecht factory for testing the crusher. The test was productive, as members from the Contractor's team could see that it took many cycles of the crusher to break the boulders down to a suitable size.

Behind the crusher was a set of grill plates which restricted the size of boulder fragments entering the slurry circuit. In the first 100m (300ft) on the Yard Lead Tunnel, there were many cases where it appeared that slightly larger than desired boulder fragments were making their way to the P2.1 pump causing blockages. Before the A Tunnel TBM started mining, and before the Yard Lead Tunnel TBM exited the rock portion of the alignment, a long weekend was utilized to modify the grill plates to decrease the opening from 140mm (5.5in) to 120mm (3.75in). See Figure 9. No more problems with blockages occurred after the change.

Airlocks were fitted to the top of the shield, and a material lock, for transferring cutters and tools, was located at 'spring-line' on the left hand side. The locks had sufficient room for 2 persons in the Auxiliary (or Emergency) Chamber, and enough room for 3 persons in the Primary Chamber. The Bubble Chamber doubled as a staging chamber for workers to organize tools and gear before entering the Excavation Chamber.

The Thrust and Articulation Cylinder pattern was very similar to the Los Angeles Metro Gold Line Eastside Extension (MGLEE), where there was active articulation, and fixed thrust jacks supported at the gland end. As mentioned in the previous section, pairs of grout ports were fitted in the upper quadrants of the Tail Shield similar in design to the system utilized on MGLEE.

SLURRY TREATMENT PLANT

Understanding exactly how a Slurry Treatment Plant works, takes a little bit of time to digest. With the help of MS, from Aubière, France, the Contractor was able to get a better handle on the process and quickly realized that MS were very advanced in understanding the slurry treatment systems for tunnel operations. The MS plant is called a Slurry Treatment Plant, as it not only screens out the muck from the slurry, but it 'treats' the slurry after the screening process to ensure the slurry properties are maintained. There are three simple steps to the Slurry Treatment Plant process. The first step, referred to as 'Scalping,' removes large pieces such as cobbles, gravel and fragments of boulders, or anything greater than 6mm (¼in). To do the scalping, MS supplied a 'Trommel,' which is a self-cleaning, rotating screen. In clays, typical shaker screens can easily plug. See Figures 10 and 11.

Sand and Silt particles ranging from 6mm (¼in) to 63 micron (1⁄16in) are separated using shaker screens and cyclones in the second step of the process which is called 'Desanding.' See Figure 12.

In the third step, a portion of the slurry is wasted in order to remove the fine particles. In order to maintain a volume balance, water and dense bentonite (mother-mud) are injected restoring the slurry properties to desired levels. In the Scalping and

Figure 10. MS STP trommel

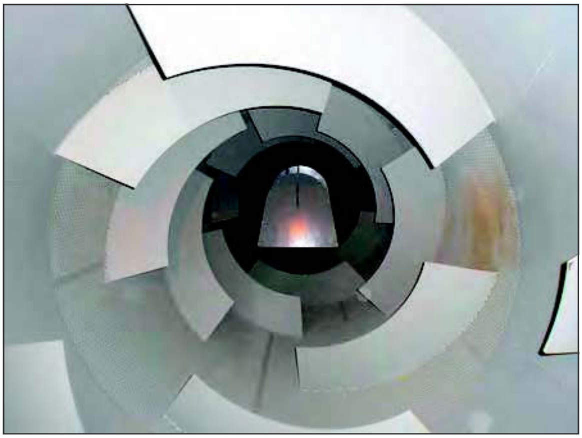


Figure 11. MS STP trommel—view inside

Figure 12. MS STP shaker screens

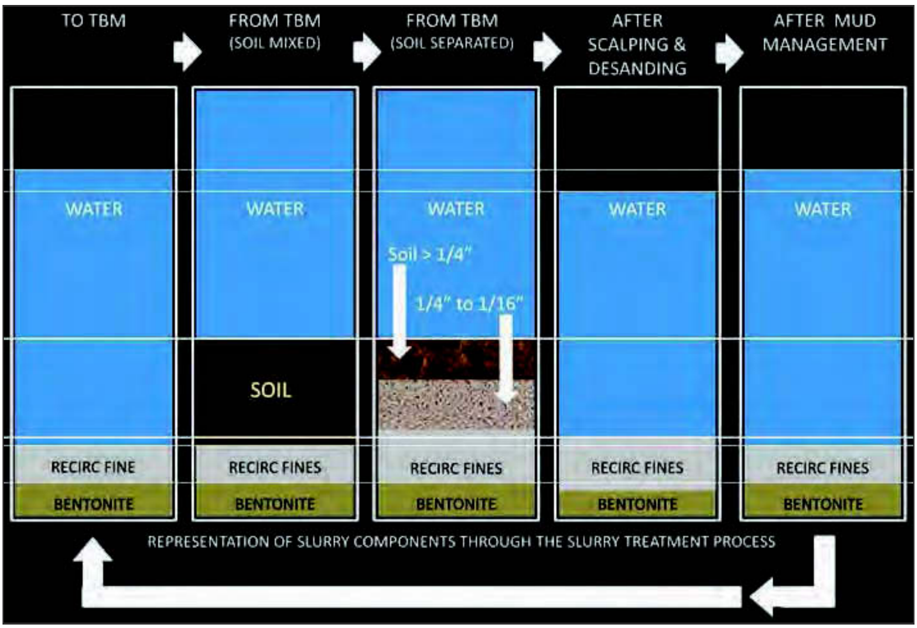


Figure 13. MS STP Mud Management—step by step process

Desanding process, slurry is lost as it coats discarded particles, so there is always more volume added than wasted. Engineers sample the slurry every time the TBM completes a mining cycle, to understand the slurry properties, and provide guidance to the plant operators on quantities to add. The addition of water and mother-mud occurs continuously as the system treats the slurry, so that slurry properties are always maintained. MS have a patent on the system they call 'Mud Management.' Figure 13 shows a simplistic, step by step outline of the treatment process.

The waste mud that accumulates in a buffer tank, is mixed with Lime, and allowed to react (flocculate) for some time in silo tanks, before being sent to the Filter Presses. See Figure 14. MS supplied four Filter Presses, which was probably a little on the high

Figure 14. MS STP filter presses

side, considering the geology was predominantly sandy and possessed very few fines, but there were portions of the Yard Lead Tunnel which passed through full faces of Gardiners Clay, and in these zones, all four Filter Presses were very busy processing the waste mud.

MS provided excellent assistance in the planning stages, and with the assistance of Pierre Longchamp the Contractor was able to size the plant smartly. In most cases, components and volume capacity of the plant were slightly oversized, as it was observed that it was common on other Slurry projects to initially underestimate things, and later be saddled with having to increase capacity. In addition, the plant could be configured in three different manners: for normal mining, for mining through concrete and for hyperbaric work. See Figures 15, 16 and 17.

The STP was two plants in one. Plumbing was arranged so that there was versatility. If the STP responsible for the critical path TBM broke down, then within approximately one hour, plumbing could be rearranged to allow the other STP to come on line. This actually occurred twice.

HYPERBARIC INTERVENTIONS

Hyperbaric Interventions were required for all tunnels. Refer to another paper in these proceedings, which elaborates more on the Engineering Challenges of the project.

Figure 18 shows a summary of the Interventions. Life Support Technologies from Tarrytown, NY helped the Contractor plan the work, developed compressed air tables, conducted training and trial hyperbaric interventions in medical locks, and provided paramedic and lock tending assistance to the crews.

BC TUNNEL EXTENSION

After contract award, the Owner extended the BC Tunnel by approximately 130m (420ft) underneath an existing Long Island Railroad high-speed cross-over switch. Because the same rising gradient was maintained throughout the extension, the cover at the end of the drive ended up being as little as 2m (6ft). The profile of the tunnel within the extended section remained fully in glacial till from start to finish, with the crown barely touching the interface with the fill at the end of the drive. The water table ranged from below invert to tunnel axis, and the Standard Penetration Test (STP) blow counts varied from 24 to 57 in the tunnel horizon, with a significant number of samples that reached 100 due to the sample tube hitting cobbles and boulders (Figure 19).

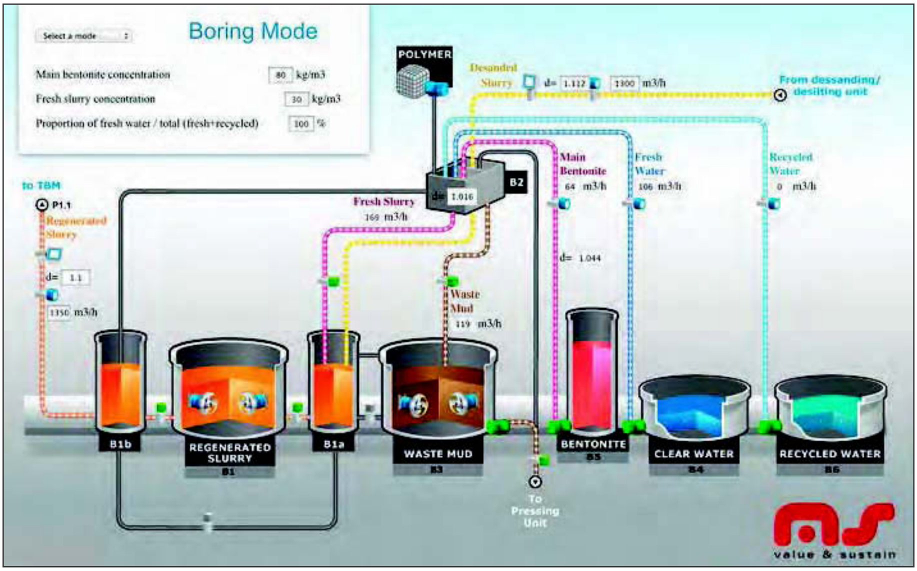


Figure 15. MS STP configuration—–boring mode

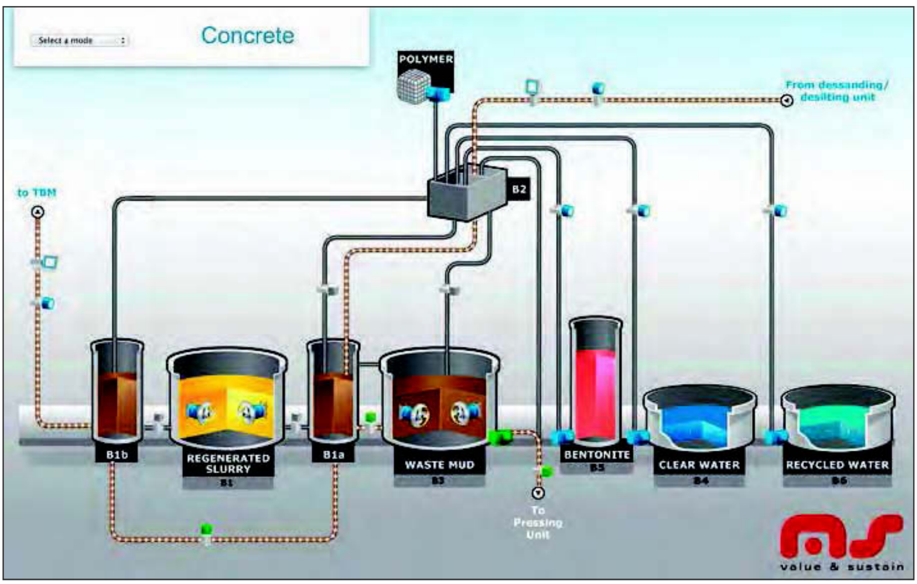


Figure 16. MS STP configuration—concrete

Contractor's Precautions

The Contractor, along with the Owner, developed a risk mitigation and action plan that was aimed at minimizing potential impacts to the operational railroad during TBM advance. The main risks were identified to be ground movements leading to surface settlements, or even sink holes, as well as slurry bursting from the pressurized Cutterhead of the TBM to the surface which is referred to as "frac-out." The following precautionary measures were implemented prior to the arrival of the TBM.

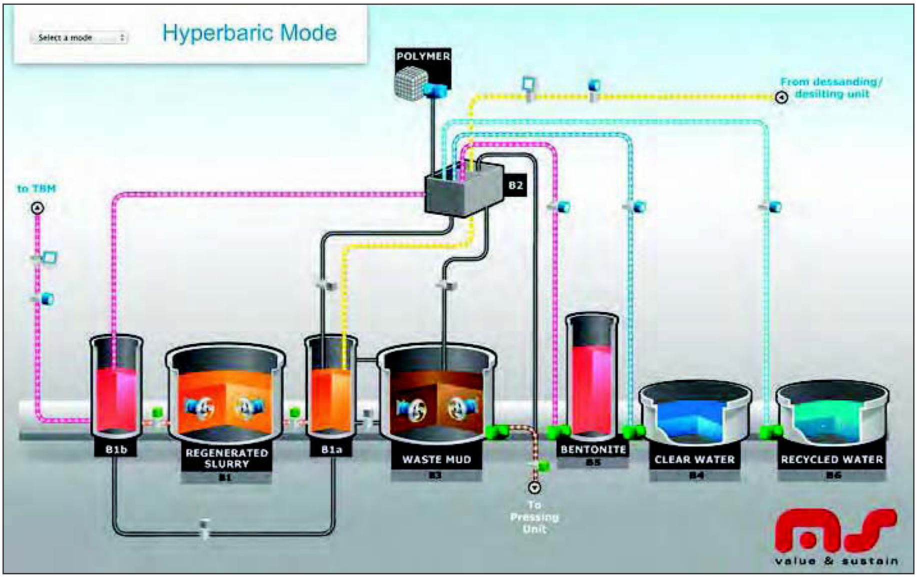


Figure 17. MS STP configuration—hyperbaric

Event	Tunnel	Date		Station	Duration	Pressure		Work performed
		Start	End		[days]	[psi]	[bar]	
1	YL	19-Jul-11	19-Jul-11	1185+69	1	33	2.3	Cutter inspection
2	YL	9-Aug-11	10-Aug-11	1189+79	2	33	2.3	Cutter change
3a	YL	6-Sep-11	12-Sep-11	1191+26	7	32	2.2	Clear blockage in cutterh. in Safe Haven 1
3b	YL	13-Sep-11	23-Sep-11	1191+26	11	32	2.2	Change cutters in Safe Haven 1
4	A	26-Sep-11	7-Oct-11	1189+87	12	14	1.0	Cutter change before Safe Haven 2
5	YL	31-Oct-11	31-Oct-11	1202+44	1	20	1.4	Cutter inspection
6a	YL	10-Nov-11	29-Nov-11	1202+78	20	17	1.2	Clear blockage in cutterh. in Safe Haven 4
6b	YL	30-Nov-11	4-Dec-11	1202+78	5	17	1.2	Cutter change in Safe Haven 4
7	A	5-Dec-11	5-Dec-11	1196+95	1	17	1.2	Cutter inspection
8	A	18-Dec-11	18-Dec-11	1199+20	1	12	0.8	Cutter change
9	YL	6-Jan-12	16-Jan-12	1215+97	11	18	1.2	Cutter change
10	YL	23-Jan-12	23-Jan-12	1217+97	1	18	1.2	Repair of bulkhead door seal
11	D	20-Apr-12	25-Apr-12	1188+86	6	16	1.1	Cutter change
12	D	9-May-12	9-May-12	1196+24	1	16	1.1	Cutter change
13	BC	29-May-12	1-Jun-12	1189+98	4	16	1.1	Cutter change
14	BC	14-Jun-12	19-Jun-12	1194+24	6	15	1.0	Cutter change

Figure 18. Summary table—hyperbaric interventions

The track operated by the Long Island Rail Road and Amtrak (Westward Freight Track) closest to the alignment of the BC Tunnel extension, along with the crossover switch (XO 813) located directly above the tunnel alignment, were taken out of service for the duration of tunneling the extension. In addition, tunnel advance was mandated to be continuous for 24 hours per day, 7 days a week without interruption to eliminate the risk of ground movements during periods of standstill.

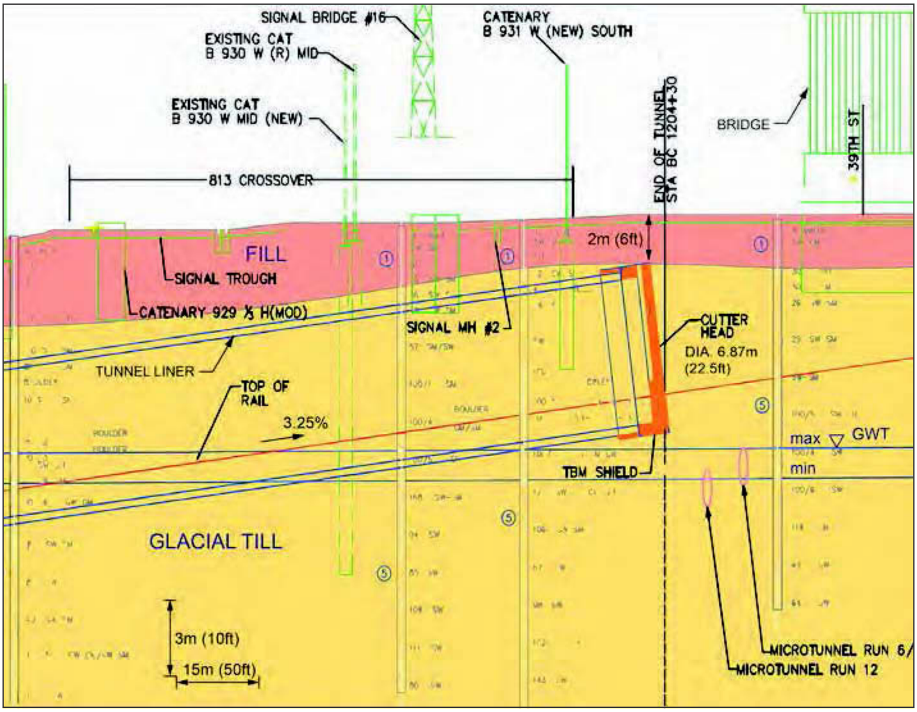


Figure 19. Profile of BC tunnel extension

Supplemental instrumentation was installed, which was comprised of a borehole extensometer, track mounted prisms monitored by automated motorized total stations, and manually read surface settlement monitoring points. A mobile Amberg trolley was pushed along the railroad tracks to measure rail movement. Tunnel convergence monitoring had to be performed at 15-meter (50-foot) intervals. An Action Level Plan was put in place with contingency measures to ensure immediate response to ground movements and/or “frac-outs” of slurry on the surface. In addition, surface spotters were provided around the clock by the Contractor to monitor any disturbances that might occur.

A catenary pole foundation (B929W) was located directly above the extended tunnel. A jacking system was installed to correct any settlement that could occur. Also a 70m (230ft) long precast concrete signal trough (containing 96 cables) running above and parallel to the tunnel alignment was exposed and suspended by straps from a steel beam to isolate the trough from any settlement.

To minimize ground movements and ensure stability of the tunnel lining, secondary grouting through the precast segmental lining of the tunnel had to be performed on every ring in two 90-degree arcs on the left and right of the tunnel, centered about spring-line using the same two-component cementitious grout that was used for primary backfill grouting through the tail shield.

A clean-up system was installed on the surface to contain potential slurry leaks. The system consisted of a 750mm (2.5ft) high silt fence installed around the perimeter of the shallow tunnel section, with four cross track digs, each spaced around 30m (100ft) apart, underneath the Westward and Eastward tracks of the Long Island Rail Road. Six inch diameter PVC pipes connected to a hydraulic pump were laid inside these digs. They had a flexible hose on the end facing the tracks for the purpose of sucking up slurry leaking to the surface. Roll-off containers were provided on the discharge end of

Figure 20. View of surface clean-up system and suspended cable trough

the pumps and vacuum trucks were kept on stand-by to empty the containers as needed, and to haul the slurry off site. In addition, sand bags, trash pumps and hoses were staged in the area as part of the contingency plan.

An exclusion zone was cordoned off on the surface above the TBM, and was advanced with the TBM. Non-critical staff was not permitted to enter the zone.

The target properties for the bentonite slurry, such as Yield Point and Filtrates were raised to improve the stability of the face. Also, the target confinement pressures were examined thoroughly. It was agreed to set the pressure of the air bubble inside the working chamber of the TBM at 0.9bar (13psi) for the entire length of the extension. For the backfill grouting, the cut-off pressure was set at 2bar (29psi) above bubble pressure.

The site with all the precautionary measures in place prior to the arrival of the TBM is shown in Figures 20 and 21.

Figure 21. Cross dig with screw sucker and vacuum truck on the other end

TBM Performance

The TBM arrived at the beginning station of the BC Tunnel extension on June 29, 2012; six days prior to the date crossover switch XO 813 and the Westward Freight Track were scheduled to be taken out of service. As a result, the TBM had to park and wait for the outage.

During the two weeks prior to the TBM's arrival at the Station of the extension origin, advance rates of around 20m (60ft) per day on average had been achieved. Because of the secondary grouting that had to be undertaken throughout the extension, the Contractor anticipated accomplishing lower advance rates, more in the region of 13m (40ft) per day. Thus, the duration for mining the 130m (420ft) long extension was expected to be around 10 days. On July 6, 2012, tunneling operations resumed. The first 24 hours of tunneling went as expected and the TBM advanced around 5m (15ft) per 8-hour shift. After that, the pressure on the suction side of the slurry return pump frequently dropped below bubble pressure, which was an indication that something was blocking the screen on the inlet of the slurry return line inside the working chamber. This necessitated frequent stoppages of the TBM for flushing of the Cutterhead with slurry to free up the blockages.

Pressure readings on one of the rams of the jaw-type stone crusher suggested further that the ram was unable to fully retract and extend as it did normally. With a malfunctioning stone crusher, cobbles and fragments of boulders began accumulating in front of the inlet screen, further restricting the flow of slurry through the Cutterhead. To be able to continue evacuating the muck from the Cutterhead without completely clogging the screen, the advance rates of the TBM had to be reduced from 30 to around 5 to 10mm/min (1.2 to 0.4in/min). As a result, the daily production rate dropped to around 6m (20ft). An intervention into the Cutterhead in compressed air to free up the blockages and to repair the stone crusher was deemed too risky given the low cover and the ground conditions above the TBM.

On day seven, it was decided to raise the bubble pressure in the Cutterhead to 1.1bar (16psi) in order to increase the pressure on the suction side of the slurry return pump. This immediately improved the flow of muck through the screen on the inlet of the slurry return line, and the advance rates of the TBM jumped up to around 20mm/min (0.8in/min). For the next two days the production increased to around 12m (40ft) per day. In the meantime the cover had diminished to less than half the TBM diameter, the bubble pressure was lowered to 1.0bar, and then further to 0.9bar, to reduce the risk of slurry migrating to the surface.

During the following seven days, production rates of 7 to 10m (20 to 30ft) per day were achieved. On the morning of July 23, 5m (15ft) short of the target end station, the spotters noticed slurry leaking to the surface and tunneling operations were ceased immediately (Figure 22). The bubble pressure was lowered to 0.5bar (7psi) to prevent more slurry from leaking to the surface. The slurry containment system proved to be working, and the spill was cleaned up in a timely manner without causing disruptions to the morning train commute. Only a minor amount of slurry spilled over to the Westward Freight Track, which was quickly contained and cleaned up.

At the stoppage location of the TBM, the cover to the Cutterhead was approximately 2m (6ft). The next day, the Cutterhead was completely filled with cement grout and the TBM abandoned in its final position. Crossover switch XO 813 and the freight track were taken back into service a few days later to the full satisfaction of the Long Island Railroad. In the following weeks, the shield of the TBM was completely gutted out on the free-air side of the bulkhead, and the trailing gear retracted to and removed from the launch shaft. A future contract will build a cut and cover approach structure.

Backfill grout operations in general went as planned. The 2-component grout injected simultaneously through two ports embedded in the tail-shield in the upper quadrants, proved to be adequate for tunneling in soft ground with low cover. The average grout take per ring, including secondary and tertiary grouting for the extension, was 5,590L (1,477gal) versus the theoretical volume of 4,680L (1,236gal). Secondary grouting through the segments ended up being 134L (35gal) per ring on average. After review of the results of the Secondary Grouting, the Owner ordered the Contractor to perform Tertiary Grouting on 18 of the total 82 rings installed within the extension of the tunnel.

The ground movements on the surface monitored directly above the TBM throughout the last 100m (300ft) of mining varied from 6mm (0.25in) settlement to 35mm (1.375in) heave. Heaving of the ground was mainly experienced in the area where

Figure 22. Slurry spill on the surface in the morning of July 23, 2012

the bubble pressure had been temporarily raised above 0.9bar (13psi). The maximum settlements of the crossover switch XO 813 tracks were in the order of 5 to 8mm (0.2 to 0.3in).

CONTROL NETWORK AND DATA ACQUISITION

Overview

Tunneling equipment and office computers were all connected via a network. On each of the two Slurry TBMs, there were two Programmable Logic Controllers (PLCs) and five PCs, all of which were Human Machine Interfaces (HMIs). Three HMIs were for Herrenknecht's Operator Screens, one HMI was for Tachibana's Annular Backfill Grout System, and one HMI was shared for VMT's Tunnel Guidance and CBP (Irus) Data Acquisition. On each of the TBMs, there were two PLCs, one for the Herrenknecht systems and one for the Grout System. In the Slurry Plant, in the each of the two Operators Cabins, there were two HMI PCs which powered four display/interface screens, of which one was a 42" large flat screen. There were two HMI's for the four Filter Presses, two HMI's for the Bentonite Mixers and Lime Reactors, and an HMI for the Main Pump House. There was one PLC that controlled all of the Slurry Plant. For the two Grout Plants on the Surface, there was an HMI and PLC for each. Two Cooling Towers also had a PLC each.

Spread throughout the site, were read-only terminals which displayed the real time operation of the tunneling systems. In the Contractor's Main Office, there was a VMT CBP (Irus) Computer Interface and an array of eleven flat screen terminals (one 42", six 27", and four 19"), configured to show the operation and data trending of all tunneling systems. See Figure 23. In addition, 4 Servers were located in a safe, designated, air conditioned room. Four terminals were located in the Contractor's Superintendent's Trailer, four in the Owner's Site Inspectors Trailer, and four in the Owner's Main office five stories above, overlooking the site.

Eight cameras were located in the Slurry Plant, and there was provision made to install four cameras on each TBM, but these were never installed. Each piece of tunneling equipment communicated on a dedicated ControlNet network, via remote ControlNet racks, independent from Ethernet traffic.

Fiber optic cable was used to link all the HMI's, PC's and PLC's to the Servers. The fiber-optic cable was specifically constructed for the project, and contained two pairs of 'single-mode' optical fiber for Ethernet and camera signals, and a pair of 'multi-mode'

Figure 23. Main site office—system control center

optical fibers for PLC control signals. The cable was constructed with a Core-Locked Indoor/Outdoor PVC Jacket and fitted with Military style robust screw connectors.

Via Virtual Private Network (VPN), through the internet, Engineers, Managers, Technicians, and other interested onlookers could access the network and monitor progress and troubleshoot problems.

Wonderware Galaxy Database Manager

The design and layout of the HMI screens were jointly developed by the equipment manufacturers and the Contractor's staff. Wonderware Galaxy Database Manager software was used which provided a very efficient means to manage, update and maintain the HMI's. With Galaxy Database Manager, the design and configuration of each HMI could be stored in a 'Galaxy Repository Server' (GR Server). If the system required update, or fixing, the Technician would open the GR, make the changes and deploy the updates whilst the system was running. With over 40 HMI's to manage, some of which shared similar graphics, the task was made very manageable. In addition, redundancy and energy sharing was setup for the system, to ensure equipment could still communicate if a single computer were to breakdown. All computers on the equipment had local database engines installed so that the system could function while being connected to the GR Server.

In the developmental stage of the project, each manufacturer developed their own Galaxy, using the Contractor's guidelines, and once all equipment reached the job site, the Galaxy's were combined to make a larger job wide Galaxy.

GEOTECHNICAL MONITORING AND INSTRUMENTATION

The fact that the tunnels were driven in close proximity to sewers, bridge footings, retaining walls, sensitive railroad infrastructure, and beneath rail yards as well as the mainline tracks that comprise Harold Interlocking with, in some places, less than one diameter of cover, minimizing ground movements and surface settlements due to tunnel excavation was critical to the successful completion of the contract. To monitor the effects of tunnel construction, a comprehensive array of instrumentation was installed on structures, catenary poles, and in the ground. The majority of the instrumentation was installed under a separate contract, and additional instrumentation was included in Contract CQ031. For Sunnyside Yard, monitoring of the tracks was performed manually, as automatic monitoring would have been difficult due to trains stored on the numerous yard tracks. For the mainline tracks, monitoring was performed in real-time using automated motorized total stations (AMTS) sighting prisms bolted to the base of the rails. In-ground instrumentation consisted of inclinometers, borehole extensometers, and piezometers, with a cluster of instrumentation (TBM monitoring zone) provided around 120m (400ft) into the drives so as to obtain data to validate assumptions made in the TBM confinement pressure analysis.

Catenary poles, signal bridges and towers were monitored using automatically read tiltmeters. Instrumentation for other structures such as bridge piers, substations, and miscellaneous railroad infrastructure consisted of either prisms read in real-time by AMTS, or manually read structure monitoring points. The monitoring data was stored electronically on a server, and was available to multiple users for viewing via the local network or the Internet. Real-time monitoring data was therefore available to the site personnel almost instantly. In addition, the software program was set up such that email messages were sent out automatically to a select number of staff whenever an instrument reading exceeded the Alert Level. The monitoring data was also integrated into the Controlled Boring Process CBP System from VMT GmbH that had been set up on the job-site. This system allowed the engineers to correlate instrument readings with TBM operational data and the current location of the TBMs. A typical screenshot of the CBP System is shown in Figure 24.

Response Levels for track movements were set up in accordance with the requirements of the Federal Railroad Administration (FRA). This resulted in establishing an Alert Level for the mainline tracks of 38mm (1.5in), with the exception of some high-speed switches where it was less. The movement limits for catenary poles had to be established on a case-by-case basis, as they depended highly upon the direction and magnitude of movements the tracks adjacent to the poles had experienced. For buildings, bridges, and retaining walls, the Alert Level was set to be 25mm (1.0in) per the contract. A comprehensive Action Level Plan was developed by the Joint-Venture, and some critical structures, such as bridge foundations, required some site specific plans to be established.

Actual settlement values ranged from 3 to 8mm (0.1 to 0.3in), with an isolated case where the settlements reached 15mm (0.6in). The mainline tracks settled by a maximum 6mm (0.25in), which was well below the predicted.

CROSS PASSAGES

In total three cross passages connecting the bored tunnels to the emergency exit structure shafts were included in the original contract work. Due to a redesign of the emergency egress and ventilation system, after contract award, the Owner eliminated two of them, leaving only one, which was located in the longest of the four tunnels, the Yard Lead, approximately halfway between the open-cut and the approach structure. The Yard Lead cross passage consists of a 1.5m (5ft) wide emergency exit and two 3m (10ft) wide ventilation openings, each with a length of approximately 3.7m (12ft). The excavation profile was entirely within the glacial till, with a cover of around 7m (24ft). The water table was approximately 4m (14ft) below the surface and 3m (10ft) above the crown of the excavation.

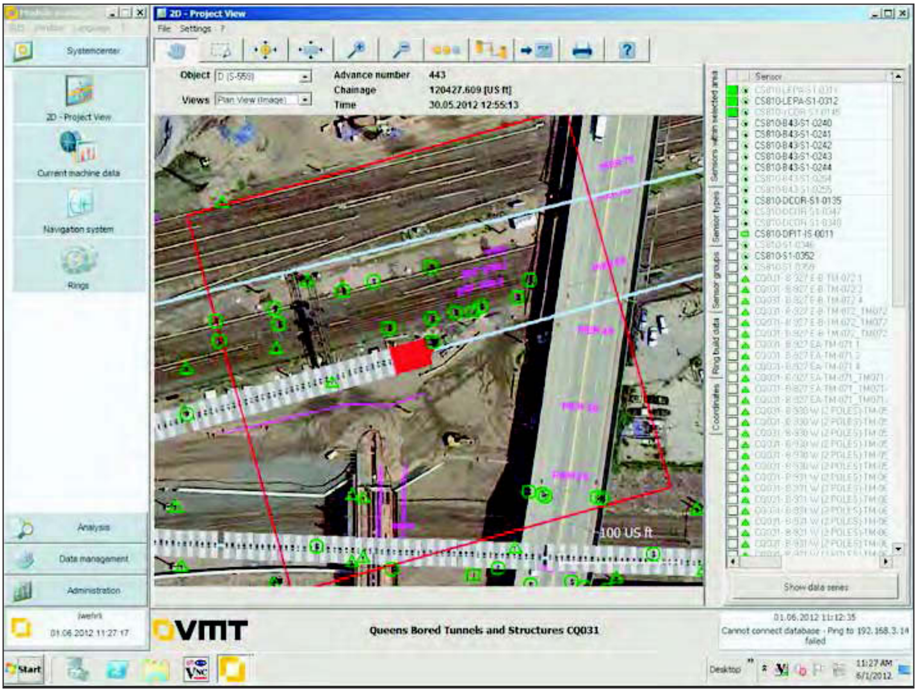


Figure 24. Screen shot of VMT CBP system

Because excavation had to be accomplished in close proximity to existing tracks, bridge piers, and bridge abutments, ground treatment around the cross passage was mandatory per the contract. The Contractor concluded that jet grouting would be the most suitable method given the properties of the soil. It was decided to extend the ground treatment across the full perimeter of the bored tunnel and to install it from the surface, prior to TBM boring, so as to also provide a "safe haven" for the purpose of inspecting and changing tools on the TBM Cutterhead in free-air.

The suggested method of construction per the Owner's contract drawings consisted of excavation in stages, in conjunction with steel sets, wood lagging support, and a cast-in-place permanent liner installed after completion of excavation. By designing a temporary support system for the tunnel segmental liner at the openings, consisting of vertical braces spaced at 1.5m (5ft) centers, the Owner assumed that the cross passage would be driven from the shaft towards the tunnel. At bid time however, the Contractor decided to opt for a more economical design that would allow excavation of the cross passage to be done from within the tunnel towards the shaft, which was considered more beneficial with respect to the overall project schedule.

The re-design, performed by Alpine-BeMo from Austria, was based on the Sequential Excavation Method (SEM). To address the anticipated ground conditions, the cross section was subdivided into three drifts. The size and advance length of the drifts was limited to maintain stability of the face and to control deformations. The excavation profile of each drift was further subdivided into top heading and bench/invert. Grouted canopy tubes, installed from within the shaft, were required as a pre-support measure. The advance length in the design was limited to 1m (3ft) in the top heading. Bench/invert was permitted to be removed in one continuous operation. The temporary ground support consisted of 300mm (12in) of shotcrete with two mesh layers and lattice girders. The support for the opening in the segmental liner of the bored tunnel was

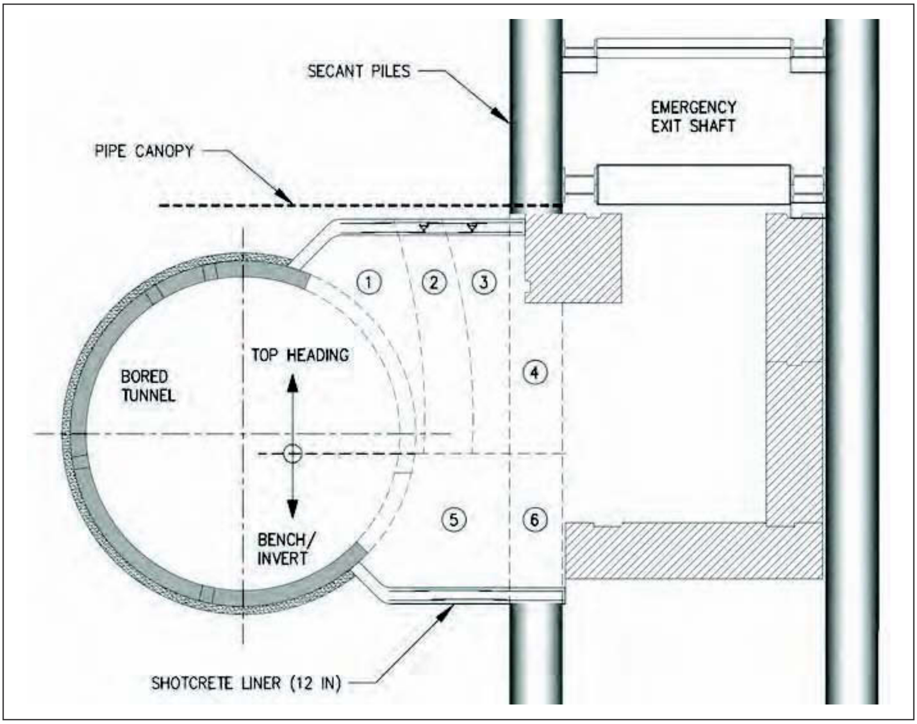


Figure 25. Cross passage excavation sequence

changed to a "hamster cage" type bracing, to allow excavation of the drifts from the bored tunnel side. Figures 25 through 28 illustrate the re-designed cross passage and its construction sequence.

To save time during installation, the "hamster cage" was designed as a collapsible system that could be brought into the tunnel on a special made carrier mounted to a flatcar, and expanded inside the tunnel, tight against the liner, by means of hydraulic jacks. The hamster cage system that was developed by Kelly Engineering is depicted in Figures 29 and 30.

The opening through the segmental tunnel liner was accomplished by cutting the perimeter using a circular concrete saw. The segments were then broken out using a hydraulic breaker mounted to a mini-excavator, which was also used initially for excavating through the jet-grout. The target strength of the jet-grout block was supposed to be 3.4MPa (500psi) minimum, and was expected to be within that range. Post jet-grout core samples however revealed an average compressive strength of around 25MPa (3,600psi). Production rates with the conventional mini-excavator were therefore low, and it was decided to replace the mini-excavator with a Brokk remote controlled demolition breaker, which proved to be the right decision.

Shotcrete was applied using the dry-mix method. The shotcrete mix was delivered in bulk bags and dumped into a silo feeding the hopper of the shotcrete pump. Liquid accelerator was added at the nozzle along with the water. The shotcrete plant was set up on the surface and a slick line ran down the Emergency Exit shaft into the tunnel through a core hole that had been drilled from the shaft in advance of the cross passage excavation.

Once the two side drifts were excavated, the cast-in-place permanent liner had to be installed in both of them before excavation of the center drift could begin. Prior to

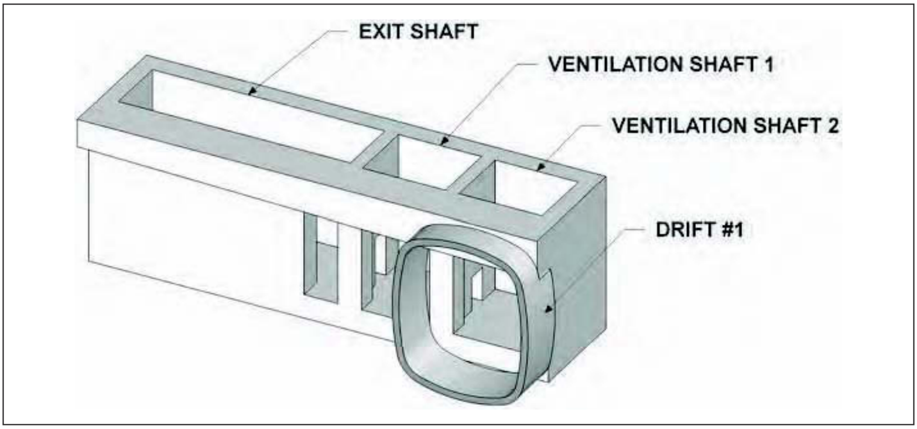


Figure 26. Cross passage drift 1 excavation completed

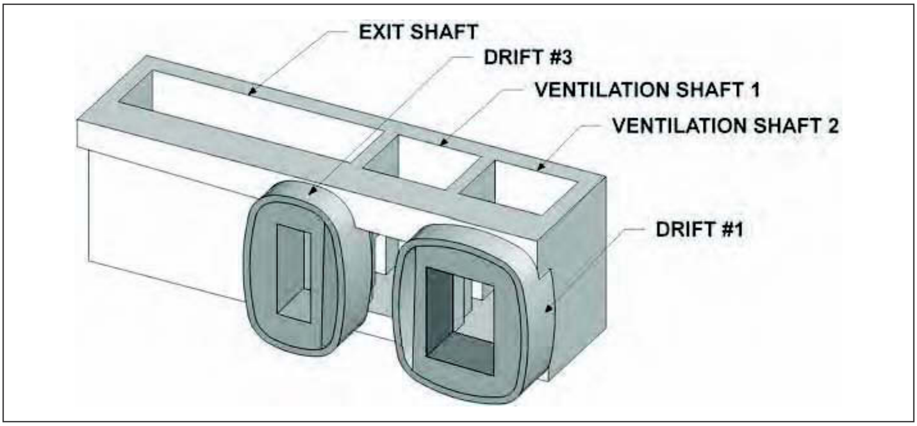


Figure 27. Cross passage drift 1 + 2 permanent liner completed

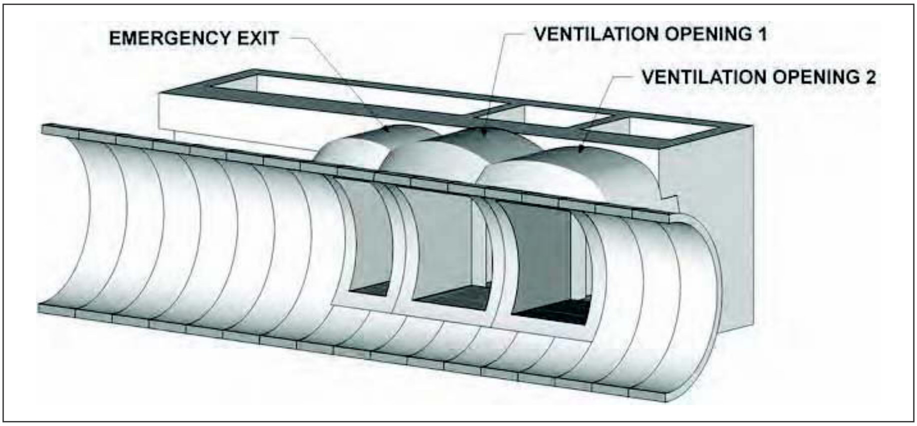


Figure 28. Cross passage completed

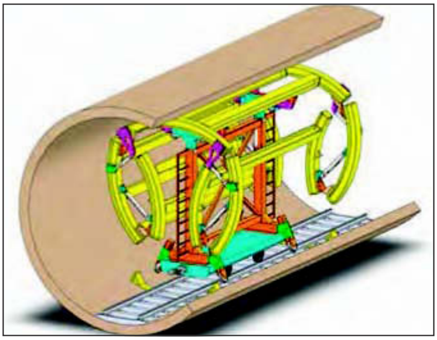


Figure 29. Hamster cage during transport

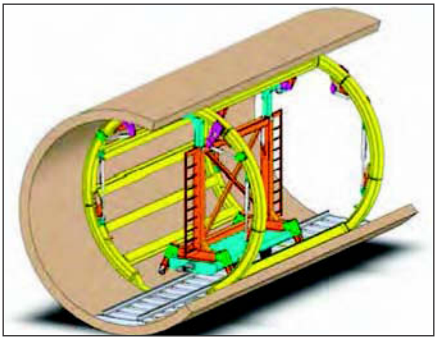


Figure 30. Hamster cage erected

pouring the permanent concrete, the shotcrete support had to be lined with a waterproofing sheet membrane. The cast-in-place liner was subsequently installed in three stages: invert, walls, and crown. The concrete was supplied in ready-mix trucks and pumped from the surface via a slick line down the shaft into the cross passage. After the permanent liner of the side drifts had reached 75% of its design strength, the center drift was excavated and afterwards lined with the permanent concrete.

SCHEDULE AND PRODUCTION

Both TBMs arrived at the Port of Newark (New Jersey) during the third week of September 2010. Upon arrival, they were transferred onto barges and moved up the East River to a temporary staging area in northern Astoria (Queens). Once the launch shaft was ready, TBM components and trailing gear were loaded onto modular trailers and flatbeds, and hauled on public roads to the jobsite during nighttime. Assembly of TBM S-558 and S-559 began on January 13 and February 8, 2011, respectively.

On May 18, 2011, TBM S-558, which in the meantime had been named 'Bobby,' was launched for mining of the Yard Lead Tunnel. About nine months later, on February 9, 2012, Bobby holed through at the reception pit completing the 1,316m (4,318ft) long drive. The best daily production rate achieved in a 24-hour period was 29m (95ft), which ended up being the highest achieved on the project. The average daily advance rate turned out to be 6.7m (21.8ft). In total five interventions into the Cutterhead for changing of tools had been performed. After completion of the Yard Lead tunnel, Bobby was disassembled and brought back to the launch shaft where it was re-assembled for mining of the 666m (2,185ft) long BC Tunnel. On May 7, 2012, three months after the hole-through, Bobby was launched for the second time to complete the BC drive in 2.5 months time. After reaching the end station on July 23, 2012, the shield of the TBM was left in the ground and all valuable parts salvaged, which took six weeks. 10.2m (34ft) was the average production rate attained per day with a maximum of 22.4m (74ft). While mining the BC drive, the TBM had been stopped twice for cutter changes and once to wait for clearance from the railroad for mining of the BC tunnel extension.

The second TBM on the job, (S-559) christened 'Penny,' began mining the A Tunnel on August 11, 2011. Roughly five months later, hole-through was celebrated at the reception pit. During the 585m (1,919ft) long drive, an average advance rate of 6.2m (20ft) per day had been achieved. The best daily rate accomplished was 21.7m (71ft). TBM advance was interrupted twice, first for a cutter change and a second time to wait for a jet-grout block to cure, which had been installed for tie-in with the future approach structure. Also, for the last half of the A Tunnel, day shift did not mine, but

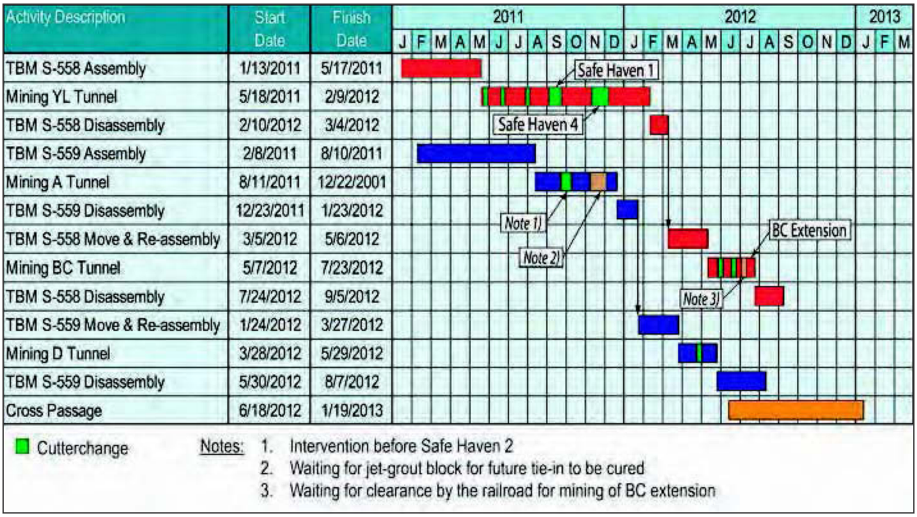


Figure 31. Schedule and production summary

instead, prepared the Launch Shaft decking for future TBM re-assembly, whilst the other two back shifts mined tunnel. After the hole-through, Penny was disassembled and transported back to the launch shaft where it was put back together for excavation of the D Tunnel. This operation took three months and finally on March 28, 2012, the TBM took off for the mining of the 670m (2,198ft) long D drive. Two months later, Penny had reached the target end station where it was abandoned by gutting the shield. The trailing gear was pulled back to the launch shaft, where it was hoisted to the surface and dismantled. The abandonment of the TBM and pulling of the trailing gear took two months. The highest daily production rate achieved was 21.3m (70ft) with an average of 13.7m (45ft) advance per day. Tunneling was stopped only once for a period of five days for changing of cutters in compressed air.

Mining operations were performed based on three 8-hours shifts except for the first two and six weeks on the A drive and YL drive, respectively, where only two 8-hour shifts were employed to facilitate training of the crews. In general, TBM advance was performed five days a week and interrupted for the weekend and union holidays except when mining under the main-line tracks where advance had to be continuous on a 24/7 basis. The best weekly and monthly advance rates were achieved on the D tunnel drive, which were 117m (384ft) and 386m (1,267ft), respectively (see Figure 31).

CONCLUSION

Contract CQ031 demonstrated that complex tunneling work can be successfully accomplished in the middle of a dense urban area, with challenging soil conditions, and beneath a mainline railroad. The keys to its success were careful planning, a focus on safety, cooperation between the parties, continuous monitoring of instrumentation, and good communication with stakeholders.

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PART 18

Pressure Face TBM Technology

Chairs

Gregg Olsen
Jay Dee Contractors, Inc.

Steve Skelhorn
McNally Construction, Inc.

THE CURRENT STATUS OF LAWS AND REGULATIONS REGARDING HYPERBARICS

Jerry East ▪ Hyperbaric Consultant

Gregory M. Hauser ▪ Dragados USA

Dan Weathers ▪

Working under compressed air for tunnel excavation has traditionally been applications where the entire tunnel was pressurized and the workers remained under pressure for the entire shift. These tunnel excavations were usually at pressures below 20 psi but there were some applications at greater pressures, with 50 psi being the maximum pressure allowed. The majority of the current regulations for Compressed Air Workers in the tunneling industry were generated for this type of compressed air excavation of the tunnel and do not reflect recent advances in technology and equipment. With advances in compressed air technology and improvements in tunnel equipment and methods, tunnels are now capable of being driven in geologic conditions that were not possible before these advances. The regulations that govern working conditions in those circumstances have not kept up with the advances and this paper will detail the current status of efforts to revise those regulations.

Tunnel boring machines have revolutionized the tunneling industry in recent years. While the techniques have evolved a great deal, the regulations in the United States have not kept pace. The regulations were written for a time when a section of the tunnel was isolated with a pressure bulkhead and the tunnel itself was pressurized. Entire work crews would pass into the pressurized area and often work their entire shift under pressure, essentially digging the tunnel by hand. Modern tunnel boring machines have changed that. It is common now for the tunneling crew to spend almost all their time working at a safe one atmosphere pressure (regular ambient air pressure), with small crews only occasionally making “Interventions” into the pressurized excavation chamber behind the cutter head, usually to inspect or change the cutters. A recent trend is many projects in soft ground require that the TBM be equipped with man locks so that the crew is properly equipped to perform hyperbaric interventions.

The Occupational Safety and Health Administration (OSHA) is the federal government agency with the responsibility and the authority to both set and enforce workplace health and safety standards. OSHA has authority in every state; however, each state also has the option of creating its own state OSHA system. If a state choses to do this Federal OSHA must approve and monitor each states plan. For a state to get their own program approved they must have safety and health regulations at least as effective as the federal OSHA program. 22 states currently have their own plan covering private sector workers.

The major hurdle facing tunneling companies is that the current regulations are technologically inapplicable and the regulations are recognized as being outdated. There exists no concerted national effort to modify the current compressed air regulations compatible with the advances that have been made by the tunneling industry. To date Federal OSHA appears reluctant to undertake this endeavor as participant states can require additional mandates beyond those required by the federal requirements. Further, states do not appear to be willing to establish reciprocation of approved hyperbaric operation and safety plans. Currently there is no concerted national effort by any group, Federal, State or Industry to update compressed air regulations into one cohesive standard that the Tunnel Industry can rely on or to use as a template to adapt to specific circumstances.

Tunnel Boring machines are not mentioned in the Federal OSHA Regulations. Tunneling technology and hyperbaric technology have both changed greatly in the years since the regulations were written. Even though the regulations are outdated, they still apply. The contractor must apply for and receive a variance from either Federal or State OSHA for any regulations that he does not comply with. Most of the state plans have standards that are the same as the Federal OSHA's, some have standards just slightly different from federal OSHA and a couple have standards that are significantly different from Federal OSHA. States may raise the required level of worker protection from that of the federal OSHA program, but they cannot lower it below the standard of protection required by federal OSHA. Even if a state has their own plan, the federal OSHA rules still apply in that state and the state cannot lower the workers standard of safety to less than what the federal plan demands. If a contractor is working in a state that has its own plan, then he looks to state OSHA for the regulations he must follow. If he is operating in a state without its own plan, he looks to federal OSHA for rules.

All federal rules and regulations including the Federal OSHA regulations are part of the Code of Federal Regulations, or "CFR." These rules are treated by the courts as being legally binding as law. The CFR is divided into 50 titles with each title representing broad areas subject to Federal regulation. The safety and health regulations for construction are found in title 29, part 1926. The rules specially for compressed air tunneling are found in title 29, part 1926, section 803. This would usually be written as 29 CFR 1926.803.

The general layout of most compressed air work involving larger diameter tunnel boring machines is to have a man lock (man lock is another word for decompression chamber) mounted on a bulkhead near the front of the machine. The workers, usually 2 or 3 at a time, enter one end of the man lock, are pressurized and then exit the other end of the man lock either directly into the excavation chamber of the TBM or into a pressurized "motor room" (housing the drive motors for the TBM) that is directly behind the excavation chamber (OSHA calls this the "working chamber"). If they pass through the man lock and enter a motor room, they must next move across the motor room to the forward bulkhead separating the motor room from the excavation chamber, and carefully open the hatch to enter the excavation chamber and begin work.

On the surface, near the entrance of the tunnel, there is a compressor plant supplying the breathing quality compressed air to the tunnel. In some instances or as required by some agencies, there should also be a "medical lock." The medical lock is a decompression chamber that is kept ready to treat decompression sickness or, perhaps, other medical emergencies if the worker also has a decompression obligation that must be dealt with. There have been instances where a hospital equipped with a medical lock is close enough to the Project to qualify for the sites medical lock facility.

It has become a common practice on larger tunnels to have a rail mounted hyperbaric evacuation shuttle that can transport workers under pressure from the tunnel to the medical lock. This provides a means to safely transport workers with a decompression obligation if the tunnel must be evacuated for some reason or the worker has an injury that is too serious to deal with in the tunnel. This evacuation shuttle is not directly required by the OSHA regulations, but it has become a common industry practice and could be required under the General Duty Clause, particularly for tunnels working at higher pressures.

THE GENERAL DUTY CLAUSE

An important part of the Federal OSHA program is a catch-all requirement that covers all conditions. This is the "General Duty Clause." The General Duty Clause reads, "Each Employer shall furnish to each of his employees employment and a place of employment which are free from recognized hazards that are causing or likely to cause death or serious physical harm to his employees."

Sometimes there is a hazard, but OSHA has no specific rule or standard dealing with it. Under the General Duty Clause, the employer has an obligation to protect workers from serious and recognized hazards even when there is no standard. Employers must take whatever action is feasible to eliminate these hazards, hence the presence of the shuttle and Medical Lock.

SOME FEDERAL OSHA HIGHLIGHTS

The Physician

"*There shall be retained one or more licensed physicians familiar with and experienced in the physical requirements and the medical requirements of compressed air work and the treatment of decompression illness. He shall be available at all times while work is in progress in order to provide medical supervision of employees employed in compressed air work.*" 1926.803(b) (1)

"*No employee shall be permitted to enter a compressed air environment until he has been examined by the physician and reported by him to be physically qualified to engage in such work."* 1926.803(b) (2)

The Medical Lock

"*A medical lock shall be established and maintained in immediate working order whenever air pressure in the working chamber is increased above the normal atmosphere.*" 1926.803(b) (9)

The medical lock must *"have 6 feet of clear headroom at the center….*" 1926.803(b) (10) (i)

"*Be kept ready for immediate use for at least 5 hours subsequent to the emergence of any employee from the working chamber."* 1926.803(b) (10) (iv)

"*Be designed for a working pressure of 75 p.s.i.g."* 1926.803(b) (10) (vii)

"*Be equipped with a manual type sprinkler system that can be activated inside the lock or by the outside lock tender*" 1926.803(b) (10) (x)

"*Be provided with oxygen lines and fittings leading into external tanks. The lines shall be fitted with check valves to prevent reverse flow. The oxygen system inside the chamber shall be of a closed circuit design as to automatically shut off the oxygen supply whenever the fire system is activated*." 1926.803(b) (10) (x)

"*The medical facility shall be equipped with demand-type oxygen inhalation equipment..."* 1926.803(b) (10) (xiv)

"Be provided with sources of air...which are capable of raising the air pressure in the lock from 0 to 75 p.s.i.g. in 5 minutes." 1926.803(b) (10) (xvi)

The Compressor Plant and Air Supply System

"…*air compressor units shall have at least two independent and separate sources of power supply and each shall be capable of operating the entire low air plant and its accessory systems.*" 1926.803(h) (3)

"*The capacity, arrangement, and number of compressors shall be sufficient to maintain the necessary pressure without overloading the equipment and to assure maintenance of such pressure in the working chamber during periods of breakdown, repair, or emergency."* 1926.803(h) (4)

"Duplicate low pressure air feedlines and regulating valves shall be provided between the source of air supply and a point beyond the locks…" 1926.803(h) (6)

The Man Lock

"*Except where air pressure in the working chamber is below 12 psig, each man lock shall be equipped with automatic controls which…shall automatically regulate decompressions. It shall also be equipped with manual controls…"* 1926.803(g) (1) (iii) This particular regulation is out dated and usually the reason for a variance request as automatic controls are no longer considered safe.

"The man lock shall be large enough so that those using it are not compelled to be in a cramped position, and shall not have less than 5 feet clear headroom at the center and a minimum of 30 cubic feet of air space per occupant." 1926.803 (g) (1) (ix)

"A…continuous recording pressure gauge with a 4-hour graph shall be installed outside of each man lock… A copy of each graph shall be submitted to the appointed physician after each shift." 1926.803 (g) (1) (v)

"*Man locks shall be equipped with a manual type fire extinguisher system that can be activated inside the man lock and also by the outside lock attendant. In addition, a fire hose and portable fire extinguisher shall be provided inside the and outside the man lock.*" 1926.803 (l) (9)

"*Equipment, fixtures, and furniture in man locks …shall be constructed of noncombustible materials*." 1926.803 (l) (10)

The Special Decompression Chamber

"*A special decompression chamber of sufficient size to accommodate the entire force of employees being decompressed at the end of a shift shall be provided whenever the regularly established working period requires a total time of decompression exceeding 75 minutes*." 1926.803 (g) (1) (xvi)

"*The headroom in the special decompression chamber shall not be less than a minimum 7 feet and the cubical content shall provide at least 50 cubic feet of airspace for each employee."* 1926.803 (g) (2) (i)

OSHA defines this "Special Decompression Chamber" as *"A chamber to provide greater comfort of employees when the total decompression time exceeds 75 minutes*." Requiring a "Special Decompression Chamber" is a remnant of old style tunneling, however, it is still a current regulation and still a requirement. Another such hold over from the early days of compressed air work is a prohibition against female lock attendants, a statute still on the books in many states that now requires a variance if a contractor has a need to employ females for this purpose.

Decompression Issues

"*No employee shall be subjected to pressure exceeding 50 pounds per square inch except in emergency*." 1926.803 (e) (5)

It is not uncommon for modern compressed air work to exceed this 50 psi limit and many current tunnels are excavated under pressures far above this.

"*Decompression to normal condition shall be in accordance with the Decompression Tables in Appendix A of this subpart*" 1926.803 (f) (1)

The OSHA decompression tables in Appendix A are obsolete. They use a slow, continuous, reduction in pressure until atmospheric pressure is reached to accomplish the decompression. The workers breathe only air during the decompression. They have a history of producing both an unacceptable rate of decompression sickness and also dysbaric osteonecrosis, a potentially crippling condition caused by death of portions of the bone.

Modern decompression tables use staged decompression where the worker stops at a specific pressure for a specific amount of time, than the pressure is reduced and he spends a specified amount of time at the lesser pressure. The process continues, following the decompression table, until he reaches atmospheric pressure. Modern decompression tables also use 100% oxygen as the preferred breathing gas during parts of the decompression. Oxygen decompression results in both a more efficient and a much shorter decompression. The pressure to which any worker might be required to work can range from slightly above atmospheric to eight times atmospheric or 8 bars, nominally 112 psi. OSHA, as set forth at 29 CFR 1926.803, limits the working pressure to which a worker may be exposed to 50 psi. The National Institute for Occupational Safety and Health (NIOSH) at "NIOSH Decompression Tables" permits the use of oxygen or oxygen and air mixtures; however the compressed air limit of worker exposure remains at 50 psi, unless a variance is granted to work at higher pressures.

The state of California has adopted the oxygen decompression tables from Revision 6 of The U.S. Navy Diving Manual as mandatory for compressed air work. The National Institute of Occupational Safety and Health (NIOSH) have recently published the "NIOSH Decompression Tables," a set of decompression tables developed for tunneling in the 1980s. Both the Navy tables and the NIOSH tables have their issues, however either is a far better choice than the federally mandated OSHA decompression tables.

Often a tunnel contractor will contract for the generation of specific tables to meet the conditions encountered on a particular project. If that happens, then the Contractor must also acquire a variance from the Federal OSHA or State OSHA tables to use that new specific table.

THE VARIANCE PROCESS

A variance is a regulatory action that permits an employer to deviate from the requirements of an OSHA or State standard under specified conditions. It is common in compressed air work for contractors to request variances from parts of the OSHA standard that they feel do not apply on their project or they find unreasonable or inadequate for some reason. Some examples of variance requests might be the requirement for the special decompression chamber, the 50 psi pressure limit and the requirement to use the OSHA decompression tables.

In order to get a variance the contractor must demonstrate that the proposed alternative means of compliance provides its workers with safety and health protection that is equal to, or greater than, the protection afforded to them by compliance to the standard from which they are seeking a variance. Often this variance process is time consuming and involves several iterations of the request.

PRESSURE VESSEL STANDARDS

Although it is not part of the compressed air work regulations, OSHA does require that decompression chambers used in tunneling, both man locks and medical locks, comply with the American society of Mechanical Engineers Boiler and Pressure Vessel Code or equivalent. The ASME Safety Standard for Pressure Vessels for Human Occupancy, usually called ASME PVHO-1, is the applicable standard.

Almost every aspect of the construction of decompression chambers is covered by The ASME PVHO-1 standard. It gives the standards for the pressure vessel shell, the examination of the welds and the testing of the pressure vessel. The only windows allowed are made from acrylic plastic and are designed, manufactured and tested to the code requirements.

It also specifies the piping system design and piping material. Most piping must be non-ferrous, such as stainless steel or brass. The piping system must be certified

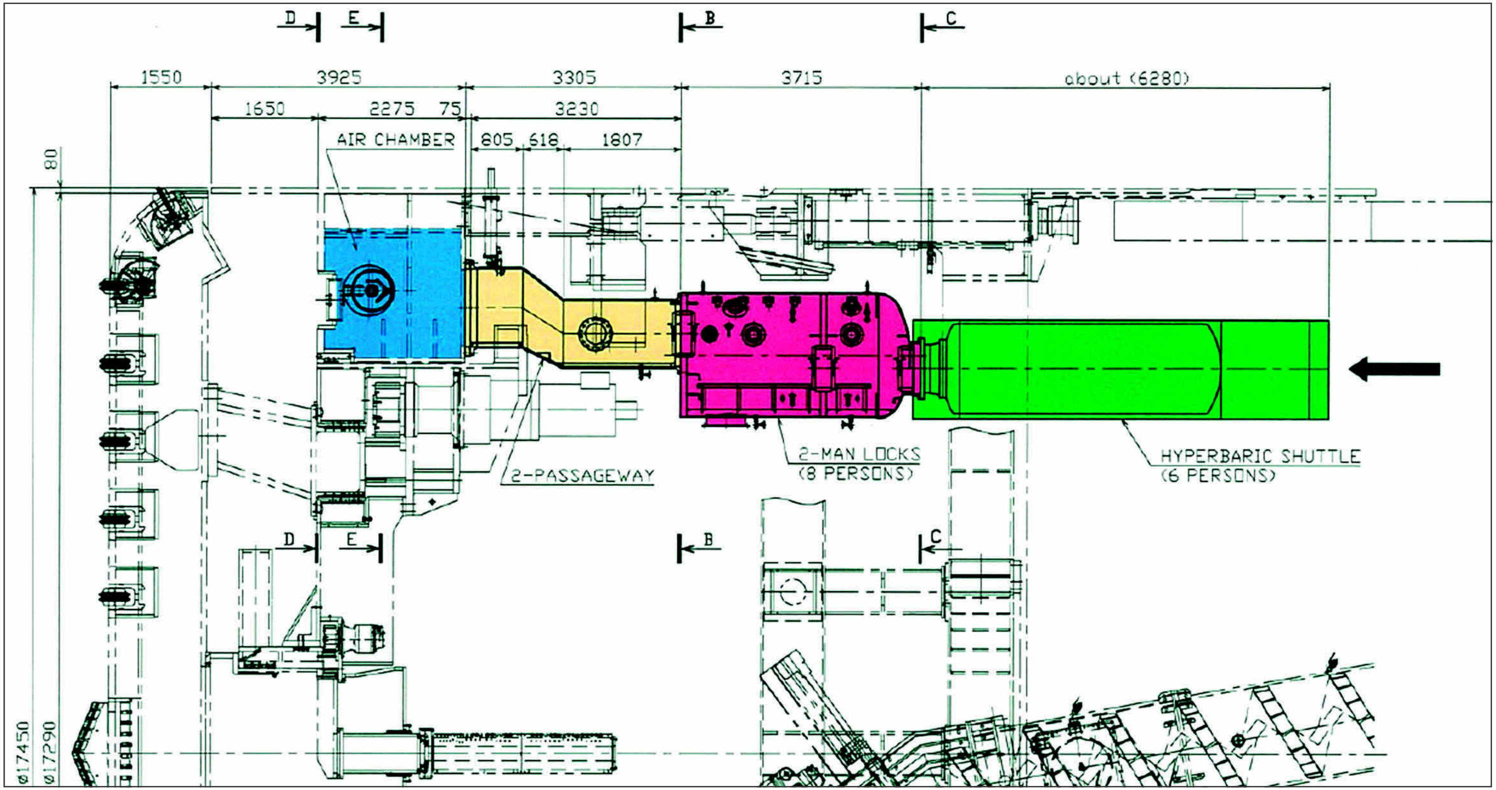

Figure 1. Green—transport vessel (DART); red—man lock; yellow—access tube; blue—TBM, not PVHO-1 construction

to conform to the PVHO standard by one of the following methods; (1) certification of compliance with the standard by a registered professional engineer, (2) Third party certification by an independent classification society competent in pressure vessel for human occupancy systems, or, (3) Written certification of compliance with the PVHO Standard by the fabricator of the piping system.

The ASME has recently started a subcommittee on tunneling. Its purpose is to address pressure vessel issues specific to the tunneling industry and evolve a set of standards specific to tunneling. Eventually the ASME PVHO-1 standard will have a new section that applies only to pressure vessels used in the tunneling trade.

MEDICAL CHAMBERS

Medical hyperbaric chambers in the United States, including ones used in tunneling, must meet the ASME PVHO-1 Standard and also additional requirements of the U.S. Food and Drug Administration (the “FDA”). Medical hyperbaric chambers used in tunneling are considered medical devices by the FDA and must meet the applicable rules.

PRESSURE VESSEL INSPECTION

Before allowing the decompression chamber or man lock to be used, many states or cities will require inspection by a Pressure Vessel Inspector. Most pressure vessel inspectors will want to use “The National Board Inspection Code” for pressure vessel inspection procedures and guidelines. The National Board Inspection code is the flagship publication of the National Board of Boiler and Pressure vessel Inspectors. It provides rules and information for inspection of pressure vessel installation and repairs. Unfortunately, the National Board Inspection Code does not yet address PVHO’s. This leaves some inspectors without the clear and familiar inspection guidelines they are used to and leaves the inspection of a complex and unusual device (the man lock) to someone that may know very little about them and has few guidelines to follow in conducting a meaningful inspection. There is also some confusion, and uncertainty in determining if portions of a TBM are or should be considered a PVHO. The instance of a man lock opening into a motor room for instance, creates the potential of that motor room being a PVHO and therefore require being constructed to those standards. To date, the determinations have been that such situations do not result in the motor room being a PVHO because when the forward hatch is opened, they are no longer a sealed vessel and therefore not a PVHO.

The National Board of Boiler and Pressure Vessel Inspectors has recently formed a committee to address pressure vessels for human occupancy used in tunnels. Hopefully there will soon be a much better set of rules for pressure vessel inspectors to follow for the devices that are used in tunnel construction.

CHANGE MAY BE COMING

The Federal Occupational Safety and Health Administration and most states recognize that the standards for compressed air work need updating. However the process of change is a slow and difficult one. Even so, change is happening. Federal OSHA has asked the ASME to review the pressure vessel requirements associated with tunnel boring machines and the ASME has formed a task group to do this. Washington State has been conducting a series of meetings to give interested parties a voice in what changes should be made to their state OSHA program and has begun writing some new regulations. California has adopted the U.S. Navy diving decompression tables as mandatory in tunneling compressed air work. All these are small changes but, taken together, they indicate a growing awareness that the regulations badly need updating and an interest in doing so.

Any Manager, Superintendent or Owner who has ever put men out under compressed air knows the danger he is exposing his men to. It is critical that the tunnel industry have the most up to date standards and consistent regulations to ensure worker safety under these conditions. The regulations should be specific to underground construction and loose the tie to the underwater applications where much of the technology has been developed. We are tunnel constructors and we employ compressed air workers, not divers.

The need for revision of existing regulations and updating the technology is recognized by the regulators and the industry. By working together we can achieve a better standard that allows for changing technology that mandates worker safety. The use of underground space will increase and the need for this revision will become greater the longer it remains unaddressed. It is in everyone's best interest to work for these revisions.

EXPERIMENTAL STUDY OF IMPACT OF SOIL CONDITIONING ON SOIL ABRASION AND CUTTER WEAR OF EPB TBMs

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ABSTRACT

One of the main operational parameters in EPB tunneling is soil conditioning. Soil conditioners are often used for several reasons such as making the muck flowable, lowering inner friction between the soil particles, mitigating soil stickiness, preparing the excavated soil to be compressible during the EPB operation, controlling the water inflow, reducing the torque on the cutterhead and other components and finally reducing the wear and tear on the cutters and other moving components in the excavation process such as screw conveyor. This paper reviews the common practice in soil conditioning and will focus on assessing the influence of the soil conditioning on tool wear. A new testing system has been developed at PennState University to evaluate soil abrasion and has been used on several soil samples from various soft ground tunneling projects around the US. Some of the tests performed on different soils and the result of soil abrasion tests on dry, moist, saturated, and conditioned soil, as well as impact of soil conditioning on torque requirement during the testing are presented in this paper.

INTRODUCTION

Soil conditioning is one of the main factors in successful application of EPB machines in tunneling projects, which involves changing the characteristics of the ground in order to make it suitable for the tunneling process. Various conditioners are applied at different points throughout the excavation process that include face of the tunnel, within the cutting chamber, inside the screw conveyor, etc. Milligan (2000) summarized the advantages of using soil conditioning as follow:

1. Increasing the stability of tunnel face
2. Improving the flowability of material
3. Reducing the friction and therefore reducing the driving torque
4. Changing the excavated material into a uniform plastic soil which leads to:
    a. Better control of pressure inside the cutting chamber
    b. Better control of groundwater inflow
    c. Better control of flow of soil in the screw conveyor
5. Reducing the clogging in the chamber
6. Better handling of excavated soil

7. Improving the safety of the personnel specially during the maintenance of the cutters/cutterhead
8. Maintaining of above conditions during tunneling operation and maintenance stops
9. And finally, reduction of wear and tear of the cutters, cutterhead, and other wear parts

Soil conditioning is done by injecting foam, polymer, water, and filler (bentonite) to the tunnel face, pressure chamber and screw conveyor. Selection of the type of foam and polymer mainly depends on soil type, geological condition (groundwater and soil permeability), and properties of tunnel boring machine (injection points, open or closed cutter head, type of foam generator, etc). The most important soil conditioners are foam and polymer. However, in some cases due to existing conditions, some other additives like anti clogging or anti wear material are used. A close look at various cases of using EPB machines in recent years show that getting the soil conditioning right is the key to successful operation of the machine. This issue is becoming ever more important as the larger EPB machines, even approaching 18 m in diameter as the case is for the SR-99 Alaskan Way project, have emerged in the tunneling projects. The larger size of the machines will pose many challenges including the additional variation in soil types and characteristics at the face, rapid changing of the soil behavior, exposure of larger area of the tunnel to presence of different soil layers with higher permeability, and finally the issue of the required torque in larger size machines that is very sensitive to the soil behavior.

CHARACTERIZATION OF FOAM AND CONDITIONED SOIL

Conditioned soil is a complex mixture of three phases including solid (soil grains), liquid (water and foaming agent), and gas (air). It is very important to characterize and understand the behavior of this mixture to evaluate its impact on the tunneling operation and functionality of the conditioning to meet its required role in the mixture. For this purpose, there are certain tests that are discussed here.

Characterization of Foam

In order to characterize the foam used for tunneling purposes, simple laboratory tests have been developed (Quebed et al. 1998) that are summarized as follow:

1. Generation test: to study the relationship between pressure generation and fluid flow in generator and foam flow rate
2. Consistency test: to quantify the foam quality (bubbles size)
3. Half-time test: to measure the necessary time for foam to lose half of its solution used originally for its generation
4. Compressibility test: to understand the foam behavior in a confined environment and under pressure changing

It must be noted that in the tests mentioned above, foam is tested separately from the soil.

Characterization of Conditioned Soil

To evaluate conditioned soil there is no agreement on a universal test but some tests are used for qualification of conditioned soil that have been adapted from concrete tests or geotechnical tests. Some of these tests are as follow:

1. Foam Penetration Test: the purpose of this test is evaluation of foam penetration into soil (tunnel face). In this test pressurized foam is pushed into the soil.

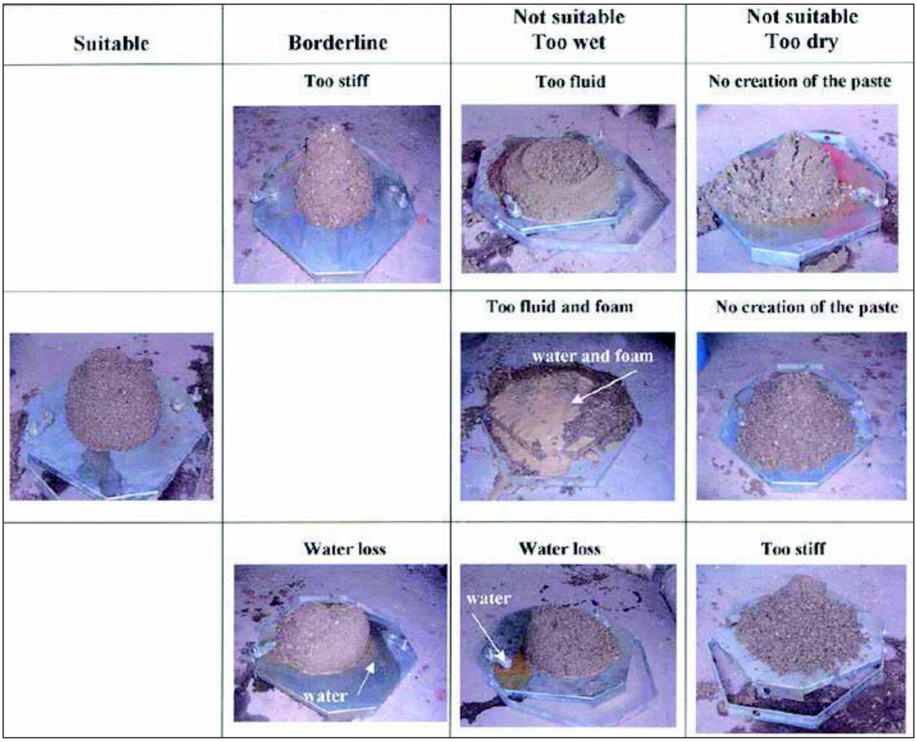


Figure 1. Reference shapes for classification of conditioned soil (Borio et al. 2007)

If foam penetration is high, then foam consumption increases and produced pressure may be insufficient. On the other hand, if foam penetration is low, control of groundwater is difficult during the operation.

2. Mixing Test: in this test, soil and foam mixed together and the variation of the electric motor power, necessary time to obtain a homogeneous mixture and the quality and behavior of the conditioned soil are evaluated.
3. Slump Test: in this test (ASTM C143), soil with a certain amount of water and foam poured into a concrete mixer and after mixing, poured into the mold. Mold is carefully lifted vertically upwards in such a way that it does not disturb the conditioned soil cone. The amount of subsidence of the top of the sample due to the weight of the column is measured which is called slump value. The overall behavior of conditioned soil evaluated and classified based on reference shapes (Figure 1). This test provides an overall index on the rheological behavior of the conditioned soil.
4. Permeability Test: to evaluate the permeability of the conditioned soil, some methods like Constant Head Test (for coarse-grained soils) or Hydraulic Compression Cell (for fine-grained soils) can be used. In general, conditioned soil is less permeable than ordinary soil.
5. Compressibility Test: in this test, compressibility of conditioned soil is evaluated. This test can be done using similar apparatus that used for permeability test and effects of pressure variation on compressibility of conditioned soil can be measured.

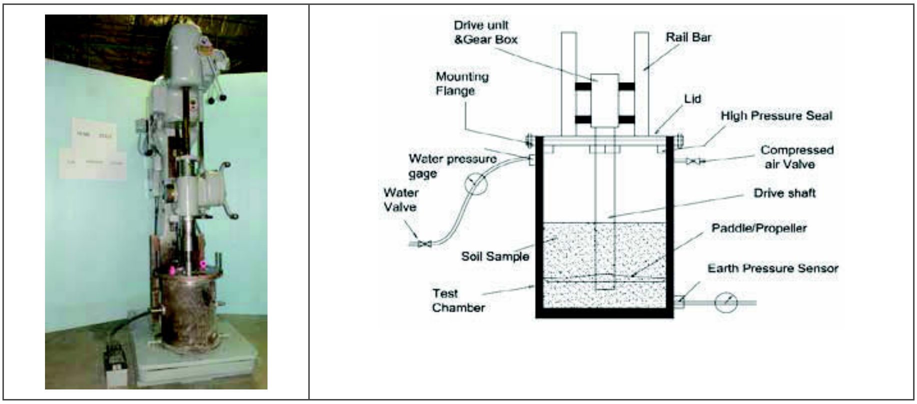


Figure 2. PennState soil abrasion testing system

6. Adhesion Test: this test is used for evaluation of adhesion between conditioned soil and metallic surface. In this test, adhesion of conditioned soil is measured by measuring the friction angle of soil. Measurement of friction angle can be achieved by using a sloping stainless steel surface (Quebaud et al. 1998), shear box (Jancesecz et al. 1999) or ring shear apparatus (Milligan 2000).
7. Cone Penetration Test: in this test, effect of foam solution type on clay soils is determined. For this purpose, a metallic cone falls down into the conditioned soil sample from a specific height and the penetration depth is measured.

STUDY OF THE EFFECT OF CONDITIONED SOIL ON REDUCING THE WEAR

As discussed in the previous section, variety of tests have been developed that look into different properties of the foam and conditioned soil for application in EPB tunneling. However, the effect of conditioned soil in reducing the wear of cutters or abrasivity of soil is absent in the literature. One of the major advantages of soil conditioners in EPB tunneling is to reduce the wear of the cutters and other components of the machine. The main theory behind the wear reduction phenomena is that the high surface area of foam constrains the soil particles and therefore decreases the friction between particles and soil grains and tool surface. PennState soil abrasion research group has developed a testing system that looks into the wear phenomena in soft ground mechanized tunneling. The proposed testing system has the capability of investigating the effect of soil conditioners on the wear as well as measuring the torque required to move a propeller through the soil. The testing device is briefly described in this section.

PennState Soil Abrasion Testing System

A unique test device is designed and manufactured at PennState University in order to address the issue of abrasion and wear in mechanized tunneling. The device consists of a cylindrical chamber 350 mm in diameter and 450 mm in length (14 × 18 inch). The chamber is partially filled with the sample. The propeller, which is intended to create maximum contact forces with the material, is attached to a drive shaft and rotates inside the cylindrical chamber at 60 rpm. The whole assembly is mounted on a drill press with a 5 hp drive unit (Figure 2). The propeller has three blades with the radius

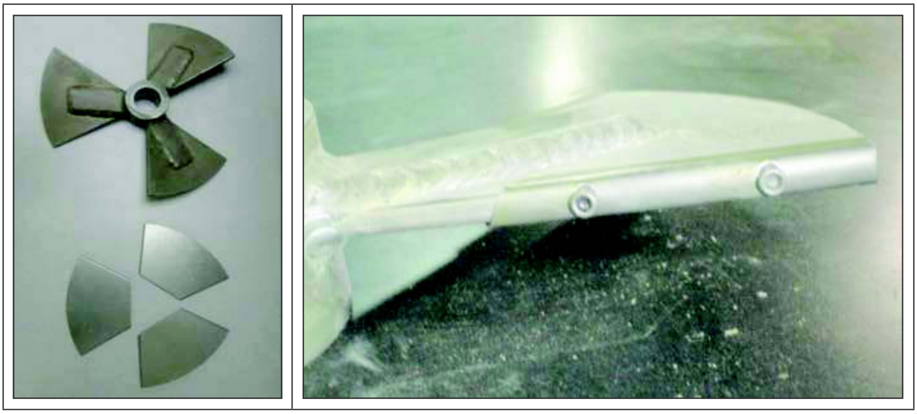


Figure 3. The propeller blade, and the mounting system of the cover on the blades

Figure 4. Direct torque measurement system

of 150 mm that are welded at 120 degrees angle. In order to avoid severe wear on the blades and also allow for more accurate measurement of the weight loss on the tools, the blades are fitted with steel covers (Figure 3). The covers can be made of different hardness (17, 31, 43, 51, 60 HRC) and weighed before and after each test to determine the weight loss during the test within a given time span. The chamber is constructed as a pressurized chamber having the capability of performing tests under ambient pressures of up to 10 bars. More detailed about the testing system and testing procedure can be found in Alavi Gharahbagh et al., 2011 and Rostami et al., 2012.

A direct torque measuring system is utilized in the testing device (Figure 4). This system measures the torque using two arms that are instrumented by using two S-shape load cells The data from two load cells are monitored by using the computer based data acquisition system and converted to torque as the testing proceeds.

The testing system can test dry, moist, saturated, and conditioned soil under up to 10 bar (~150 psi) of pressure if needed. A foam generator device (Figure 5) is used for generating foam with different concentration, and foam expansion ratios (FER). The generated foam is then added to the soil in a mixing device to meet a specific foam injection rate (FIR) and is charged to the test chamber for testing.

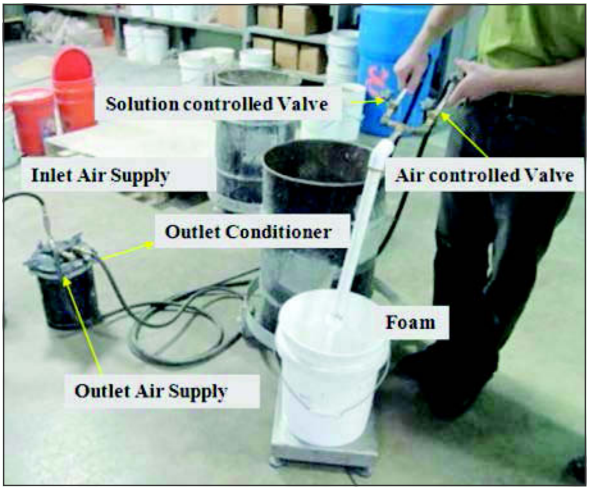


Figure 5. Foam generator device

RESULTS AND DISCUSSION

In order to study the effect of soil conditioners on abrasion, several tests were performed on three different materials by using different soil conditioning arrangements. The materials that are used for this study are silica sand, crushed rock sample from a tunnel in Washington DC, and crushed rock sample from a tunnel project in Indianapolis. The rock samples are crushed to less than 4.75 mm in size for testing.

Soil Conditioning Tests on Silica Sand

A series of preliminary tests were performed on silica sand samples. This sample is selected due to high quartz content as an abrasive testing material. Table 1 summarizes different performed tests on silica sand samples. As shown in Table 1, a total of 8 tests were performed on silica sand sample. Prior to each test, a slump test is performed to capture the optimal conditioning parameters based on Peila et al., 2009. Figure 6 displays the results of performed Slump tests on silica sand samples under different conditions. Figure 7 shows the samples at the end of testing with the soil abrasion testing device.

Figure 8 displays the results of soil abrasion tests for various moisture content and conditioning of silica sand. The comparison between conditioned soil and unconditioned soil shows that applying soil conditioner results in significant reduction of the soil abrasivity and wear on machine tools/parts. The most severe condition is when the soil has 10% water content and has shown a wear of 22 gram in only 10 minute of tests. The same soil shows only 0.6 gram of weight loss when conditioned by Meyco SLF-47 conditioner at proper FER and FIR.

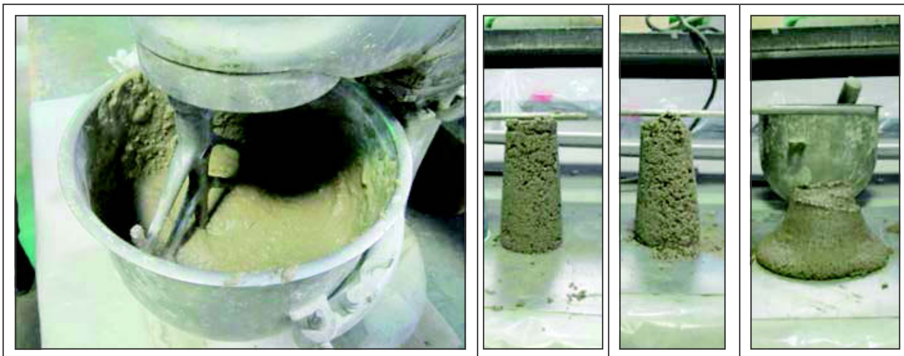
Soil Conditioning Tests on Muck Sample from Washington, DC

A set of three tests were performed on sample of muck from ongoing tunneling project in Washington DC in order to compare the difference in abrasive properties of the muck with and without conditioner. Prior to testing the samples were crushed to less than 4.75 mm in size. Table 2 is the summary of the test results.

Figure 9 displays the performed Slump tests on the samples as well as the condition of the sample after performing the abrasion test. Figure 10 shows the compacted samples at the bottom of the chamber after 5.5 min of abrasion testing on sample with

Table 1. Soil conditioning tests performed on silica sand samples

Soil	Moisture Content (%)	Testing Time (min)	Weight Loss (g)	Conditioning Properties
Silica sand	0–Dry	30	12.9313	—
Silica sand	10	10	22.0670	—
Silica sand	15	10	10.4559	—
Silica sand	15	30	0.6453	3% Conc. Meyco SLF 47, FIR=25%, FER=17
Silica sand	15	30	3.9132	3% Conc. ABR5 ,FIR=25%, FER=17
Silica sand	15	30	2.9787	1% Conc. AQF-2,FIR=30%, FER=17
Silica sand	15	10	13.3139	0.125% Quik Mud D-50 mixed with water and mixed with dry sand
Silica sand	15	30	2.9530	0.125% Quik Mud D-50 mixed with water and mixed with dry sand +1% Conc. AQF-2, FIR=28%, FER=14



(a) (b) (c) (d)

Figure 6. (a) Mixing soil with the conditioner; (b) slump test on 15% W silica sand sample; (c) slump test on 15% W silica sand sample conditioned with 3% concentration ABR 5, 25% FIR, and FER of 17; (d) slump test on 15% W silica sand sample conditioned with 3% concentration Meyco SLF 47, 25% FIR, and 17% FER

w=10% as well as the wear on the covers. The test was interrupted by the propeller stalling in the chamber within 5.5 minutes, as large amount of weight loss (41.55 g) was observed after measuring the weight loss of the covers at 5.5 minutes. Addition of conditioner to the sample has reduced the weight loss of the covers to about 1 gram during a 30 minute test. See also Figure 11.

Soil Conditioning Tests on Muck Sample from Indianapolis

A set of five tests were performed on sample from tunneling project in Indianapolis to compare the difference in abrasive properties of the sample with and without conditioner. In addition, since torque plays a significant role in EPB tunneling, torque is measured during each of these tests and comparison is made between the conditioned and unconditioned samples on the torque requirements. Prior to testing the muck samples were crushed to less than 4.75 mm in size. Table 3 summarizes the results of testing for this sample.

Figure 7. (a) 15% W silica sand sample conditioned with 3% concentration ABR 5, 25% FIR, and FER of 17; (b) 15% W silica sand sample conditioned with 3% concentration Meyco SLF 47, 25% FIR, and 17% FER after 30 minutes of test

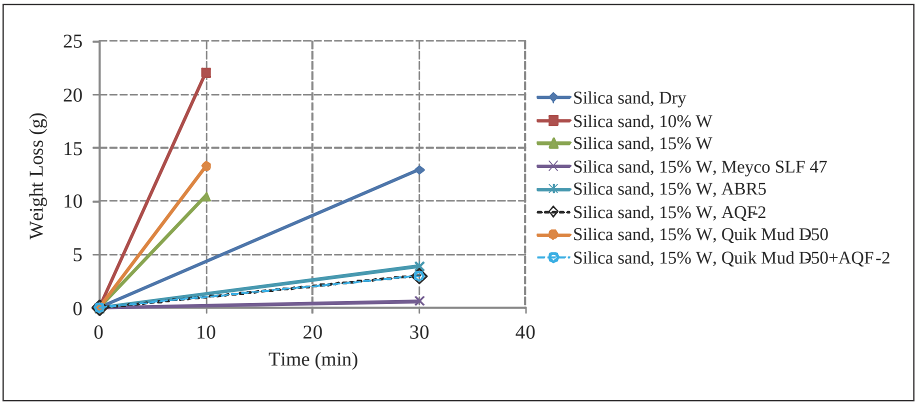


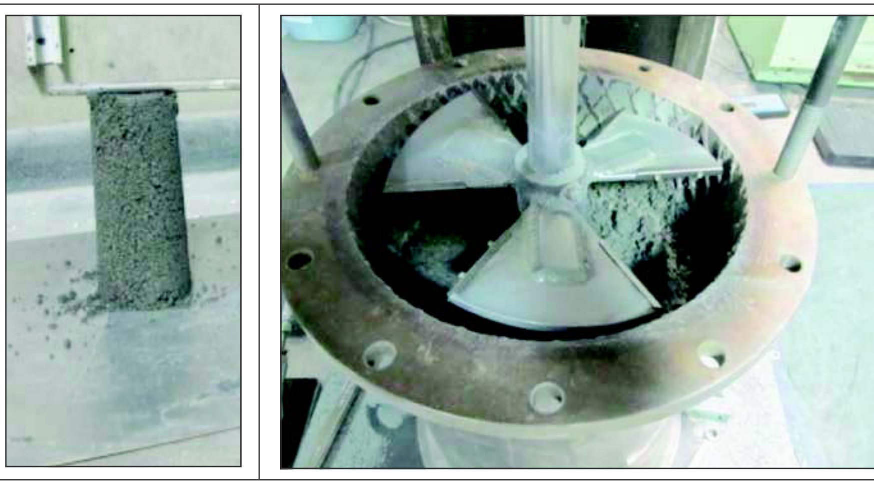
Figure 8. Results of testing with soil conditioner on silica sand samples

Table 2. Soil conditioning tests performed on samples from a tunnel in Washington, DC

Soil	Moisture Content (%)	Testing Time (min)	Weight Loss (g)	Conditioning Properties
DC Tunnel	0–Dry	30	8.8929	—
DC Tunnel	10	5.5	41.5526	—
DC Tunnel	10	30	1.0902	3% Conc. Meyco SLF 47, FIR=50%, FER=10

Figure 12 shows the tested samples in different conditions after abrasion testing. Due to high amount of torque in tests with 7.5% and 11% water content, shear pin which connects the propeller to the shaft inside the testing chamber was sheared. This pin is primarily used to protect the testing device against high amount of torque that could potentially damage the motor and gear box of the testing system. The weight loss of the covers is not reported for these tests since the tests were stopped.

As one can observe in Table 3, this sample is not very abrasive due to the mineralogy of the sample being primarily limestone (or dolomite), but the main issue about this material is cementation of the grains which leads to rapid consolidation by compaction



(a)

(b)

Figure 9. Slump and soil abrasion tests on muck sample from Washington, DC, tunnel with (a) w=10%; (b) W=10% conditioned with 3% concentration Meyco SLF 47, 50% FIR, and FER=10

under low moisture contents. Figure 12 part (b) displays the material at the bottom of the chamber after performing of the test on this sample with w=11% for less than 5 seconds. Due to high amount of torque (T=755.12 N·m) shear pin was sheared and the test is stopped. Figure 13 displays the measured torque during the test for 5 performed tests.

As it is displayed in Figure 13 parts (a) and (b), adding soil conditioners to the sample from Indianapolis with 15% water content reduces the applied torque to almost half (from 81.4 N·m to 41.7 N·m). In addition, it should be noted that the weight loss of covers is reduced from 3.331 g to 1.448 g respectively.

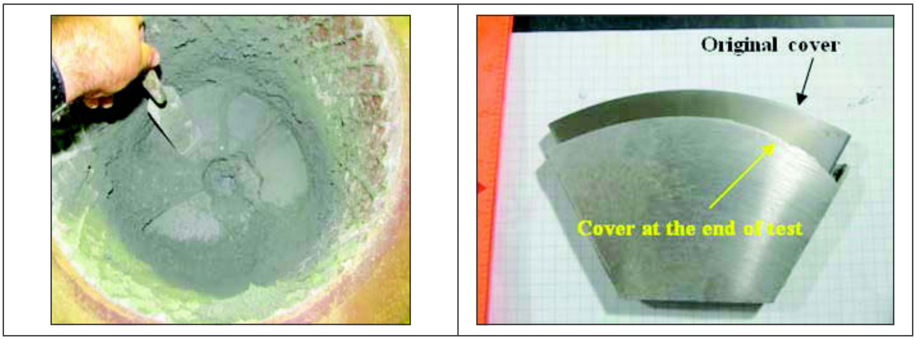


Figure 10. (a) Compacted material after 5.5 min abrasion testing on sample with w=10% from Washington, DC, tunnel; (b) comparison between the original cover and one of the covers after 5.5 min abrasion testing on sample from Washington, DC, with w=10%

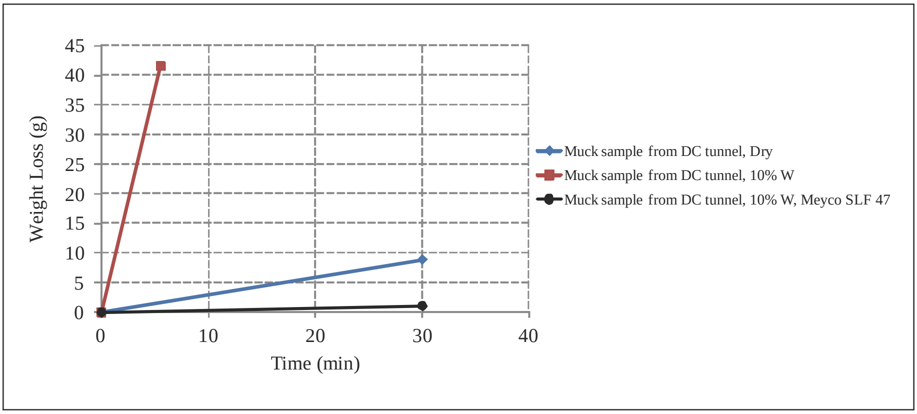


Figure 11. Results of testing with soil conditioner on Muck sample from Washington, DC, tunnel

Table 3. Soil conditioning tests performed on muck sample from a tunneling project in Indianapolis

Soil	Moisture Content (%)	Testing Time (min)	Weight Loss (g)	Conditioning Properties
Indianapolis Tunnel	0–Dry	214.5	2.005	—
Indianapolis Tunnel *	7.5	1476.48	—	—
Indianapolis Tunnel *	11	755.12	—	—
Indianapolis Tunnel	15	81.4	3.331	—
Indianapolis Tunnel	15	42.7	1.448	2% Conc. ABR 5, FIR=20%, FER=15

* No abrasion results are available due to high amount of torque and interruption of the testing

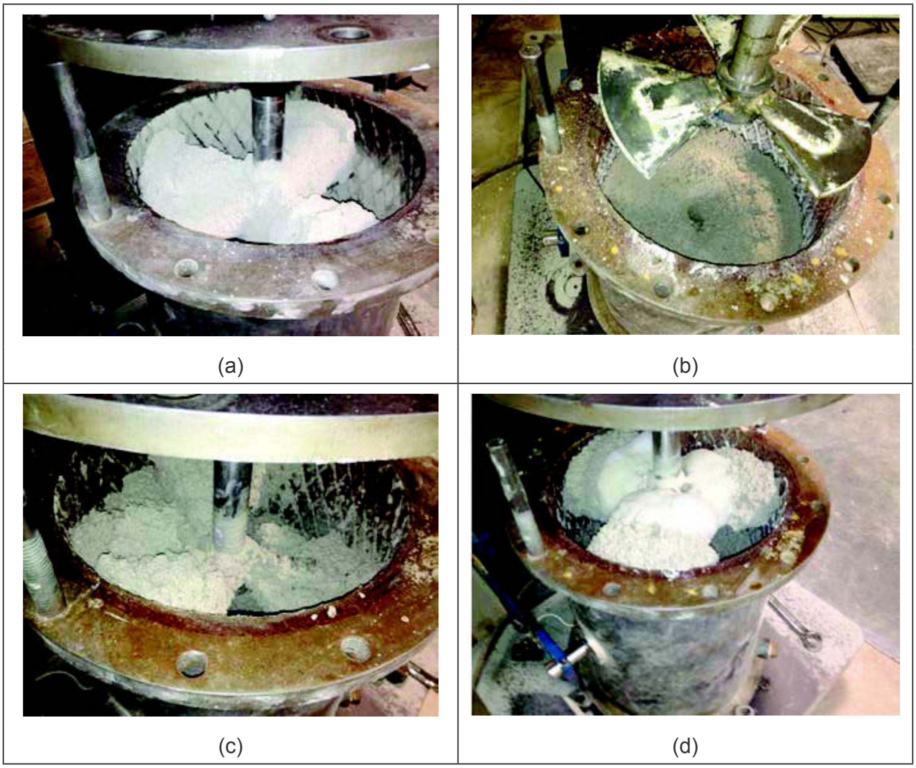


Figure 12. Condition of the samples after abrasion tests in (a) dry condition; (b) W=11%; (c) W=15%; (d) W=15% conditioned with 2% concentration ABR 5, FIR of 20%, and FER=15

CONCLUSIONS

The testing of various soil types by the soil abrasion testing device at Penn State University shows that the application of proper soil conditioning can reduce the abrasion, and hence the wear of the tools and inner parts of the tunneling machine as well as significant reduction of the required torque. While the results are much anticipated and confirms the main reasons for introduction and application of soil conditioners in soft ground tunneling, especially with EPB machines, the magnitude of the reduced wear and torque could not be easily measured in laboratory setting before. The initial testing shows that the reduced wear when the soil is properly conditioned could be an order of magnitude lower than the wear of the tools in slightly moist samples, just dry of optimum water content for compaction as measured by proctor test. In the case of hard rock tunneling machines, the muck could severely wear the cutterhead, specially that it is common practice to use a small amount of water for dust suppuration at the face which could be less than 5–10% of muck weight and could make the muck very abrasive. The result of test showed that in the case of the tunnel muck from Washington DC area the wear was reduced from 44 gram in 5.5 minutes in moist muck to around 1 gram in 30 minute of testing when proper conditioning was applied. This is a reduction by a factor of over 200 times. Similar results could be expected in terms of the torque reduction in sticky ground in soil or rock, where the torque could be reduced by over 50% when the muck is properly conditioned. Additional testing is underway at PennState to evaluate the impacts of FER and FIR on the abrasion and torque

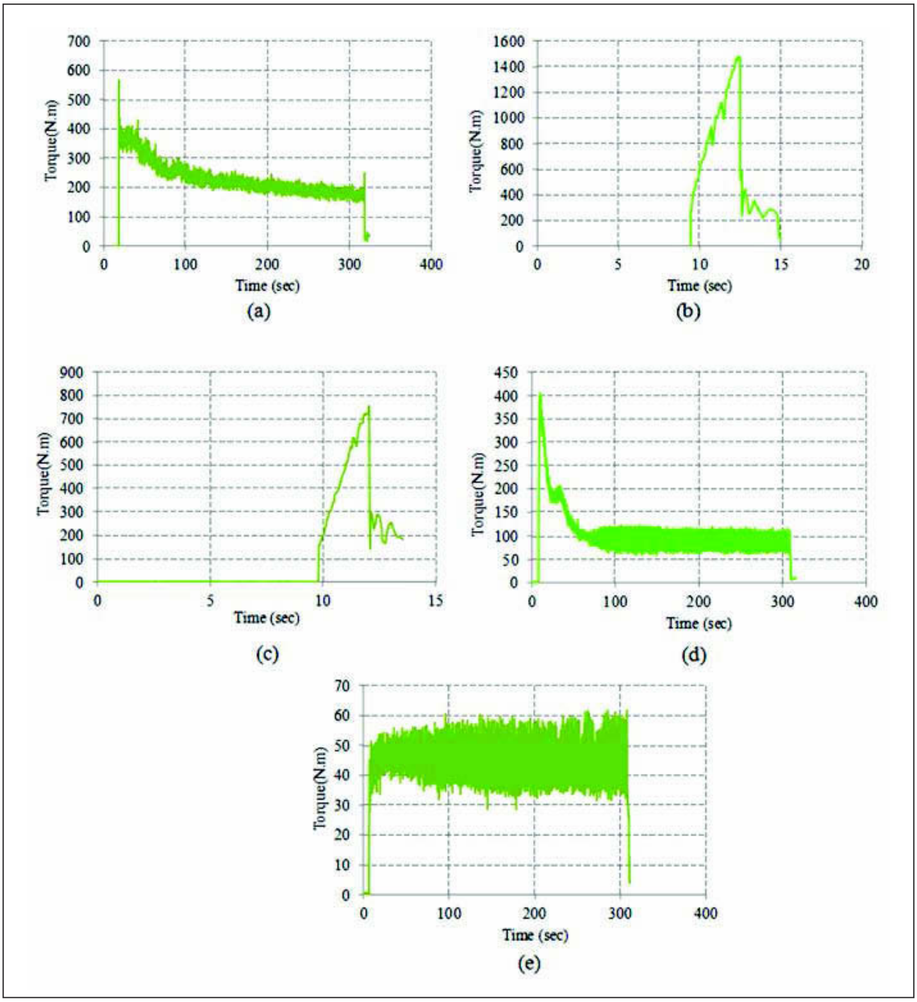


Figure 13. Measured applied torque during the abrasion testing for Indianapolis tunnel samples in (a) dry condition; (b) w= 7.5%; (c) w=11%; (d) w= 15%; (e) w=15% conditioned with 2% concentration ABR 5, FIR of 20%, and FER=15

reduction to establish a trend for fine versus course grain soil/muck and offer some practical solution for optimization of these parameters for improving the performance of various TBM machines in ongoing tunneling projects.

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KEEPING THE CHAMBER FULL: MANAGING THE "AIR BUBBLE" IN EPB TUNNELING

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ABSTRACT

Soil conditioners mixed with compressed air to create foam are often added to the confined pressurized environment of EPB TBMs in order to reduce cutterhead and screw torque and properly condition the excavated material. An air "bubble" at the top of the TBM excavation chamber can form if the compressed air is not entrained in the EPB Muck. The formation of such an air bubble can lead to a variety of problems such as: excessive drop in EPB pressures between tunnel advances, less stable face support and possibility of water and material flowing into the excavation chamber, over excavation and possibility of surface settlement, potential blowouts through the screw or to the surface, an increase in the chance of methane explosion by providing oxygen into the mix etc. In this paper, several key parameters such as muck apparent density, EPB pressures, geology, foam expansion and injection ratios are used as tools to identify the presence of a "bubble" in the chamber, in addition to practical procedures for eliminating "bubbles" and the conditions which contribute to their formation are presented.

INTRODUCTION

Shielded TBMs have almost become the exclusive tunneling method in soft grounds and soil. A variety of machines have been introduced in the past couple of decades for soft ground applications. These include the slurry shield TBMs and earth pressure balance (EPB) TBMs. Home (2010) estimated the total number of TBMS (Slurry and EPB) that have been used globally between 2005 and 2010 to be around 350 units. As is shown in Figure 1 and despite the variety of parameters influencing the selection of EPB versus Slurry TBMs, the use of EPB TBMs has grown rapidly when compared to slurry TBMs.

Conditioning the excavated material is one of the most important components in the operation of an EPB TBM. This "ground conditioning" provides many benefits including: increased face stability, improved workability of excavated material, reduction of friction and cutterhead torque, enhanced control of pressure inside the cutting chamber, better control of groundwater, improvements in the flow through the screw conveyor, less material adhesion during handling, reduction of wear and tear of the cutters, cutterhead, and other wear parts, and finally improving the safety of the personnel during the maintenance of the cutters/cutterhead (Milligan, 2000).

Soil conditioning is performed by injecting foam, polymer, water, and filler (bentonite) to the tunnel face, excavation chamber and screw conveyor. Selection of the type of foam and polymer mainly depends on soil type, geological condition (groundwater and soil permeability), and location of tunnel boring machine (injection points, open or closed cutterhead, type of foam generator, etc). The most widely used soil conditioners

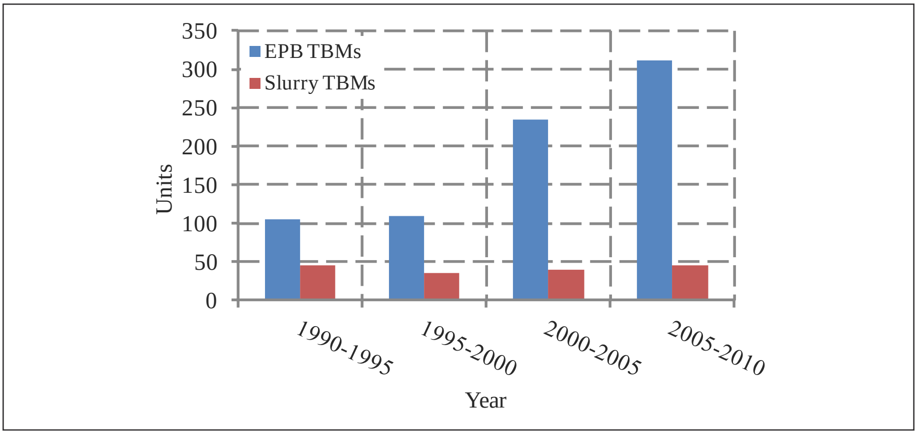

Figure 1. Number of EPB and slurry TBMs used globally between 1990 and 2010 (Home, 2010)

are foam and/or polymer and in some cases due to existing conditions, anti clogging or anti wear additives are also used.

Despite the development of several laboratory tests that investigate the different properties of foam and conditioned soil, there is a limited amount of research in the application of soil conditioners in the field. Parameters such as the foam injection ratio (FIR), foam expansion ratio (FER), agent to water ratio, location of injection ports on the cutterhead or inside the excavation chamber and screw, the designated volume of foam to be injected from each of these ports with respect to their location, etc. are among the practical settings that are mainly determined by the TBM operators or the field engineers experience. The majority of the tunneling industry seems to use soil conditioner settings such as FIR and FER based on the manufacturer's recommendations instead of objective test results.

When foam is mixed with EPB muck the foam is anticipated to be entrained within the muck in the form of tiny bubbles dispersed throughout the mass of muck. If the EPB muck is oversaturated with air bubbles some foam may not be entrained in the muck but instead percolates to the top of the excavation chamber. The excess accumulation of foam can create a "bubble" at the top of the cutterhead chamber. This pressurized bubble will support the face in a similar manner to traditional compressed air tunneling is used as a means of ground support. However the pressurized air, having very low viscosity can escape into the surrounding soil or through leaks in the TBM or along the shield. During the TBM advance, this may not be a serious problem since the foam is continuously injected into the cutterhead. However, during periods between advances, such as ring-build periods or downtime between shifts, weekends or holidays the escaping air can reduce the EPB pressures in the excavation chamber below those levels that are adequate to support the face. It is standard practice in compressed air tunneling to continuously add makeup air and great effort is made to provide multiple levels of redundancy in order to maintain operation of the air compressors that continuously supply makeup air. However, similar safeguards are not normally in place for EPB tunneling.

Formation of an air "bubble" at the top of the TBM excavation chamber can lead to a variety of problems most notably EPB pressure drops when tunneling is paused for extended periods between advances leading to inadequate face support and possibility of water and material flowing into the chamber. Following from the risks posed by precipitous drops in face support are: misinterpretation of ambient soil and water pressures which can lead to over excavation and possibility of surface settlement, potential blowouts through the screw and to the surface, an increase in the chance of methane explosion by providing compressed air rich in oxygen into the mix. These issues are not simply theoretical considerations; recently a methane explosion occurred inside the excavation chamber of Herrenknecht EPB TBM (2.56 m diameter) which had been excavating the Selimpasa waste water tunnel in Turkey. The explosion occurred at the beginning of the second shift after approximately 10–15 cm of excavation. The explosion caused the excavated material inside the chamber to be blown out of the screw conveyor followed by a fireball. The methane had accumulated inside the chamber for around 2.5 hours and the explosion was apparently initiated by sparks created by the friction between the screw conveyor and its casing. It was reported that at the start of the shift half of the chamber was full of muck and the oxygen-rich compressed air needed to create the explosive atmosphere had accumulated there due to foam that had collapsed and released the air during the stoppage (Tunneling Journal, 2012). This accident in addition to several similar scenarios shows the importance of carefully management of the accumulation of compressed air "bubbles" in the excavation chamber of an EPB TBM.

Further, a common practice in EPB tunneling is to calculate the appropriate EPB pressures that should be maintained while tunneling along the alignment prior to launching the TBM. These calculated pressures are then compared to the ambient face pressures that occur between TBM advances. The concept is that the EPB pressures should be above ambient pressures such that EPB pressures will increase while the TBM is advancing and that this EPB pressure will dissipate after the advance as the liquid component of the EPB muck equalizes with the ambient hydrostatic pressure in the soils. This concept has been presented in an article by Skelhorn published in the June/July 2011 issue of Tunneling Journal. "If the pressures as measured in the head during mining are higher than the pressures measured at rest, there should be no possibility of over-mining." If there is free air in the chamber in the form of an air bubble, this concept is at risk since the air being lighter much less viscous than water can escape will can allow the measured EPB pressure to drop below the hydrostatic. This event will likely show an EPB pressure trend that will initially drop after the tunnel advance, followed by a rise in the EPB pressure as groundwater replaces the escaping air and equalizes with the ground water table. The flow of water (and possibly soils) into to cutterhead to replace the escaping air may not provide adequate face control to prevent ground movements.

In this paper, the authors will discuss how they used several key parameters such as muck apparent density, EPB pressures, geology, FER and FIR as tools to help identify the presence of a "bubble" in the chamber. In addition, they will discuss the conditions which contribute to the formation and the practical procedures for eliminating "bubbles" or compressed air accumulations in the excavation chamber.

IDENTIFYING THE PRESENCE OF AIR BUBBLE IN THE CHAMBER

One of the important tools that can be used to capture the presence of an air bubble in the chamber is the determination of the apparent density of the excavated material in the chamber. In this case, apparent density is an interpolated parameter calculated from the face pressure sensor's data. By considering the relative height of the pressure sensors inside the chamber and the pressure difference between different sensors, the apparent density of material located at different elevations inside the chamber can be calculated. In the next section, a case study is presented in which the concept of apparent density was used to capture the presence of an air bubble in the excavation chamber.

University Link Light Rail Tunnel (U230) in Seattle, WA

The Sound Transit University Link project is a 3.15 mile light rail extension that will run in twin-bored tunnels from Downtown Seattle to the University of Washington, with stations at Capitol Hill and on the University of Washington campus near Husky Stadium. The project is broken up into several parts, where the U230 portion encompasses the excavation of Capitol Hill Station and installation of 0.73 miles of twin-bored tunnels from the Capitol Hill Station (CHS) to the Pine Street stub tunnel (PSST) in Downtown Seattle. The joint venture of Jay Dee Contractors, Inc., Frank Coluccio Construction Company, and Michels Corporation was awarded the U230 Project. They were responsible for the construction of twin-bored (northbound and southbound) tunnels each with an approximate length of 3,880 feet from CHS to PSST. The tunnels are lined with concrete segmental lining having an outside diameter of 20 feet 7 inches and an inside diameter of 18 feet 10 inches.

Geotechnical Properties of the Ground

The geological description of this project can be divided into fluvial deposits, glacial deposits, lacustrine and glaciolacustrine deposits, all of which have been glacially overridden and are therefore highly over-consolidated. The Soil Groups (SGs) defined for this project are Blue SG that represents over-consolidated fine-grained, plastic soils, Turquoise SG that represents over-consolidated fine-grained, non-plastic soils, Yellow SG that represents over-consolidated fine to coarse sand, with varying amounts of gravel, silt, and clay and Purple SG that represents normally consolidated fine to coarse sand, with varying amounts of gravel, silt, and clay (Purple SG is not present on the tunnel alignment and only encountered at PSS and CHS stations).

Specification of the EPB TBM

The Tunnel Boring Machine (TBM) for the U230 project was built by Hitachi Zosen. The TBM was 21 feet and 1.54 inches (6440 mm) diameter EPB Tunnel Boring Machine which was considered as a complete tunneling system with muck conveyor, lining erection system and integral power-packs. This TBM was made up of three main sections: a forward shell which contained the cutterhead and the main drive; a stationary shell which housed the propulsion system, steering articulation joint, and screw conveyor; and a trailing shield which contained the lining erection equipment, the tail shield to precast liner sealing system and the annulus grout injection components. Both direct drive electric for main drive and electro-hydraulic power were used in the TBM for excavation, conveying and lining erection. The cutterhead is designed to excavate through glacial till and outwash deposits including boulders based on the information provided in the U230 Geotechnical Baseline Report (U230 GBR, 2009). The cutterhead is designed with opening limitations to prevent the passage of rock fragments larger than approximately 18 inches (457 mm). The TBM has two 31.5 inch (800 mm) diameter ribbon type screw conveyors configured in series. This configuration allowed for optimum control of EPB pressures in varying soil conditions. All machine controls, instrumentation and monitoring devices were centralized on the operator's control panel. The operator's control panel was placed in a position which afforded an unobstructed view of the lining erection area. The cutterhead discharge was monitored via closed circuit television. The six earth pressure sensors were fitted through the bulkhead of the pressurized chamber and provided precise monitoring of face pressure as shown in Figure 2.

Calculation of Apparent Density for U230 EPB TBM

There were 6 "EPB" or total pressure sensors fitted to the excavation chamber bulkhead of the U230 EPB TBM with a pair installed in three different levels: upper, middle,

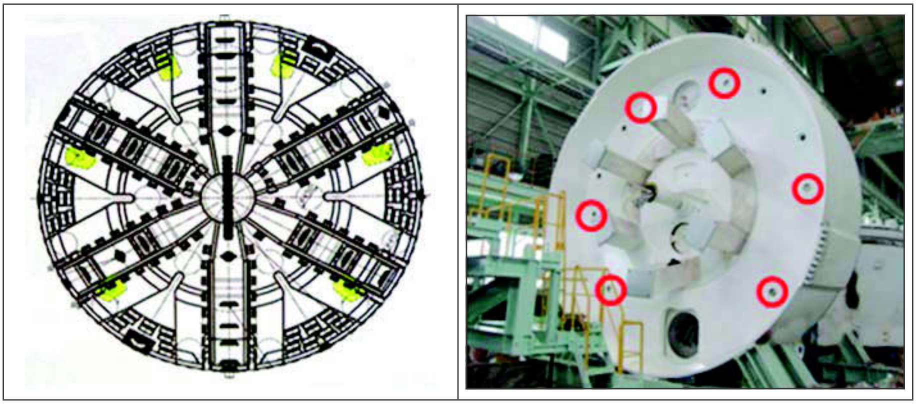

Figure 2. Location of the 6 earth pressure gauges

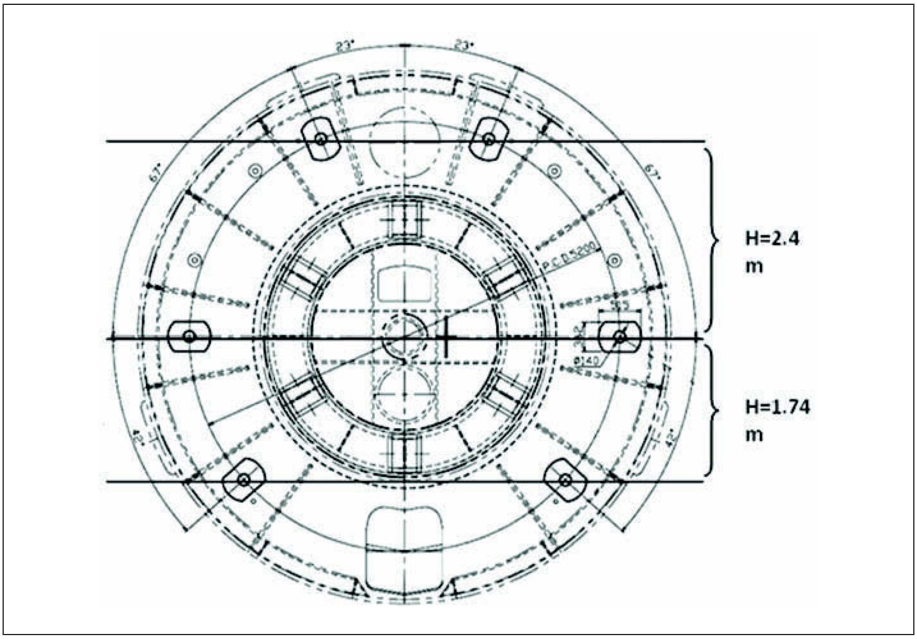

Figure 3. Schematic drawing of the location of 6 pressure sensors in the EPB chamber

and lower. The height difference between these three levels is indicated in Figure 3. The pressure difference or "delta"between the two levels could be obtained by subtracting the average pressure of two sensors in each two levels and then converted from bar to density by dividing the "delta" by 0.24. For example, if we assume that the chamber is full of water, then the pressure difference between upper and middle level and middle and lower sensors should be exactly 0.24 and 0.174 bar respectively since the specific gravity of water is 1 and according to Figure 3 the elevation difference between top and middle sensors is 2.4m. When the delta value is higher than 0.24 the density will be greater than water and this would indicate a degree of chamber filling. Conversely of if the value is below 0.24 this would indicate that a bubble has likely formed in the chamber.

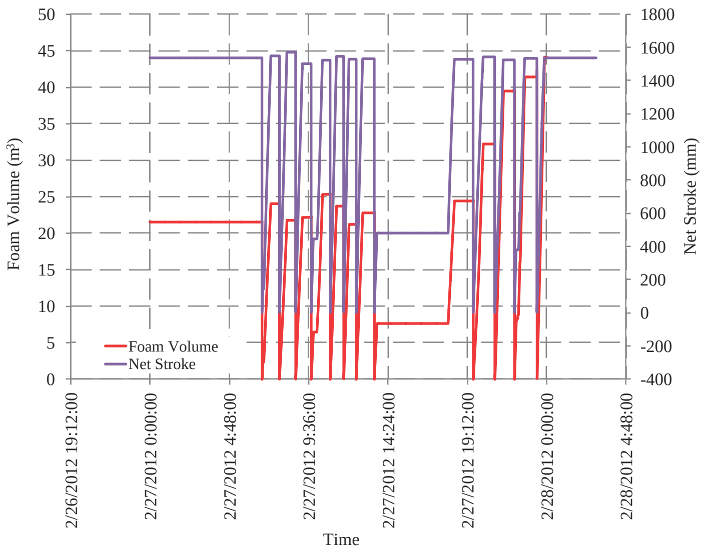

Figure 4. Net stroke versus foam volume for the study area

Capturing the Air Bubble

An example from the U230 project helps to illustrate the concept of identifying triggers which cause reduction in the excavated material apparent density. Figure 4 shows the jack net stroke versus the total used foam during the operation of the TBM. The tunneling operation is performed in two shifts. In the first shift, mining is being performed fairly consistently with no apparent problems and on average approximately 24 cubic meters of foam is injected into the plenum (Figure 4). Figure 5 shows the calculated apparent density in the area between upper and middle sensors and the area between middle and lower sensors as well as the average pressure in the mid part of the plenum during the day shift. The red line in Figure 5 displays the limit corresponding to the air bubble area. If the apparent density drops to less than 1, that is a sign that an air bubble has formed in the chamber.

During the night shift, the TBM operator increased the Foam Injection Ratio progressively in every push (Figure 4). Despite the decrease in the amount of torque and increase in the advance rate, a lot of foam was injected into the plenum and the density of the material in the plenum dropped significantly (Figure 6). This is an indication that the air bubble is formed and grew as it can be seen in Figure 6.

A simple example has been presented in this paper in which the air bubble formation risk can be monitored by using the data from the EPB sensors in the chamber and by considering the concept of muck apparent density. During the construction of the U230 project, a program was developed in which a apparent density plot was generated by extracting the pressure data from the Data Logger System during the excavation. This plot was used by the field engineer to reduce the risk of formation of an air bubble in the chamber.

PROCEDURE FOR ELIMINATING AIR BUBBLES IN THE CHAMBER

The U230 TBM featured several ports or bulkhead penetrations distributed both above and below spring line which could be used for various functions including removal of

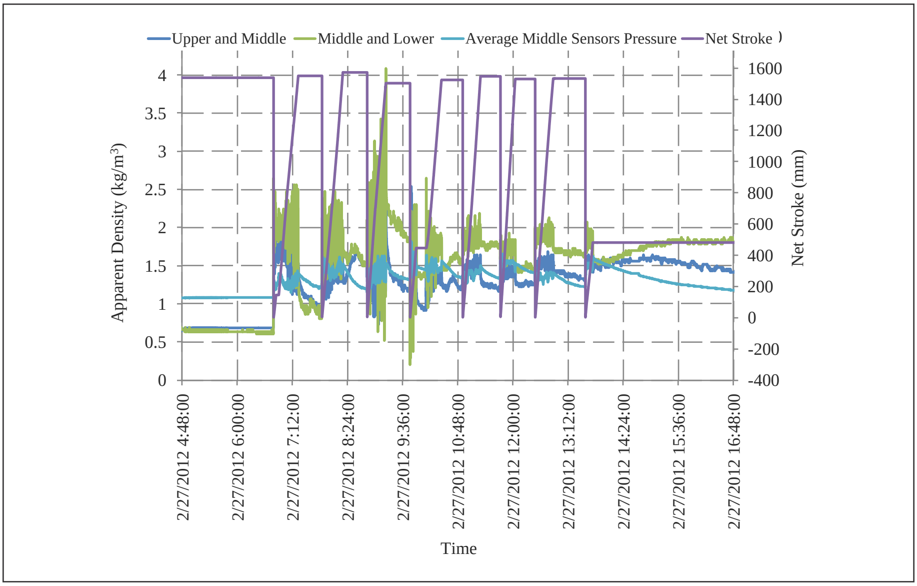

Figure 5. Parameters used to capture the air bubble in the chamber (day shift)

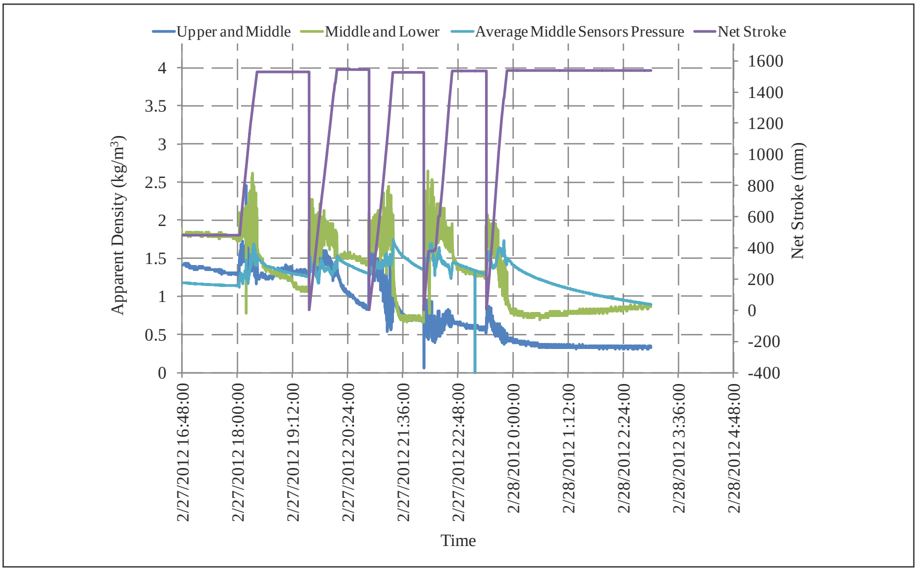

Figure 6. Parameters used to capture the air bubble in the chamber (night shift)

air bubble accumulations in the crown of the excavation chamber. One of the most straight forward approaches to remove these accumulations is by bleeding the air using a simple hand operated ball valve fitted to one of the ports adjacent to the crown of the excavation chamber. Based on the U230 experience it is recommended that when operating with foam ground conditioning in low permeability ground such as clay, the facility for venting should be standard practice while the excavation is in progress. The location where the air is exhausted from the excavation chamber must be properly ventilated and a gas monitoring device should be installed close to the outlet to ensure the air escaping is not contaminated with methane or other gases. Figure 7 shows one of the ports that was used for air bleeding in the U230 project.

In the case of the U230 TBM the venting of the excavation chamber was done manually. The worker responsible to bleed the air was kept in contact with the TBM operator and field engineer via mine phone. In this way, the air bleeding could be monitored and performed precisely under supervision of the field engineer.

Figure 7. Air vent port used for bleeding accumulations of compressed air on U230 project

Reducing the amount of applied foam into the chamber is one of the other ways that can be used in order to eliminate the unwanted formation of an air bubble. Further, the use of neat conditioning agent added to water and pumped directly into the tool gap without adding air to create foam can be another approach that has been successfully used in non-permeable ground in Seattle. This technique, along with the application of additional injected water into the chamber instead of foam can minimize the volume of compressed air required to effectively condition the materials encountered.

RECOMMENDATIONS FOR FURTHER DEVELOPMENT

The authors recommend further investigations into the identification and management of the air bubble development in EPB tunneling. There is little information regarding the limiting factors in the entrainment of air in EPB muck. The authors hypothesize that more air can be entrained in sands, silts and gravels than can be entrained in clays. This is based on observations of EPB muck in clay stratigraphy that occasionally shows discrete chunks of clay in a slurry of dissolved clay in water and foam. It would seem appropriate direction to develop a laboratory testing method to identify and quantify the boundaries of the maximum percent of air entrainment that can be achieved with each of the different soil groups; sands, silts and clays. This would provide guidance on the appropriate FIR and FER settings for the soil conditioning foams to minimize the potential for air bubble accumulation in each of these soil conditions.

Another recommendation for further development is for the TBM equipment manufacturers to develop a control process to automatically detect the presence of air bubbles and possibly automatically dissipate the air bubble by automatic discharge of the air from the top of the TBM plenum or annulus. Caution is suggested in this approach. It should be noted that the problems created by the development of an air bubble are predominantly during the downtime between TBM advances. Any air relieved should be replaced with stable material; by either injecting material with higher viscosity than air or water into the cutterhead chamber as the air is released or by advancing the TBM slightly to maintain EPB pressures as the air is released.

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UNTANGLING THE MYSTERY OF SOIL CONDITIONING IN EPB TUNNELING

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ABSTRACT

The Earth Pressure Balance (EPB) method has recently gained more prominence in excavating tunnels in complex geology under high hydrostatic head. Soil conditioning is one of the main aspects of EPB tunneling and if properly done it enhances the performance, economy, safety and maintenance of the project. While the benefits of Soil Conditioners (SC) are well established, excessive use might lead to adverse results. Theoretical knowledge currently available can only provide guidance but experience plays the greater role in identifying the correct dosage of SC to use for different soil matrixes. This paper presents the experience gained from several projects and explores the operator's perception of a well conditioned muck.

INTRODUCTION

Soil Conditioners (SC) are used in pressurized face tunneling to enhance the excavated soil characteristics which improves the ability of extracting the muck from the plenum. Although SC has always been used in tunneling and their benefits are well established, their proper usage is still a mystery. The limited theoretical knowledge currently available can merely provide guidance. Experience and onsite tests play the greater role in establishing the appropriate type and dosage of SC to be used in different soils.

Williamson et al. (1999) documented the inception and history of SC and presented some formulas to calculate the dosage of various SC in different soils. Anagnostou and Kovari (1996) studied the importance of soil conditioning in stabilizing the tunnel face. They concluded that one of the ways to lower the shear resistance of the excavated material is by using SC. Soil conditioning is done by injecting foam, polymer, water, or bentonite slurry in front of the Tunnel Boring Machine (TBM) cutterhead, in the plenum, and screw conveyor. Selection of the SC mainly depends on soil type, geological condition (groundwater and soil permeability), and configuration of the TBM (injection points, type of foam generator, etc). The most prominent SC are foam and polymer, however in some cases other additives like anti-clogging or anti-wear are used.

Throughout the pressurized tunneling industry, there are different opinions about the usage of soil conditioners. These opinions are usually based on different experience gained by different individuals. For instance, some people are of the opinion that water alone can successfully treat all types of soils and are against the usage of other types of SC in the EPB operation. Others have the same strong conviction about bentonite slurry. A third group generates their own formulas and conditioners based on their experience. While we can list few other opinions, the fact remain that the majority of the industry utilizes commercialized foams and polymers. In addition to different opinions about the usage of foams, the specification of applied foam is quite a mystery by itself. Foam Injection Ratios, Foam Expansion Ratios, and surfactant concentration are characterized differently amongst industry personnel. If you ask a TBM operator on how to characterize good foam, his answer would probably be "if the foam sticks to the palm of your hand while it is facing the ground that is great foam" (See Figure 1). This statement is possibly very true but it lacks the theoretical explanation and the actual dosages to reach this result, it is largely based on trial and errors. Furthermore, the composition of the surfactants is a tightly held secret by the manufacturers and is often disguised by trade names and thus these materials are not well understood. For all these reasons, engineers typically use the manufacturer's dosing recommendations.

Figure 1. Operator's idea of good foam

There are many advantages in using SC in EPB tunneling such as increasing the stability of tunnel face, improving the flowability of excavated material, transforming the muck to a uniform plastic soil, reducing the cutterhead torque, better control of groundwater, reduction of wear and tear of the cutters and cutterhead, and finally improving the safety of the personnel specially during maintenance interventions (Milligan, 2000).

THE IMPORTANCE OF SOIL CONDITIONERS IN EPB TUNNELING

To recognize the value of SC in EPB tunneling, the concept of this method has to be understood. The basic principal of the EPB method is that the excavated material itself is used to stabilize the tunnel face by counterbalancing the earth pressure. As the TBM advances at the face, the cutterhead excavates the ground in front of it and the excavated spoils are then stored and controlled in a pressurized chamber (plenum) located behind the cutterhead. During the excavation process, a screw conveyor extracts the excavated material from the plenum in controlled volumes. Synchronizing the screw conveyor speed with the rate of advance of the TBM and equalizing the volume of materials entering and exiting the plenum establishes earth pressure balance during the excavation process. Figure 2 illustrates the EPB concept.

In order for the EPB system to work properly, the excavated material has to be homogeneous, impermeable and in a plastic form. In situ soils rarely have these characteristics which prompted the need to enhance its properties. SC are introduced to the excavate soils to achieve this goal. SC are usually injected in front of the cutterhead and enter the plenum with the excavated soil. With the rotation of the cutterhead, the SC mixes with the soil and form an altered soil matrix having the desired consistency.

TYPES OF SOIL CONDITIONERS

There are numerous type and brands of soil conditioners ranging from a material as simple as tap water to a highly complex chemical composition. Nonetheless, the majority of the SC used in the EPB tunneling method can probably be grouped under the four major categories listed below.

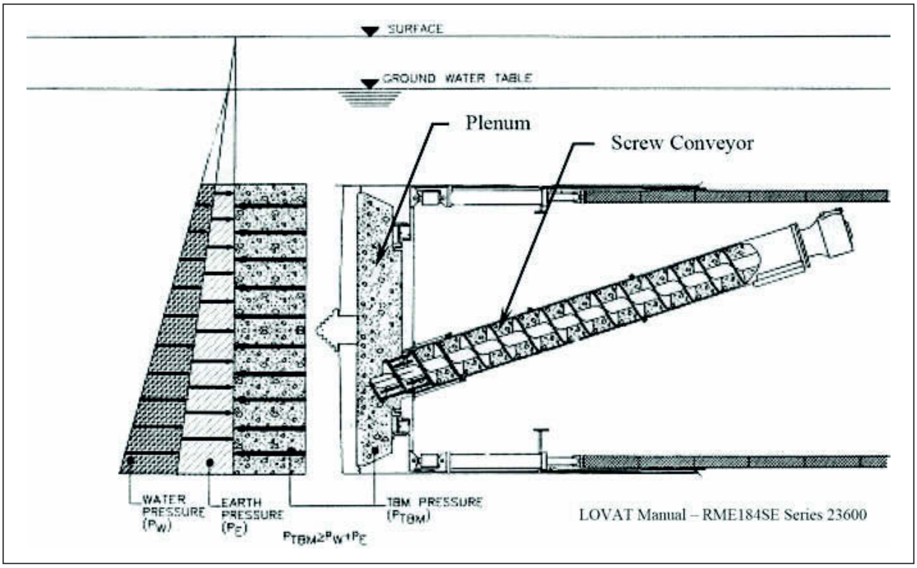

Figure 2. Earth pressure balance concept

Water

Water is the most basic form of SC used in EPB tunneling and while some might believe that it is enough to condition the excavated material, it is the authors' opinion that it is not sufficient by itself. Water lacks the structure to keep the particles in suspension in coarse soils and requires prolonged mixing time to be absorbed in clayey soils. It also does not help mitigating the wear on the cutterhead and in some cases it actually can increase the wear (Rostami et al. 2012; Alavi Gharahbagh et al. 2013). Nevertheless, it is the primary ingredient in all other SC. Water can also be a great supplement to other SC for dry soils by adding it directly to the plenum. Since it does not dissipate, it adds to the volume of spoils and disposal costs.

Bentonite Slurry

Through the early advancements of EPB tunneling, bentonite slurry was mixed with the excavated soil in the plenum to create a modified soil matrix. The low permeability of this modified soil allowed for the ability to counter balance the pressures at the face and its plastic flowability made it easy to extract through the screw conveyor. The injection of the bentonite slurry in front of the TBM face was the first type of SC utilized (Williamson et al. 1999).

While this type of SC can work very well in coarse grain soils, its benefits are limited for fine soils. However, the main disadvantage of using bentonite slurry is that it introduces large quantities of extra material that has to be removed and disposed with the excavated muck leading to added costs that can be significant.

Polymers

Polymers are mainly used with coarse grained soils to prevent flocculation and create a plastic matrix that helps removing the material from the plenum. Although bentonite slurry can be used for this purpose, a significantly smaller quantity of polymer is required to achieve similar results. When mixed with water, polymers essentially form long-chain molecules and keep the soil particles in suspension.

Polymers can be used on its own as an agent or mixed with foam and slurry as an additive. Utilizing polymers as an agent during tunneling through granular soils with high hydrostatic pressures can be very effective in controlling the water inflow. They have the capacity of absorbing large quantities of water and make spoils easier to handle. They are typically added to the mixing chamber (plenum) where the rotation of the cutterhead allows them to bind with the water, the resulting matrix has the capability of keeping gravel, cobbles and even small boulders in suspension.

Contrary to the common believe that polymers can be added in the screw as a last resort to control water inflow, the authors' experience indicates that TBM screw conveyors do not have sufficient mixing capability. This can result in strings of polymer coming out with the uncontrolled spoils that are very slippery and may create unsafe conditions for workers in the heading.

Foam

Utilizing foam as a soil conditioning agent was first done in the early 80s. At the time of its inception, it was considered a breakthrough in the EPB tunneling method (Williamson et al. 1999). It can arguably be said that the use of foam helped popularizing EPB tunneling and allowed it to be utilized in ground conditions beyond its original boundaries. Currently foam is the most dominant type of SC used in EPB tunneling method because it carries several advantages over the other types of conditioners.

Foams are inherently metastable and will disperse over a period of time which is considered one of its main advantages over the slurry since it does not produce extra material for disposal. Foam also proved to play a significant role in reducing cutting tools wear (DiPonio et al. 2007; Shinouda et al. 2011; Alavi Gharahbagh et al. 2013). Furthermore, foams can be used in a wide spectrum of soil types which make them very versatile and with the recent development of additives like anti-clays, its scope of application has been broadened even more. Realizing its importance in EPB tunneling, the authors devoted their efforts in this paper to present their successful experience with foam as a soil conditioning agent.

ANATOMY OF FOAM

Foam is the product generated by mixing a surfactant with water and introducing compressed air to the resulting mixture. While this concept sounds simple, identifying the mixing ratios to achieve the optimum foam for each soil type is challenging. In order to look at the subject in more detailed manner, several definitions need to be addressed:

- Foam: is defined as a product generated from the combination of a foaming solution and air.
- Foaming Solution: is basically a mixture made from water and surfactant.
- Foam Expansion Ratio (FER): is the ratio between the volume of air at working pressure and the volume of foaming solution.
- Foam Injection Ratio (FIR): is the ratio between the injected volume of foam at working pressure and the bank volume of excavated soil.

Foam is basically air dispersed as bubbles in water resulting from turbulent mixing of foaming solution and air. As the excavated material gets mixed with foam in the plenum, soil particles disengage from each other and foam occupies the gaps created. The surfactant coating the bubbles' membrane acts as a lubricant and thus reduces the surface tension between soil particles.

Good quality foam has to possess two main characteristics, stable structure and consistency otherwise it will prematurely disintegrate. Foam has to survive the EPB TBM excavation process but should dissipate before transporting the spoils. The stability of the foam structure is directly related to the strength of the bubble, smaller size

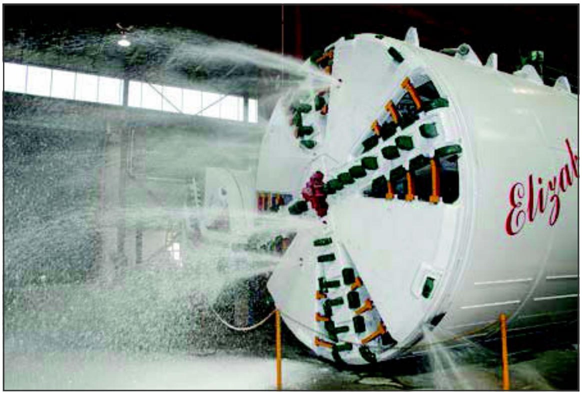

Figure 3. Demonstration of foam injection from TBM cutterhead injection ports

bubbles are inherently stronger than larger ones. Also polymer additives could be used to reinforce the bubble membrane. Consistency on the other hand is the uniformity of the bubble size, foam with uniform bubble size will yield a homogeneous soil paste. Furthermore, bigger air bubbles tend to encapsulate the adjacent smaller ones producing an even larger weaker bubble which leads to an unstable foam structure.

The foaming solution is a mixture of surfactant and water, the surfactant concentration depends on soil type, foam type and brand. Manufacturer's recommendations typically range from 1–6% depending on the foam type. An EPB TBM usually has a foam solution storage tank with enough capacity for two excavation cycles. During the excavation cycle, solution and compressed air are pumped through foam generators where foam is produced and pumped directly to the injection ports at the face of the machine. Figure 3 shows a demonstration of foam injection from the TBM cutterhead injection ports. Bubble size is strictly controlled by the foam generators. It is also worth noting that the foam generators should be mounted as close as possible to the face since foam degrades rapidly if pumped for long distances.

FER and FIR are two parameters that are frequently adjusted by the operator. FER is established by the volumetric ratio of solution versus compressed air pumped through the foam generator. FIR on the other hand is controlled by regulating the amount of foam injected into the ground. EFNARC (2005) specified a range of 5–30 for FER and 10–80% for FIR. It is the authors' experience that FER of 6–20 is sufficient which is well within the specified range but the FIR can sometimes exceed 100%. It is to be noted that foam properties are determined in atmospheric pressure but actual conditions are usually under several bars of pressure. Since foam is a compressible material because it contains air, this fact has to be taken into consideration when establishing FER and FIR values.

As it is evident from this write-up and several others, foam is very favorable conditioner if used properly but if misused it might lead to adverse repercussion that sometimes can be severe. Especially if an Air Bubble formed in the excavation chamber, this issue is discussed later on in this paper.

PROJECTS GEOLOGY

The work presented in this article is the cumulative experience gained from excavating three tunnels using EPB Tunnel Boring Machines. The geotechnical conditions of all three tunnels were defined by over-consolidated glacial geology. The BWARI tunnel is located in Columbus Ohio, the majority of its alignment was comprised of either fine-grained non-plastic soils or Till. Coarse sand and gravel were encountered in less than ten percent of the drive. There was also a significant amount of boulders along the tunnel path. The Brightwater project is located in Seattle Washington, more than sixty percent of its alignment was fine-grained plastic soils, about sixteen percent was fine to medium sand with varying amounts of gravel silt and clay, and a small portion of the drive was fine-grained non-plastic soils. Boulder concentration was significantly less than the BWARI project. The Sound Transit ULink (ST U230) tunnel is located in Seattle Washington, the majority of its alignment was fine-grained plastic soils, about ten percent was fine-grained non-plastic soils, and less than ten percent of fine to coarse sand. In addition, the TBM was driven through ground improved zones and Controlled Density Fill (CDF) blocks. While foam was used as the primary SC throughout the entire lengths of all these projects, polymers were introduced in some instances to control the water inflow.

EXPERIENCE ON THE USAGE OF DIFFERENT ADDITIVES

BWARI Project

On the BWARI project an in-house generated foaming solution was used, experience gained from a previous project was applied to establish the mixing formulas. The foaming solution comprised of surfactant, water, and cellulose which is a form of polymer that was used to reinforce the structure of the solution. A surface mixing plant was utilized to mix and store the solution. The solution was then transferred to holding tanks on the TBM and used to produce foam by introducing compressed air to it through a foam generator. The use of foam was very successful in controlling the face pressure and mitigating the wear of the cuttering tools. This became evident when foam was accidentally not delivered to the face due to swivel leak, an incident that resulted in a ripper wear rate three times higher compared to the rest of the project (Shinouda et al. 2011). This reinforced our confidence in using foams on succeeding projects. Using the cellulose (polymer) as an additive was also essential in controlling the water inflow at some locations.

Although foam quality was very good, tunneling through the Till soils resulted in significant wear on the cutting tools which resulted in the authors' belief that an anti-abrasion additive should have been used which was not easy due to the configuration of the foaming plant. This reason together with the frequent clogging and breakdown of the plant diverted our attention to the commercial foams on the other two projects. Also the cost of manufacturing the plant played a role in the decision. Having this experience, it was decided to use foams and additives for soil conditioning in Brightwater and ST U230 projects. Several foam manufacturers studied the geology along the tunnel alignment and provided their recommendation regarding types and dosages of foams/additives that should be used for each soil group. This process involved mixing tests on soil samples extracted during shaft excavation as well.

Brightwater Project

At the initial stages of tunneling in the Brightwater project, performance of numerous types and brands of foams were studied. Ultimately foam and anti-clogging additives were selected as the best SC to breakdown the stickiness of clay. Very sticky clay was frequently encountered along the alignment of this tunnel, clay would form a cake on the cutterhead face that would encompass the cutting tools (rippers). This would prevent the rippers from cutting the ground in front of the TBM and significantly increase the thrust force which would then reduce the advance rate. The excavated material would exit the screw conveyor in big chunks of clay and although it appeared to be covered with SC on the outside, it was very well compacted and unconditioned clay on the inside. The first occurrence of these circumstances prompted the use of foam impregnated with anti-clogging additive.

In one instance the excavated soil (sticky clay) bridged-over the entrance of the screw conveyor in the plenum, this prevented the spoils at the top of the chamber from being extracted through the screw. The excavated material that was trapped at the top half of the plenum became very compacted and the TBM would not advance. It is worth noting that at this point the top EPB pressure cells were reading a higher value than the bottom ones, an uncommon situation but is understandable in this case. After inspecting the conditions in the plenum, it was noticed that the muck was overly dry. Due to limitations on the flow capacity of the TBM SC system, increasing the foam injection ratio was not possible therefore it was decided to inject water into the plenum through a valve in the bulkhead. Anti-clogging foam and water were used through the majority of this project. Conditions did occasionally require the use of polymer, however this was to reduce the permeability of the muck and had little wear reduction effect (Frank et al. 2010).

Sound Transit ULink Project

The ST U230 project comprised of twin tunnels that are part of the city's transit system. An EPB TBM was used to excavate the northbound (NB) and southbound (SB) tunnels of this project. During the planning stages of the project, Polymer reinforced foaming agent was considered for the fine to coarse soils and fine-grained non-plastic soils and anti-clogging additive was considered for the fine-grained plastic soils. But after a short trial with the anti-clogging additive in the NB tunnel, the operators opted out to using the polymer reinforced foam for the rest of the project. This was probably due to the fact that changing the type of SC during tunneling is time consuming and with the changing nature of the ground, this had to happen frequently. It would be beneficial to consider allocating a space for two types on SC on the TBM trailing gantry, a fact that was overlooked during the design of this machine. For the most part the chosen type of foam was adequate for the job except for one location in the SB tunnel when sticky clay was encountered. Nonetheless, this problem was resolved by increasing the FIR and adding water.

The tunnel alignment passes under a major highway, an area that was stipulated to be a settlement sensitive zone. Two CDF blocks were constructed along the alignment of Northbound and Southbound tunnels at the highway location with the length of 110 and 120 ft respectively. Since the compressive strength of the CDF was very high, the operators used a large amount of SC. Tunneling through these blocks was significantly slower, excavation time for a five foot push increased from 15 minutes to 60 minutes. With the added quantities of SC and slower advance, the extracted spoils were very wet and issues related to material handling and transportation was raised. In order to reduce the water content of the material, polymer was injected into the plenum. Field mixing tests were performed and despite the fact that it was an experimental phase to determine the amount of polymer needed, the results were impressive and the water content of the muck was reduced. After running several experimental tests, the optimum polymer injection ratio was determined and applied successfully to the ground. Figure 4 and Figure 5 show the untreated and treated soil sample respectively.

IMPORTANCE OF OPERATOR'S INPUT

Since the proper use of SC in EPB tunnels is largely based on experience, it was logical to realize the importance of the operator's perception of the subject matter. During tunneling on the ST U230 project an effort was made to have extensive discussions with the TBM operators regarding their experience and understanding of SC and their use in EPB tunneling. It was enlightening to see things from their point of view and the open dialogue paved the road to reach conclusions that helped bring the project to a successful completion. This section presents the operators view on some aspects of SC.

Figure 4. Untreated soil sample—50% saturation

Figure 5. Soil sample mixed with polymer

All of the operators agreed that the reason for using the foam is to manage the muck, control the face pressure, and to reduce the wear on the cutters. But it was interesting to learn that they rank those choices differently, as a matter of fact the majority of votes went to reducing the wear rather than conditioning the ground. They stipulated that different types of SC can be used to manage the muck and face pressure but the added value of the foam is its ability to mitigate wear.

The operators judge the effectiveness of their conditioning settings by the values of torque they are seeing on the cutterhead and screw conveyor. Nevertheless, they also check to see the consistency of the muck coming out of the screw. Some of the operators think that the torque can be managed by the cutterhead rpm and thrust force regardless of the foam quality.

Although the operators agree that the type of soil dictates which type of foam should be used, they really dislike the idea of changing foam types due to the fact that it is time consuming. The idea of good foam in their point of view is that it should have a consistent strong body that can be held in the hand without running away and its bubbles do not break down quickly; basically it should look like “shaving cream.” In general the authors think that this might be a quick practical method to visually inspect the appearance of the foam but should not be the bases for characterizing the quality.

Foam Expansion Ratio (FER) and Foam Inject Ratio (FIR) are two parameters that the operators frequently change during mining depending on the ground condition but they seldom change the surfactant to water ratios. Based on the operators’ opinion, for clay material, FERs between 12 and 15 percent should produce “good” muck, but in sand higher values of FER (about 18 percent) could be used. Usually the operators set the FER depending on the soil type and change the FIR based on the consistency of the muck. In addition, the operators believe that water should be added during the mining in clay soils, but not in sandy or gravely ground because it will reduce the efficiency of the foam. For sand and gravel ground bentonite should be used in order to transform the muck to a plastic state.

Operators correctly believe that bentonite can be used during the excavation of different types of soil. For example, in clay it helps in reducing the “stickiness” characteristic of the clay and in sand or gravel it help with the consistency and conveying of the muck. But this defies the whole purpose of using foam which dissipates and thus eliminate the extra spoils that need to be disposed off. It should be noted that some operators use bentonite only to support the face during downtime (i.e., weekend or cutterhead interventions). Operators also strongly believe in the ability of polymers in controlling water inflow. They believe that polymer should be added in the plenum in small doses and it should never be injected through the foam generators because it clogs them.

One of the great management plans that was implemented on the ST U230 project was the TBM Operation Weekly Meetings. During the meeting, group of engineers that analyzed the implemented operational parameters of the TBM during the previous week excavation time frame discussed their findings with the TBM operators and mechanics. Type of soil conditioning, FIR, FER, volume of injected water, grout volumes, grout pressures, etc. were among the discussed parameters. Operators would then share their experience and provide their input on how to solve any arising issues. In addition, the type of geology and the issues that may come up during the next week period of tunneling based on the GBR and GDR were discussed in the meeting. These meeting proved to be very successful in managing any possible issues in the best possible way and most importantly, it helped operators of different shifts to unify their methods to effectively advance the machine through the ground.

MANAGING THE AIR BUBBLE IN THE CHAMBER

One of the problems that can happen during EPB tunneling is the accumulation of large quantities of air at the top of the plenum. This air accumulation, if it occurs, is labeled “Air Bubble.” The EPB method depends on the material inside the plenum to counterbalance the face pressures. This mandates that the plenum be always full of relatively uncompressible materials, this is not the case if an Air Bubble is formed.

The formation of an Air Bubble in the plenum of an EPB machine is always a risk and can lead to a whole spectrum of undesirable consequences. It can result in an inadequate face support and uncontrolled flow of materials into the plenum, this would amount to over excavation and ultimately the possibility of surface settlement. Also a potential blowout through the screw is imminent if the ground in front of the machine is impermeable. However, the greater risk is the safety of the personnel in the heading. The oxygen contained in the Air Bubble could just be the required catalyst to cause a methane explosion as was the case in one of the recent tunneling accidents in Turkey (Copura et al. 2012). It should also be noted that the existence of the Air Bubble will

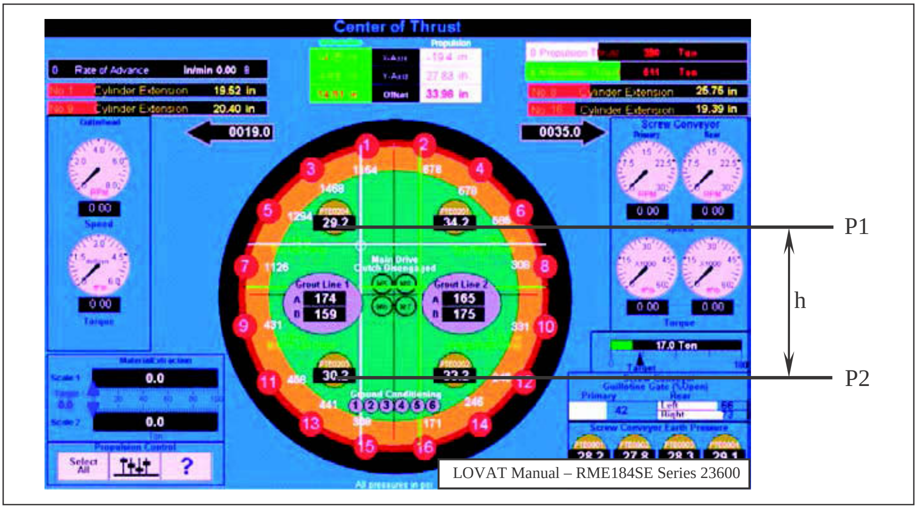

Figure 6. Operator's screen display of EPB machine

produce erroneous data from the bulkhead pressure sensors and unexplained pressure spikes, both are factors that would impair the ability of proper EPB tunneling.

Foam could be a major contributor to the Air Bubble. Foam contains a large quantity of compressed air and if it prematurely dissipates, this air will separate from the foaming solution and accumulate in the plenum. While this emphasizes the importance of producing foam that is stable for its intended life cycle, it also indicates that excessive use of foam can lead to adverse effects.

It is imperative to promptly manage the Air Bubble to reduce its risks and the first step is to detect its presence. One of the best ways to make sure that the plenum is free of the Air Bubble is by identifying the "apparent density" of the material in it and compares it to a minimum threshold value. Since the plenum is inaccessible during tunneling, calculating the actual density of the material cannot be accomplished. Nevertheless, the "apparent density" can be calculated utilizing the data collected from the bulkhead pressure sensors and their relative locations to each other (Alavi Gharahbagh et al. 2013). Figure 6 illustrates the operator's screen display of an EPB TBM.

Since water is probably the least dense material that can exist in the plenum, its apparent density should be established as the minimum threshold value. It is to be noted that this issue is complicated and deserves to be presented separately in more details thus the discussion here is not intended to be conclusive but rather briefly shedding some light on the topic. The authors' experience suggests that the apparent density should be considered as one of the real time monitoring parameters of the operation. A more detailed study on this subject is presented in Alavi Gharahbagh et al. (2013).

CONCLUSIONS

This paper summarized some of the valuable experiences that were gained during the tunneling operation on three EPB projects in the United States. The main findings of this paper can be summarized as follow:

- Foam dissipates over time which eliminates any extra materials to be transported, this reduces the costs of shipping spoils offsite. Foam has a great ability to mitigate wear of TBM components. These two advantages make foam superior to other SC.
- The Air Bubble in the excavation chamber can be catastrophic if not managed properly. Calculating the Apparent Density can be a great tool in identifying the existence of an Air Bubble and can be easily implemented as one of the real time monitoring parameters of the operation.
- The capacity of the foam delivery system should be in excess of anticipated quantities needed. While this might sound as a given fact, it is sometimes cost or space prohibitive to accomplish. But it should be noted that if the injected foam quantities are not sufficient to condition the soil, it might be imperative to reduce the TBM penetration rate which would affect production and sometimes not advisable while tunneling through some soil types. Also, it is advisable to have the ability of retaining two types of foam on the TBM with an easy method of switching between them.
- Operators' participation in defining and understanding the foam parameters is essential for the success of the operation. Identifying those parameters is largely experimental and the operators have "hands-on" experience and they possess the knowledge of how the TBM reacts in different grounds. Combining their practical experience with the experimental tests and theoretical knowledge would be a great advantage not only in controlling the soil conditioning operational parameters but rather to the whole operation of the EPB TBM. It is strongly recommended to allocate time for a weekly meeting with the operators and mechanics to discuss the progress of tunneling and identify any possible improvements.
- Polymers should not be injected in front of the machine but rather pumped directly into the plenum. Small quantities of Polymer can be sufficient to control water inflow given enough time to mix properly in the mixing chamber. Adding polymers in the screw conveyor is not beneficial in stopping water inflow. In is also worth noting that polymer should not be passed through the foam generators because it can easily clog them.

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EAST SIDE ACCESS—QUEENS BORED TUNNELS ENGINEERING CHALLENGES

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ABSTRACT

East Side Access Contract CQ031, in Queens, New York, involves the construction of 10,500 feet of pressurized soft ground tunnels, beneath rail yards and mainline tracks in the Sunnyside Yards. The project was awarded to GTF, the Joint Venture of Granite Construction Northeast, Inc., Traylor Bros., Inc. and Frontier-Kemper Constructors, Inc. (hereafter referred to as 'Contractor') and was managed by New York's MTA Capital Construction Company (hereafter referred to collectively as 'Owner'). The Designers of the project were (a tri-venture of Parsons Brinckerhoff, STV and Parsons Transportation). Planning the work for the Queens Bored Tunnels and Structures was a monumental task. The four 22-ft diameter tunnels were constructed through soft ground under numerous active rail tracks in the Queens Sunnyside Yard. Along the way, the Contractor engaged with the Owner and its Designers to solve numerous issues as they arose. Concepts specified during bid time were re-evaluated when problems occurred, and alternate ideas were implemented. A 'Case Study' paper, published in these proceedings, describes the project in general, whereas the following paper touches on some of the challenging tasks engineers had to deal with for the project to be a success.

INTRODUCTION

Slurry TBMs were chosen by the Contractor. This type of tunneling machine has a proven record for accurately controlling and maintaining face pressure as it mines a tunnel, and it is also able to mine through variable ground types such as rock, mixed face (rock and soils) and soils. Technical discussions related to the start-up of tunneling and the impact of cement contamination in the slurry system generated some concerns. When a Slurry TBM encounters cement, the slurry can coagulate and may not exhibit the essential properties required to adequately support the ground.

LAUNCH BLOCKS

Often during a tunnel start-up phase, in the first 15 meters (50ft) or so, ground can be lost due to a number of factors, and the consequent result can be surface settlement. In order to mitigate this risk, the Contract specified that the soil outside of the Launch Shaft, where the top three soft ground tunnels exit, be 'treated' to increase its strength and reduce its permeability.

Since ground treatment was jet grout, which utilized high volumes of cement, slurry properties would deteriorate, and therefore, it was necessary to form a block of sufficient size and strength to support the ground pressures. It was determined that a cross section of two tunnel diameters was required and the length needed to be equal to the length of the TBM shield, plus half a diameter in front of and half a diameter behind the TBM. The TBM for Queens was approximately 6.9m (22.5ft) in diameter and 10.7m (35ft) long, so the Contractor recommended dimensions of 13.7m × 13.7m × 17.4m

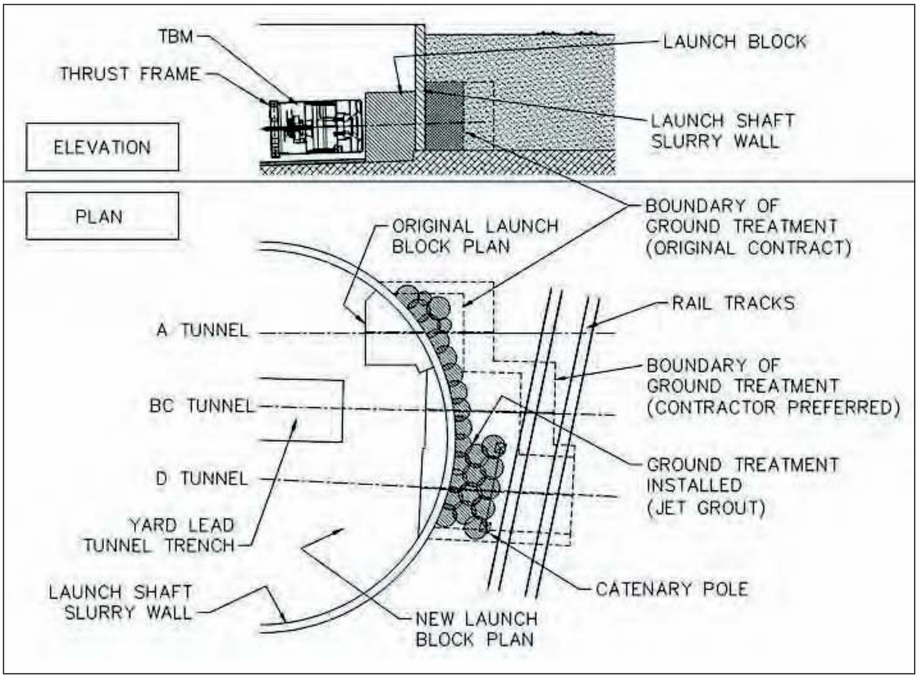

Figure 1. Launch shaft ground treatment

(45ft × 45ft × 57ft), and considered minimum acceptable dimensions to be 13.0m × 13.0m × 16.8m (42.5ft × 42.5ft × 55ft). The rationale being that 2 rings (3m or 10ft) of grouted tunnel liner behind the TBM and 3m (10ft) of treated ground in front of and around the TBM provided sufficient security. The proposed blocks were larger than those identified in the original Contract scope. See Figure 1.

The ground treatment (jet grouting) work began in the area outside of the Launch Shaft, but issues arose. Once the work moved into the rail storage yard, the time available for production was significantly reduced as there was limited availability of mandatory rail support crews. Progress was slow, and it became apparent that the cost for rail support resources and the cost for delays associated with the lack of rail support would be significant.

With ground treatment, there is uncertainty associated with how extensively the soil has been stabilized as you cannot see the final result. Coring and testing is the only means to ascertain whether it is adequate.

When all the risks were assessed and considered, and cost and schedule impacts for Ground Treatment were projected, it became apparent that an alternative solution to mitigate the risk of surface settlement was warranted.

Analysis of Alternative Risk Mitigation Measures

A future contract, CQ032, consisted of constructing a concrete structure within the Launch Shaft. Upon studying the preliminary design, it appeared feasible to construct part of the CQ032 structure inside the Launch Shaft ahead of time, to create concrete launching chambers, or Launch Blocks, for each TBM. With these Launch Blocks in place at tunnel startup, the TBM could be parked inside and sealed behind so that mining operations could start immediately in full pressurized closed mode, and the jet grout outside the Launch Shaft could be eliminated. See Figures 2 and 3.

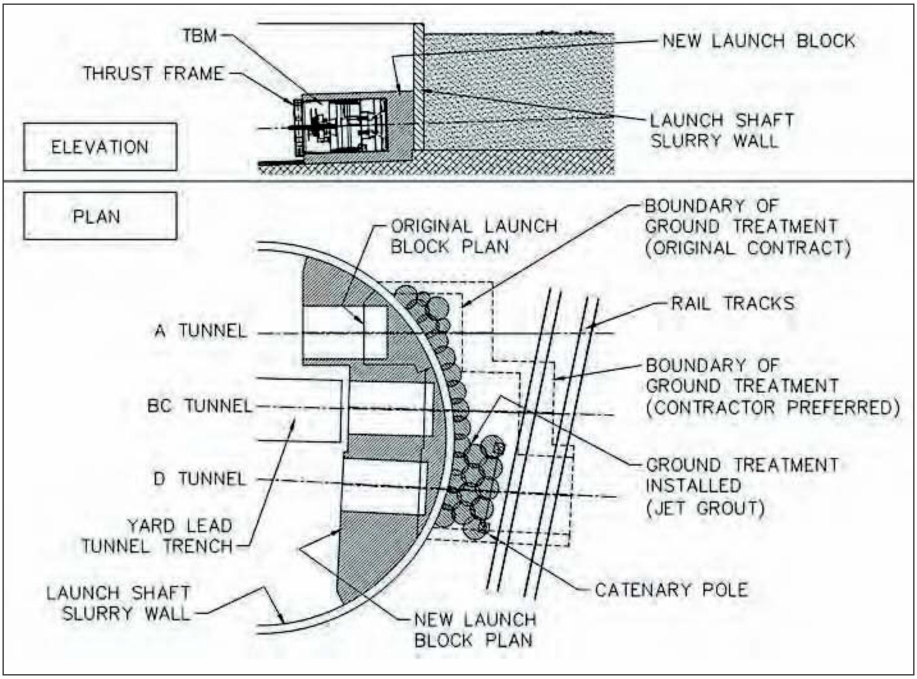

Figure 2. Launch blocks—general design configuration

Figure 3. Launch blocks—during construction

The solution to construct an external reinforced concrete structure that was structurally sound and something that could be physically seen was an obvious benefit. The launch blocks would act as buttresses to the launch wall, minimizing the potential for movement of the wall when the three tunnels mined through it, which reduced the potential for groundwater leaks at the exposed wall-rock interface. The new plan reduced the risk of surface settlement significantly and also eliminated the potential cost over runs and delays associated with the ground treatment work in the railroad territory.

Since the Launch Blocks replaced part of the scope of the follow on contract (CQ032), savings could be realized, and a portion of, these savings were used to fund the Launch Blocks together with some additional work required to install the TBM. Per the original Contract, without the Launch Blocks, there was a tight space to install the TBMs between the end of the Launch Shaft (the Q-Tip) and a rail bridge. With the Launch Blocks in place, the TBMs had to be installed on the other side of the rail bridge and jacked approximately 60m (200ft) under the bridge and into the Launch Blocks.

TBM Cradles, Rails, Channels, Jacking System, and Equipment

The TBM cradles were self-supporting and had 200 ton capacity Hilman Rollers fitted to the bottom. The cradles also had guides and attachments fitted to them for the jacking system. Rails were required within the Launch Blocks to slide the TBM in, and to center the TBM within the Launch Block cavity. In the original scheme, cradles were to be fixed, and anchored in to the concrete invert of the Launch Shaft. The same cradles were to be used at the Reception Pits to retrieve the TBMs at completion of mining. In the new scheme, the fixed cradles utilized at the Reception Pits, were used for the BC and D Tunnel TBM installation where there was some space between the mobile cradle and the rails inside the Launch Blocks. The guide channels were 460mm (18in) wide, and provided a flat, smooth, strong surface for the Hilman Rollers to travel on. Fitted to the channels were angles which accommodated a Jacking System. A diagram of the channel configuration is shown in Figures 4 and 5. The Jacking System was quite simply a set of brackets and hydraulic cylinders that pushed or pulled the TBM cradles. A diagram of the jacking system is shown in Figure 6.

Foundations and Rock Support for TBM Launch

The assembly of shields for A, BC and D Tunnels occurred in the same location. From the assembly location, the shields were jacked forward and sideways to align with their respective Launch Blocks. The assembly location and the area where the shields were jacked forward lay adjacent to the Yard Lead Tunnel trench. Since the TBM shields were originally going to be assembled at the launch locations, the heavy weight of the shields was not factored into the trench wall support design. To complicate matters, the invert area in the Launch Shaft, where the TBM assembly was now going to occur was not solid bedrock. The rock dipped down, and for 70% of the area, there was only soil. Mueser Rutledge of New York had designed the trench wall support in the area of interest, and had only allowed for loads of the TBM Trailing Gear, which were significantly less than what was now required for the assembled shields. Preparation of the Launch Shaft was already behind schedule and threatening to delay TBM assembly, and the originally approved plan was nearing completion of construction when it was realized that it would not be strong enough for the new TBM assembly scheme. The solution adopted was to construct an independent platform that could take all the weight of the shields, and therefore not overload the existing trench wall support.

Within a week, a design for the ‘TBM Assembly Platform’ was formulated by the Contractor and Mueser Rutledge, approved by the Owner, and construction began almost immediately. See Figure 7. Resources responsible for tying rebar and pouring the concrete invert in the area were utilized to install additional rebar and concrete to form the TBM Assembly Platform. The Contractor’s Superintendents creatively came up with a plan to install and anchor the mini-piles after the concrete was poured. Subcontractor Nicholson was finishing up mini-piles for other portions of the trench wall, and were able to move directly over to install the piles for the Launch Platform, saving on mobilization costs.

In addition to the construction of the TBM Assembly Platform, Mueser Rutledge determined that additional rock bolts were required in the solid bedrock portions of the trench wall.

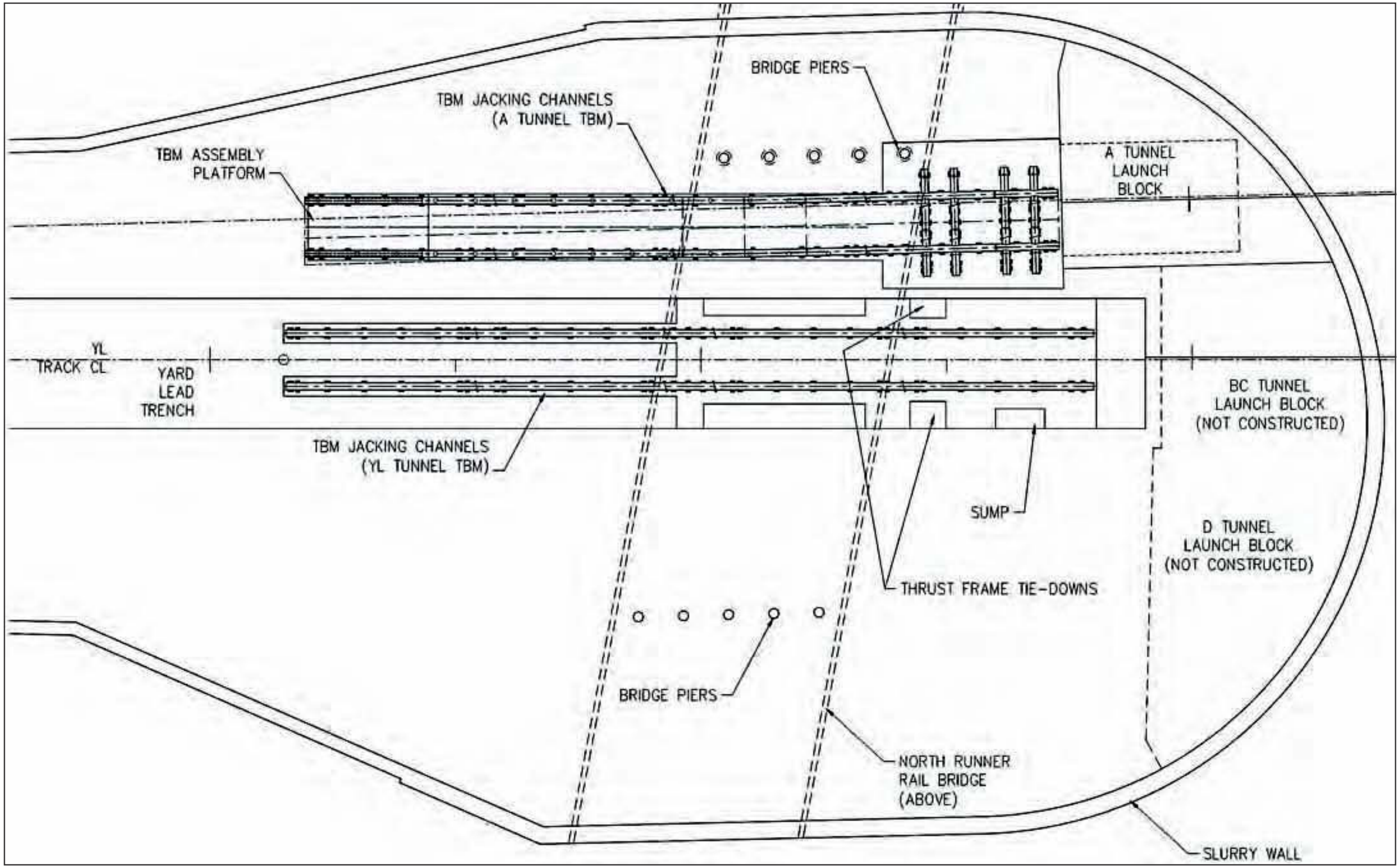

Figure 4. Channel configuration—yard lead and A Tunnel TBM assembly

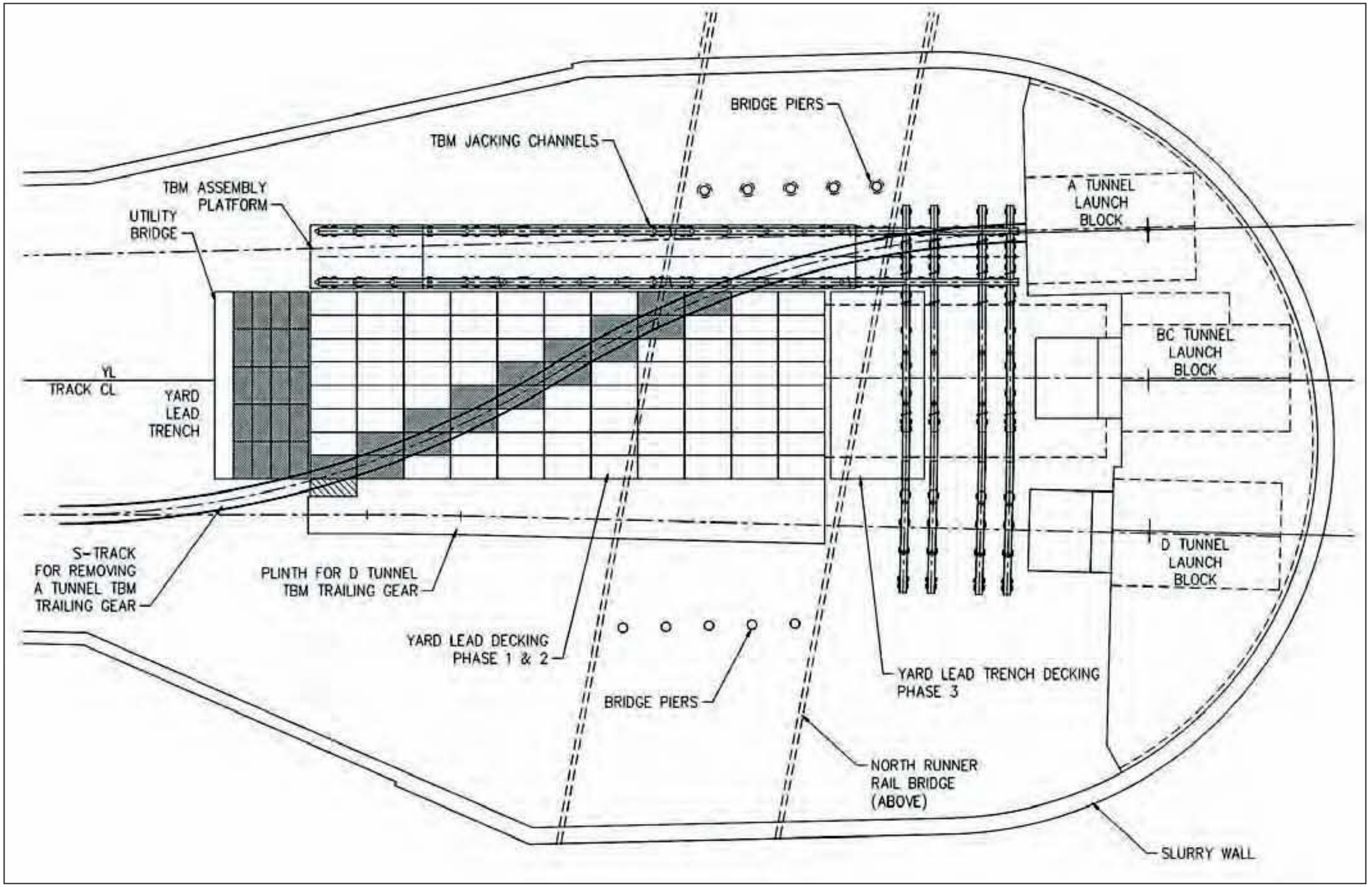

Figure 5. Channel configuration—D and BC Tunnel TBM assembly

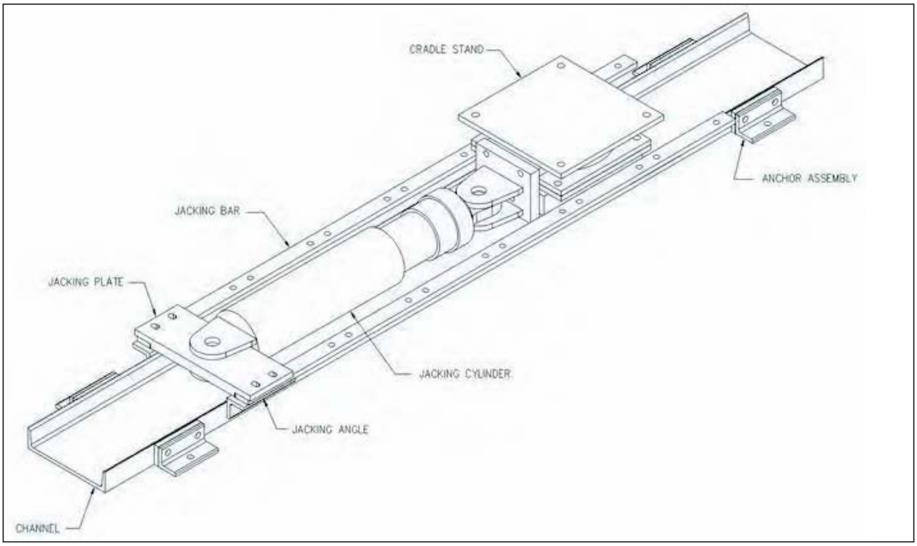

Figure 6. Jacking system

Construction

The TBM for the YL Tunnel was the first to launch, and was on the critical path. However, as that tunnel was to be launched into a full-face of competent rock, it did not require a launch block. The TBM for the A Tunnel, which required a Launch Block, was the second to launch. The schedule was very tight for the A Launch Block, whereas there was plenty of time available to construct the BC and D Launch Blocks. The Owner had provided the design drawings to satisfy the final concrete structural and durability requirements. The Contractor was required to analyze and determine the structural requirements for the TBM launch. The Contractor's Professional Engineer, Mr. Ed Heine determined the additional structural requirements (see Figures 8 and 9), which consisted of:

- A keyway in the base of the Launch Blocks at the rock/concrete interface to resist the horizontal thrust reactions from the TBM launch.
- Vertical shear keys in the adjacent walls between Launch Blocks (between A and BC, and between BC and D). This provided additional horizontal resistance to withstand horizontal thrust reactions from the TBM launch.
- Fourteen 64mm (2.5in) diameter vertical tie-rods in the Launch Blocks, each pre-stressed to 311tons, to resist the overturning moments, and to clamp the sandwich of concrete lifts together.
- Four groups of four (sixteen in all) 64mm (2.5in) diameter horizontal tie-rods to anchor the Thrust Frame loads and transmit them into the Structure.

It was originally envisaged that the portions of the A and BC Launch Blocks which required modification afterwards, would have a cavity formed and then filled with lean mix. When analyzing the TBM loads however, this was deemed not possible, and a full reinforced block was required, making the demolition work more difficult than expected. Also, when analyzing the loads on the A Launch Block, due to the fact that the BC Launch Block was not yet constructed, the A block was only able to support 77% of the TBM thrust. This did not tprove to be a problem however, as no more than 50% of the thrust was required at start up.

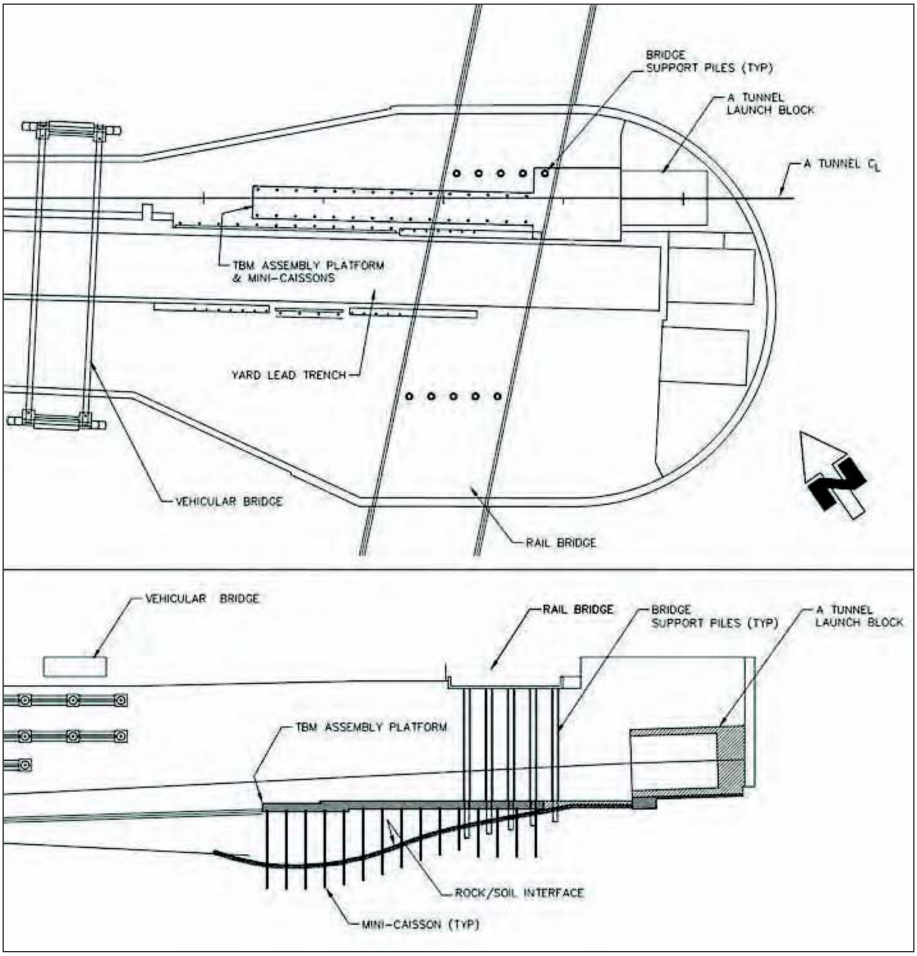

Figure 7. TBM assembly platform

A collapsible steel form was purchased from Everest Equipment Co. of Quebec to provide a circular hole within the Launch Blocks of 6,934mm ±25mm (22ft-9in ±1in). The outer diameter of the TBM shield was 6,833mm (22ft-5in) and for normal tunneling, the diameter of the excavated tunnel cut by the TBM Cutterhead was 6,858mm (22ft-6in), so there was an additional 25mm (1in) annular gap inside the Launch Block. If the annular gap was too big, grout could travel easily around the skin of the shield and lock in the TBM. If the concrete form was not set correctly, or not made correctly, and the diameter was less than 6,858mm (22ft-6in), then the TBM would not fit. The risk of grout migration around the shield skin still existed, so in order to prevent this from happening, Bullflex grout bags that were continuous around the circumference (except at the TBM support rails) were affixed on 1.5m (5ft) centers, around the hole inside the Launch Blocks. Two 6 inch PVC casings per grout bag were installed in the top of the Launch Blocks to provide external access for connecting grout hoses. See Figure 10.

Delays in preparing the Launch Shaft for the tunneling held up the start date for TBM assembly and the start date for Construction of the A Launch Block. The Launch

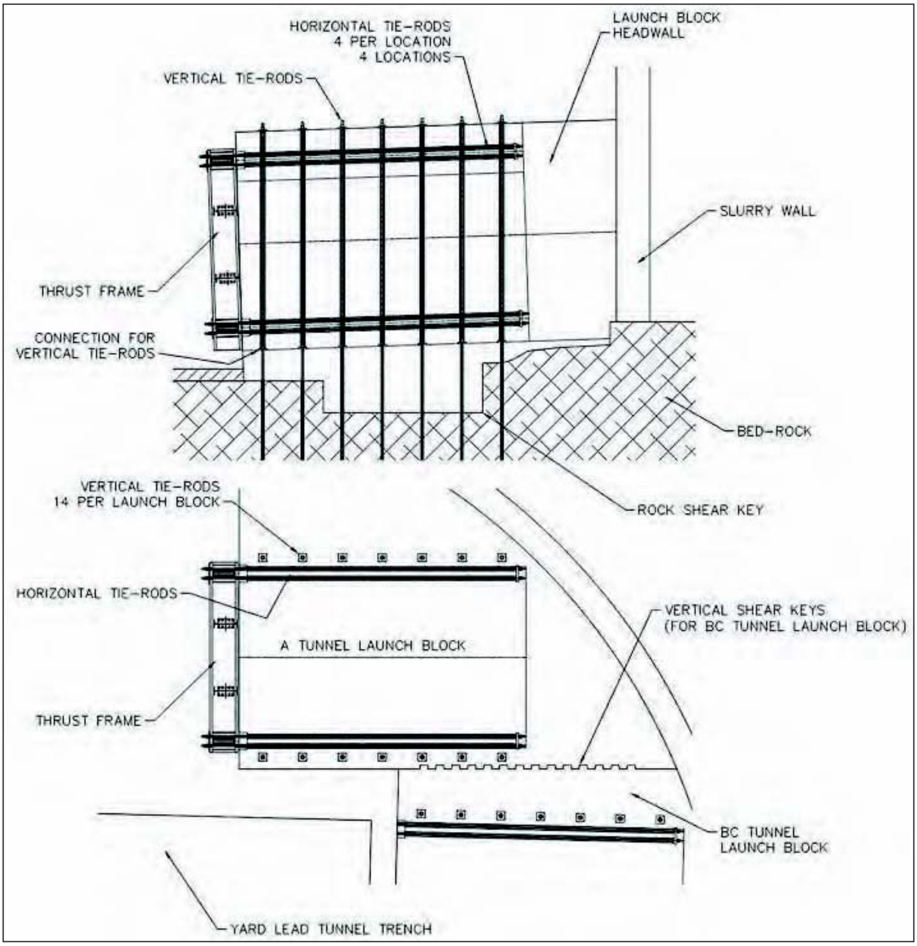

Figure 8. Construction of launch blocks—additional requirements

Figure 9. Construction of launch blocks showing vertical and horizontal tie rods

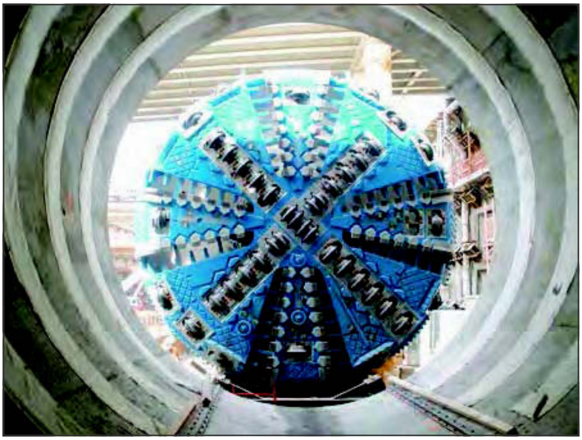

Figure 10. Bullflex bags in the launch block for the A Tunnel

Block construction never impeded the TBM assembly schedule, but it was ready just in time.

Launch Details

Although it took some extra work and time to insert the TBM shields inside the Launch Blocks, its overall effect on start-up was very beneficial. Just prior to commencing mining, the erector was within a few inches of its limits to travel back and install the first ring. Before each segment was installed, tail seal grease was smeared over the tail seal brushes and reverse spring plates to make sure they were protected from backfill grout. With the first ring installed, and with its circumferential gasket pushed hard against the seal plate, the small void behind the ring, approximately 360mm (14in) long, was sequentially grouted in 1.2m (4ft) lifts, paying attention to volume placed and accounting for any leaks around the steel seal plate. Once this first ring was grouted effectively, the bubble pressure was raised to about 50% of the theoretical value, and the TBM, with Cutterhead rotating and slurry circuit active, was pushed to the concrete face inside the Launch Blocks. At completion of the first push of 1.5m (5ft), when the TBM was still within the Slurry Wall of the Launch Shaft, and an additional 1.5m (5ft) length of backfill grout had been placed behind the tunnel liner, the bubble pressure was raised to full theoretical value. The only remaining obstacle that existed was mining through the remnants of the abandoned jet grout program which caused difficulties with the slurry properties. Treatment of the slurry with chemicals continued until the TBM was outside of the jet grout zone, and shortly after, mining progressed as if it were anywhere within the alignment.

At the rear of the Launch Blocks, during initial startup, there was a minor degree of grout leakage around the interface of the seal plate and the inner part of the Launch Block. On A and D tunnels, prior to building the first ring, a caulking compound was placed between the interface, whereas for the BC tunnel, Oakum was used instead and provided a better result than the caulking. The two component annulus grout, which gels within 20 seconds, was very beneficial in sealing up the breaches as well.

For each push while the shield was in the Launch Block, the tunnel crew would first inflate the Bullflex grout bag nearest the end of the Tail Shield. Then mining would begin and grout would be pumped through the tail Shield to the remaining void behind the tunnel liner. Engineers kept track of grout volumes for the initial 14-inch void, the Bullflex grout bag volumes, and the void behind each tunnel liner, to ensure the entire void was properly filled. Once the TBM had mined approximately 11m (35ft), the Tail Shield was buried and grout volumes corresponded to normal mining operations.

Synopsis

The Launch Block concept prevented Contract delays, and reduced risk to the tunnel construction process as well as to the adjacent railroad property. In addition to these benefits, it was paramount that the shallow tunnels started well and without a hitch. The Launch Block concept was developed jointly by the Contractor and Owner, and through cooperation, the jointly designed scheme provided better guarantees and proved to be a great start to a successful project.

YARD LEAD TUNNEL MIXED FACE ZONE

For the lower tunnel, the Yard Lead (YL), the initial 370ft portion of tunneling was in very hard Gneiss rock (up to 186MPa or 27,000psi UCS), and then transitioned to a full face of glacial till soil as was present in the upper tunnels. The surface settlement risk was not in the initial stages of mining, but in the 24m (80ft) long transition from rock to soil. The Contract specified Ground Treatment be done in this zone.

For reasons similar to the ground treatment scope outside the Launch Shaft, the Contractor proposed increasing the size of the ground treatment block specified in the Contract, so that it would act as a structural arch when the bentonite slurry became contaminated with cement. This would have almost doubled the volume of jet grouted soil.

The Decision

For the mixed face zone along the YL tunnel, there three decision paths; construct the original jet grout zone per the Contract, construct the larger jet grout zone proposed by the Contractor, or do no jet grouting at all. The task of conducting the original jet grout scope operations in the middle of the Sunnyside Yards was planned, but was going to be challenging, and schedule issues due to railroad resource requirements to support the jet grouting scheme were becoming a reality. The Owner initially suggested the elimination of the ground treatment, and after consultation with Pierre Longchamp (former Technical Director of Bouygues) and Werner Burger of Herrenknecht, it was decided that 'doing nothing' was a reasonable risk for the Owner and Contractor to take. Some things that influenced the decision were:

a. The Herrenknecht Slurry TBM, which contains a bubble chamber, could maintain an accurate face pressure through mixed face conditions. This had been demonstrated particularly well on the SMART Tunnel in Kuala Lumpur.
b. Besides the obvious challenge of mining through the mixed face zone, borings of the soil ahead indicated the presence of massive boulders up to 15ft in diameter, and potential UCS of 372MPa (54,000psi), more than double the strength of the bedrock. The potential for poor TBM performance would not just be confined to the 24m (80ft) mixed face zone, but also in the boulder field.
c. The major risk was whether the TBM Cutterhead (or more specifically, the cutters) could successfully pass through the mixed face zone and the boulder field without causing the TBM to stop. Extensive design considerations had already been implemented on the new Slurry TBM Cutterhead, so the Contractor felt reasonably confident about the TBM performance. In order to further mitigate this risk, stocks of various styles of cutters were purchased and a modified cutter inspection and performance plan was developed. Various cutter styles would be trialed in the first 100m (300ft) of the YL Tunnel to establish which performed best, and therefore, which provided the best

chance for mining through the mixed face zone and boulder field without stopping the TBM. Since the TBM was in rock for the first 100m (300ft), Cutterhead interventions could be done in free air.

Another consideration was that if an intervention was necessary in the mixed face zone, the jet grout would provide a safe haven for workers to maintain the Cutterhead in a safe manner. However, contingency plans were already in place for accessing the Cutterhead outside treated soil zones, and these could be implemented in the mixed face zone if required. Given that the Cutterhead was new, and that tools would be changed within the rock immediately before the mixed face zone, it was concluded that the risk of having to perform an intervention, repairs and tool changes in the mixed-face zone was low.

Synopsis

In all, the decision to eliminate the ground treatment at the mixed face zone was a good decision. The TBM made it through the zone successfully. The cutter experimentation, the subsequent research and cutter selections leading up this difficult stretch of ground paid off. In addition, the learning curve and typical start-up bugs for a new TBM were over by the time the difficult ground was reached. A lot was learned in the short stretch of rock tunnel. Three different styles of cutters were trialed. The cutters expected to perform the best did not. Luckily, the Contractor had covered its bases and had stocks of other style cutters, which turned out to perform much better in the hard Gneiss rock, and coped well with the extraordinarily hard boulders that were encountered. The detailed review of risks associated with this decision was vindicated, and the co-operative decision ensured that delays to schedule were minimized which enabled the Owner to utilize railroad resources for other critical elements of work by not tying them up on ground treatment work that ultimately was proven to be unnecessary.

CUTTERHEAD DESIGN AND CUTTER MANAGEMENT

Traylor's experience with using disc cutters in soft ground started 16 years ago on the San Diego South Bay Ocean Outfall (SBOO), and continued on the Los Angeles tunnel projects: the North East Interceptor Sewer (NEIS) and Metro Gold Line Eastside Extension (MGLEE). All of these tunnels were mined in Southern California in difficult ground, with variable forms of Alluvium deposits consisting of sandy abrasive soils, cobbles and boulders and mixed face conditions of rock and soil.

The geology for the Queens Bored Tunnels consisted of glacial wash soils with boulders, mixed face (rock and soil), and rock. So, at first glance, one would assume that what worked in Southern California would work in Queens, but that is where the similarities ended. In Queens, the boulders were considerably harder, up to 372MPa (54,000psi) UCS, whereas boulders in the Southern California area were closer to 100MPa (15,000psi) UCS. Also, the Gneiss rock, up to 186MPa (27,000psi), was not only harder than the 83MPa (12,000psi) Sandstone encountered on the Upper Reach of NEIS, but required considerably more energy to fracture. The Queens Glacial Till contains very little in the way of fines, making it extremely non-cohesive, and also very permeable.

The Contractor consulted with a number of experts early on in the bidding process of the project, and their feedback was that compressed air loss thru the ground could be very severe, and that compressed air Cutterhead interventions could be very risky, or perhaps not possible at all. It is important to understand that the Contractor realized this risk, and took the advice seriously. Planning and decisions made for the tunnel operations were all centered on the worry that if the TBM were to get stuck in the ground, there was little hope of rescuing it quickly if compressed air operations were not successful.

Cutterhead Design Philosophy

Traylor had experimented with all styles of cutters, and had found that mono-block style cutters performed best in fractured rock or soft ground where cobbles and boulders existed. Mono-block (MB) style cutters do not have replaceable rings, but instead, the ring and hub are one piece, making the cutter more robust and more resilient to high shock loads. The cutters last longer than typical replaceable ring cutters which often shatter or crack. On the flip side, the MB cutter is typically a 'one-use' cutter.

Longer lasting cutters, even though more expensive and not re-useable, pay for themselves when one considers the cost to change cutters in compressed air. For each compressed air intervention, there are considerable costs associated with the operation, most of which are time related costs. In New York, this is compounded by larger-than-normal crews. Time related costs associated with setup and demobilization of the intervention, for compression and decompression, and for inefficient work in the compressed air environment, are very high. For example, at 2.2bar (32psi) working pressure, it could take a 4 hour intervention shift to change a cutter, of which 1.5 hours would be spent changing the cutter, and the balance of the time would be spent compressing and decompressing. A 4-hour shift with New York crew sizes, on a critical path tunnel, could cost the project USD40,000. The cost of a MB cutter is USD10,000, whereas a Replaceable Ring (RR) cutter is about USD3,000. For this case, using some simple math, the MB cutter is worthwhile if it lasts 16% longer than the RR cutter. From Traylor's experience in California, some special MB cutters lasted more than twice as long as RR cutters, so experience proved it was worthwhile. Also, with fewer interventions there is less risk of injury, whether due to working in confined space, or working in compressed air.

For the project in Queens in particular, knowing that there could be possible complications when entering the Cutterhead in compressed air, this reinforced the notion that long lasting cutters were the key to success. It was just a matter of determining which cutter lasted the longest.

Both TBMs were configured the same. The critical path ran through the YL Tunnel, and it was the only one of the four tunnels that had a full face of rock to mine through. However just in case there were issues associated with the YL Tunnel TBM during testing, shipping, or re-assembly, the Contractor wanted the flexibility of being able to replace it with the other TBM. Also, as it turns out, the TBM assigned to the 'soft ground' tunnels, was able to deal with mixed face conditions (rock and soil). In order to accommodate mining through the Gneiss rock and other extreme geological conditions in Queens, the Cutterhead was designed like a rock TBM and had 42 cutter locations. Compare this with the Los Angeles MGLEE Cutterhead, a similar size, which had 21 locations, of which three locations in the center were left open for muck flow. The basic philosophy was to increase the capacity on each cutter by decreasing its discs from two to one, and to supply a sufficient quantity of cutters to provide adequate cutter spacing to effectively cut the rock at the face. For Queens, 17-inch single disc MB cutters were used for the outer 34 locations, using the same bearing and sealing package as the double disc cutters used for MGLEE. So, effectively, these cutters were twice as strong. In the eight center locations, 17-inch MB double cutter assemblies (quads) were used, with each disc's bearings and seals equivalent to the other single-disc cutters. At location 25, there were two cutter housings on the same circular path, to accommodate a test cutter.

Vidaplate, chromium carbide composite steel, was used for wear protection over the outer third of the Cutterhead. The inner wear protection was Hardox.

Grizzly Bars at the Cutterhead openings allowed boulders less than 300mm (12in) to enter the excavation chamber. All larger boulders would be broken down by the cutters. The crusher would be responsible for breaking down the boulders to a smaller size (approximately 130mm or 5in) so they could travel in the 305mm (12in) pipes of the slurry circuit. It was important to balance the workload on the Cutterhead and the crusher. If the Cutterhead openings were too large, the crusher would be overloaded, limiting the advance rate of the TBM, and possibly increasing the risk of damage or malfunction of the crusher. On the other hand, if Cutterhead openings were too small, the cutters would have to do more work, which could increase the frequency of cutter changes.

Operationally, the Contractor limited the advance rate of the TBM to 32mm/min (1.3in/mm) for most of the tunnel operations. This was done to protect the Cutterhead and crusher. It was better to go slow, and make it to the end, than go fast, and perhaps never make it to the end. On the YL Tunnel, there was a 370m (1200ft) stretch of geology towards the end of the drive that contained few boulders. This zone provided the opportunity to increase the advance rate to 60mm/min (2.4in/min), but the decision to speed the TBM up would be made at a later date, once the Contractor had a better feel for the performance of the cutters and other tunneling equipment.

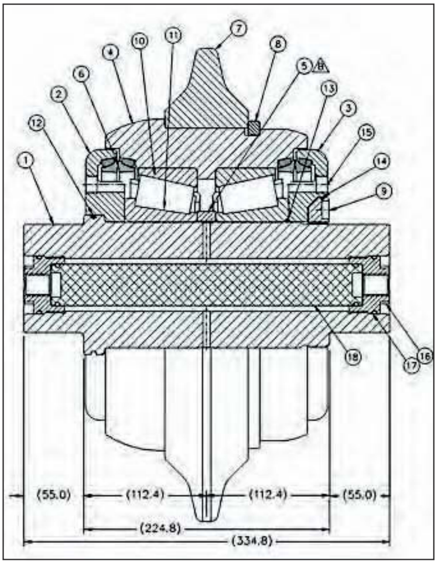

Figure 11. Disc cutter—replaceable ring type

Cutter Styles and Performance

The following cutters were trialed in the first 100m (300ft) of the YL Tunnel:

- MB-1-TCI—MB Single Disc with Tungsten Carbide Inserts and Hard facing–4340 base material
- RR-1-TS—Replaceable Ring Single Disc–Tool Steel base material
- MB-1-TS—MB Single Disc–Tool Steel base material

Later in the project, as an experiment, MB-2-TS (MB Double Disc–Tool Steel base material) cutters were used on the outer locations of the Cutterhead, but their performance was not much different that the MB-1-TS. Figures 11, 12 and 13 show the different styles of cutters.

Over a 12 year period, with the assistance of CTS from Seattle, WA, Traylor had developed and perfected the design of a mono block cutter with Tungsten Carbide inserts (MB-1-TCI), and experienced phenomenal results. The cutters have a series of chisel shaped tungsten carbide buttons pressed into the ring of the cutter, and hard facing is welded around the inserts to prevent the parent metal from eroding away. For the second reach of tunnels on MGLEE, which was approximately 1,200m (4,000ft) long, the cutters lasted the entire length through very abrasive ground. And, to add to this achievement, based on reports, the same cutters probably could have lasted the entire 3,350m (11,000ft) length of the Sound Transit U220 tunnels in Seattle. The advantage of the MB-TCI cutter, whether it has single discs or double discs, is that it does not reduce in diameter when it wears, so using it on the gauge cutter location is particularly advantageous. Also, if the ground is very soft, the carbide inserts tend to cog the cutter around, and therefore prevent flat spots. Most importantly, the cutter is extremely durable, and was very successful in breaking cobbles and boulders in the

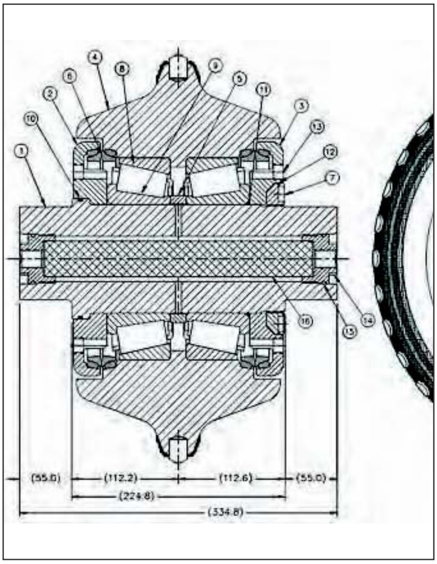

Figure 12. Mono block with tungsten carbide inserts and hard facing

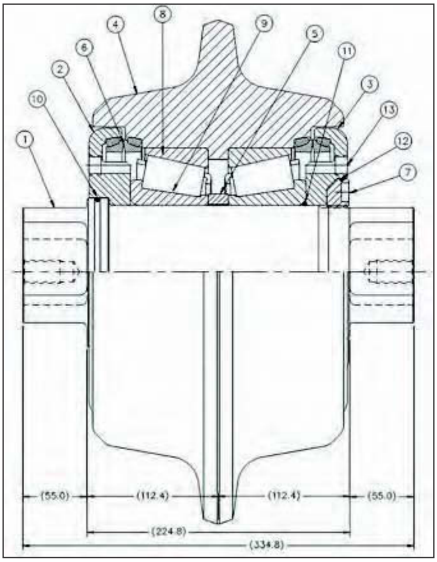

Figure 13. Mono block tool steel

Figure 14. Damaged cutter—first 20ft of yard lead tunnel

Southern California Alluvium. So, the MB-1-TCI style cutter was the first to be trialed on YL Tunnel TBM. No one really knew how long these cutters would last in the Queens geology, and there were some concerns that they may not be strong enough.

The MB-1-TCI cutters lasted 6m (20ft), and most were destroyed. See Figure 14. They had met their match, and it was obvious that this style cutter had its application limitations. The next worrisome thought came later, when there was concern voiced that the Cutterhead (with worn cutters) had not mined a hole big enough for the TBM shield to fit through. Inspection of the gauge cutters showed that they had actually fared well, as they were only trimming a small portion of rock. Away from the gauge area though, where cutters were spaced farther apart, the parent metal of the cutter discs had crushed due to the load. For the MB-TCI cutters to work in the rock, the parent material would need to be stronger than 4340, or the cutter spacing would need to be decreased. Neither was a possible solution in the short term. Inspection of the overcut confirmed that the Cutterhead had bored a large enough hole. So once cutters were changed to the new style, mining proceeded, and after successfully mining 11 meter (35 feet), the length of the shield), everyone was able to relax.

Based on the experience with the YL TBM, some but not all of the cutters on the A Tunnel TBM Cutterhead were changed prior to startup. By Contract, there should not have been any rock for the complete length of the A or BC Tunnels. However, when the Launch Shaft was excavated, the rock elevations were higher, and the A Tunnel TBM now had to mine through a mixed face at the beginning of the tunnel. It was unknown how much the rock encroached into the tunnel alignment and for how long, but it appeared to be only about 0.6m (2ft), so the Contractor changed out the MB-1-TCI in the outer 14 cutter locations to Mono Block Tool Steel (MB-1-TS) type cutters. MB-1-TCI cutters remained in the other locations as it was important to understand how well these would fare through the boulder fields ahead.

The new style cutters installed on the YL Tunnel were the standard Replaceable Ring Tool Steel cutters (RR-1-TS) which the Contractor knew would perform well in the rock, but would not be reliable in the mixed face zone. It was necessary to get a feel for the performance of this cutter and compare it against the next cutter style planned, the Mono Block Tool Steel cutters.

After mining an additional 52m (170ft) (61m into the drive), the cutters were replaced with Mono Block Single disc Tool Steel cutters (MB-1-TS). The RR-1-TS style cutters had performed well, and each cutter was measured to determine its wear. These cutters served as the benchmark. It may seem that the change to the next cutter style was a little early, but it was necessary to understand whether MB-1-TS cutters could perform as well as the RR-1-TS cutters. After mining an additional 30m (100ft) (90m into the drive), the cutters were measured and although they appeared to be wearing marginally faster than the replaceable ring cutters, they were still performing well.

There was another 21m (70ft) of rock tunnel remaining for potential free air inspections, and for changes. The Contractor did not want to cut things too close and be left without the opportunity of changing to the optimum cutters, so at the 90-meter (300-foot) mark the decision was made to stick with the MB-1-TS cutters. These cutters provided the best chance of making it through the journey ahead since they were a mono block construction and could handle the mixed face and boulders better. Periodic checks of the cutters were carried out during the next 21m (70ft), and all appeared OK, but as the TBM neared the 113-meter (370-foot) mark, which was very close to the rock/soil interface, the advance rate fell dramatically to 5mm/min (0.2in/min), and the PLC limits placed on the main bearing load would not allow additional thrust. A quick inspection of the gauge cutters revealed that wear was well within the wear limits, which meant that something else was wrong. Most likely, the shield was in some kind of bind in the rock bore and the extra high drag forces were preventing the TBM from advancing. The PLC calculation for protecting the main bearing estimates the shield drag, so if this were extremely higher than normal, the load calculation could be too conservative. Similar instances of slow TBM advance had occurred previously, and each time, the PLC calculation for the main bearing was adjusted, but there was no safety margin left. With the excavated tunnel bore being so unyielding in the rock, operators that were used to steering shields in soft ground quickly learned that gentle easy steering was required in the rock to prevent contact between the shield and the rock surface. Herrenknecht granted the Contractor permission to increase the thrust marginally for a 15m (50ft) length of tunnel. 10m (30ft) after the PLC limits were changed, the TBM hit the rock/soil interface, and the main bearing load decreased immediately.

Synopsis

The YL Tunnel TBM mined through the mixed face and the boulder field for an additional 125m (410ft) without problems. The next location for major Cutterhead overhaul was

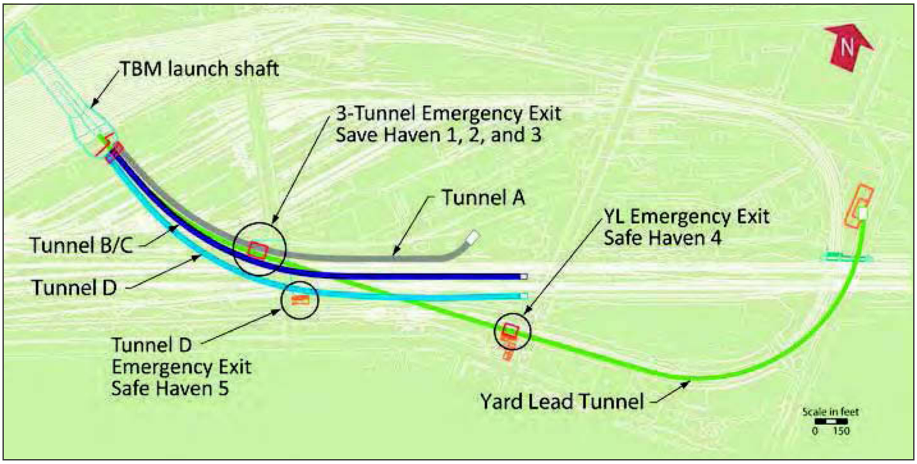

Figure 15. Safe Haven locations

planned to be within the Support of Excavation (SOE) of the Three Tunnel Emergency Exit (3TEE) Structure, where a Safe Haven would provide the opportunity to perform the Cutterhead maintenance in free air. All previous Cutterhead interventions had been conducted in free air, but things were about to change.

SAFE HAVENS

As previously mentioned, there was concern that Compressed Air Cutterhead Interventions could be problematic, or perhaps not possible, as there was shallow cover and the ground was very permeable. The Slurry TBM was able to cake the face of ground in front of the Cutterhead using the Bentonite Slurry, but advisors had warned that over time the cake deteriorates and requires regeneration. How long would the cake last? At what stage should workers stop working, pack up and retreat to safety before the cake deteriorates to the point where air loss is so high that the ground cannot be supported again? What would be the effect from vibrations of trains passing above? Traylor had extensive experience working in compressed air and, on the Los Angeles NEIS contract, had used Bentonite slurry to cake the face. More than likely, if caking was successful, workers would have approximately 24 hours before they would need to retreat and re-cake. But rules would need to be established once, or if, Compressed Air work was required, as the nature of ground dictated.

At bid time, the Contractor developed a plan to create Safe Havens—zones of treated ground that would generally allow cutter changes and Cutterhead maintenance to be performed in free air. Even though cutters had lasted 1,200m (4,000ft) in the California tunnels, it was obvious that the Queens geology was more challenging, and that cutter life would be less. If cutters could last 300m (1,000ft), there could be a way to change cutters in free air in Safe Havens strategically placed along the alignment. When studying the Contract Plans and the locations of the emergency exit structures along the alignment, it was recognized that the site for the Structures provided the best opportunity to build the Safe Havens. The concept was taken one more step. Part of the structures, or part of the ground treatment for emergency passageways, could be used to create the Safe Havens. Figure 15 shows where the emergency exit structures were positioned in respect to each tunnel.

More or less for all tunnels, the spacing between the start, the structures, and the end was roughly within a 300-meter (1,000-foot) range, with the exception of the last

670m (2,200ft) of the YL Tunnel. However, as mentioned earlier, the geology for 370m (1200ft) of the last 670m (2,200ft) of the YL Tunnel was quite favorable, so cutter wear could be minimal in the zone. Also, a full face of clay was predicted for 490m (1,600ft) along this last portion of that drive, which would provide the opportunity to enter the Cutterhead in free air, or at least provide a better guarantee for successful compressed air work.

The concept to construct Safe Havens was presented to the Owner at bid time, and the Owner expressed interest in the idea. During the process of negotiating the contract, new language and payment provisions were added to the Contract to accommodate the construction of five Safe Havens at the following locations:

- Three Tunnel Emergency Exit Site Location
  - Safe Haven 1—for Yard Lead Tunnel
  - Safe Haven 2—for A Tunnel
  - Safe Haven 3—for BC Tunnel
- Yard Lead Emergency Exit Site Location
  - Safe Haven 4—for Yard Lead Tunnel
- D Tunnel Emergency Exit Site Location
  - Safe Haven 5—for D Tunnel

The Safe Havens would provide a safe means for workers to change cutters tools. The face would be stable and work could be done in an efficient manner in free air. As a secondary safety benefit, if there were complications and compressed air work were required, the face would be stable, and air loss would be reduced.

Engineering the Safe Havens

At bid time concepts for Safe Havens 1, 2 and 4 were well developed. Problems existed with concepts for Safe Havens 3 and 5, but since these were for the second set of tunnels, it was felt that there would be adequate time to resolve the issues after finalizing the design for the more critical Safe Havens. Upon Contract Award, the Contractor and Owner met on a regular basis to refine the concepts and finalize design details. The Safe Havens would take some time to construct, so it was important that the Contractor's team building the Structures were well informed of the Safe Haven scope so plans could be integrated and coordinated.

Safe Havens 1 and 2 were the most important Safe Havens, as they would be the first locations where the Contractor could assess the condition of the Cutterhead. Lessons learned from stops at these Safe Havens would dictate how the Contractor would operate in the future. The concept presented at bid time provided the safest and most reliable means to enter the Cutterhead in free air. The Safe Havens would be located inside the secant pile SOE of the Three Tunnel Emergency Exit, and jet grout would be placed in the forward portion to ensure the ground was stabilized around the Cutterhead, so that workers could safely work for extended periods. Additional jet grout would be placed at locations where the TBM entered and exited the SOE to ensure the tunnel liner was adequately sealed for future shaft excavation. Later, after analyzing some specific site constraints, the tunnel exit jet grout had to be eliminated, and it was felt that it would be better to replace the entry jet grout with an extra row of secant piles. The shaft SOE secant piles would be socketed in to the bedrock, and a jet grout curtain would be placed around the rock interface to ensure a water tight seal. This was required for safe excavation of the shaft later. Plans for the two Safe Havens inside the SOE were more or less agreed to by the Contractor and the Owner, Mueser Rutledge had completed the design of the shaft SOE, and plans were submitted for approval. See Figure 16 for details. All that was required was access to the site, to begin preparatory work.

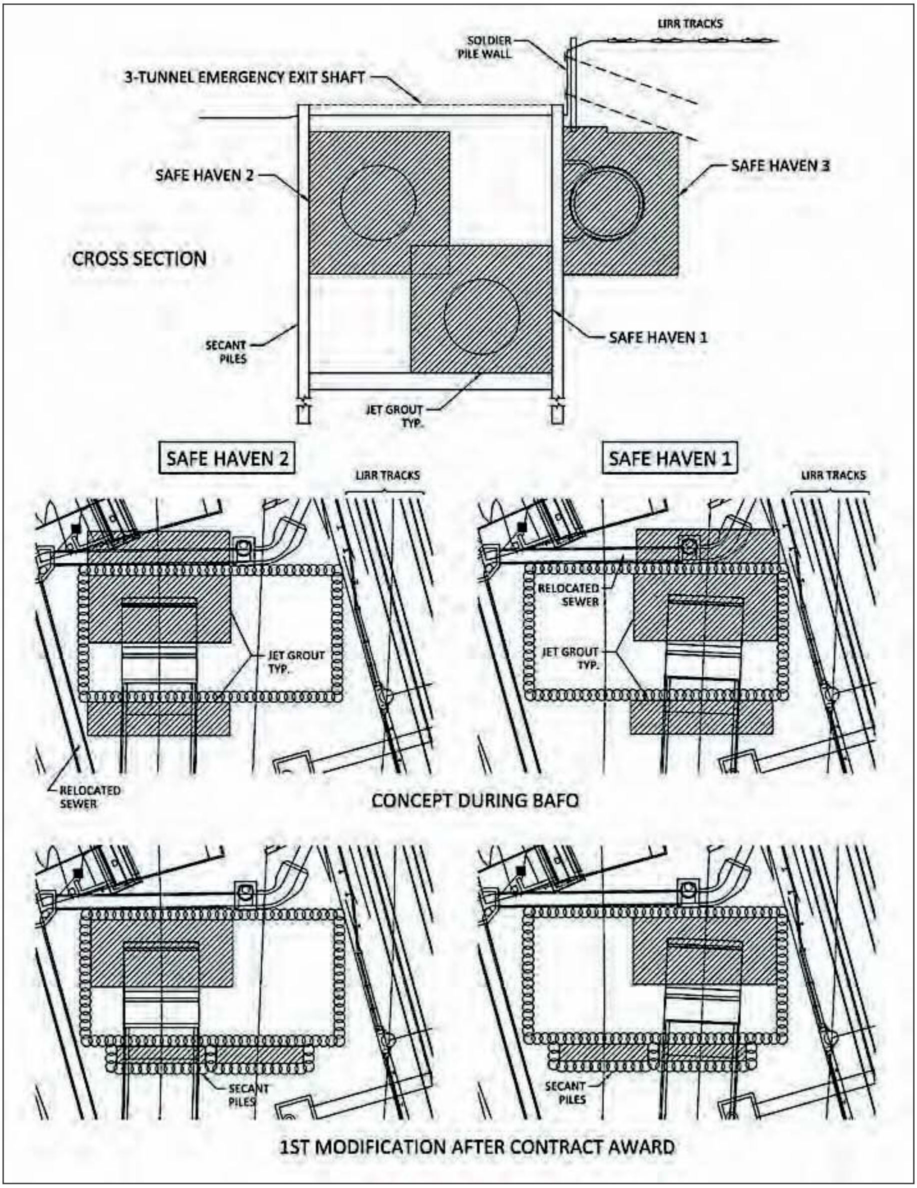

Figure 16. Safe Havens 1 and 2—designs during BAFO and after contract award

For Safe Haven 4, the biggest issue with the original concept was that a portion of the jet grout block was under active rail tracks. An angled drill pattern for jet grout could be used, or by using secant piles around the boundary, the size of jet grout block could be reduced. Since the ground reportedly contained many cobbles and boulders, there was concern that the boulders could shadow the jet grout stream, and permit portions of the ground from being properly treated. Secant piles were stronger and more water tight, as all of the ground in the path of the secant pile was removed and replaced with concrete. The smaller sized Safe Haven cost less and could be constructed outside the

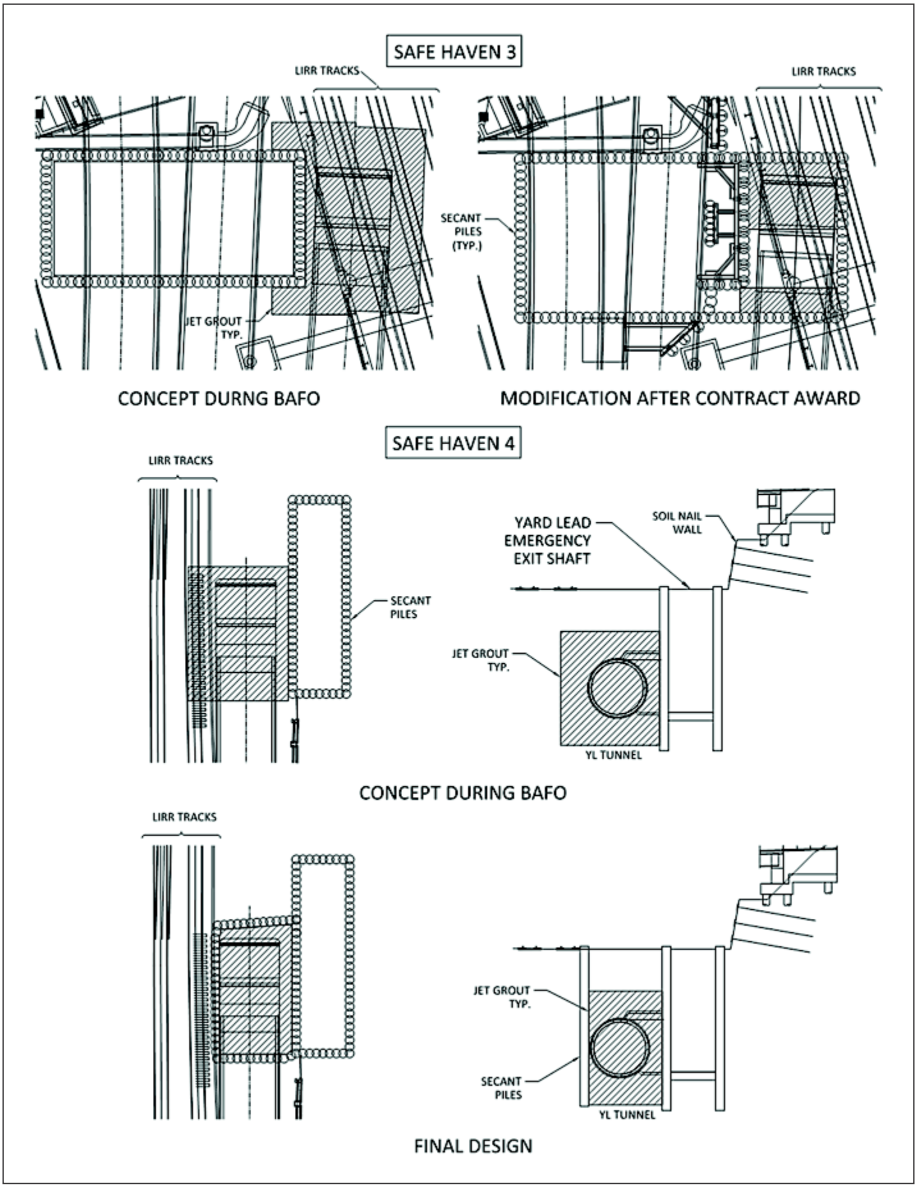

Figure 17. Safe Havens 3 and 4—design

railway tracks causing less disruption. Figure 17 shows the bid time concept and the final design. The secant piles surrounding Safe Haven 4 did not extend to bed rock, so there was some risk that it could leak in the bottom if too much 'shadowing' of the jet grout occurred. Approximately half of the Safe Haven would later be used for the construction of emergency cross passageways from the structure to the tunnel. Plans were prepared and approved, and work commenced on site.

Safe Havens 1, 2 and 4 were very similar in design. The methodology used at each would be the same, and lessons learned along the way could be applied to the future.

Attention was then focused on the remaining two Safe Havens. As mentioned earlier, both of the remaining Safe Haven concepts had challenges.

The BC Tunnel passed very close to the outside of the Three Tunnel Emergency Exit shaft SOE, so the emergency cross passageways were very short. The original location chosen for Safe Haven 3 was at the cross passage, but for the most part, it was located underneath mainline railroad tracks. An angled drill pattern would be required to form the Safe Haven jet grout block, which concerned the team. A smaller Safe Haven using secant piles would still extend into the track ROW, so was not feasible. Another realization was that the Safe Haven location was within the mandatory 24-hour, 7 day-a-week non-stop mining zone. A new location was proposed between the Amtrak rail yard, and the mainline tracks, sited before the mandatory non-stop mining zone, and away from active rail tracks. A reduced size Safe Haven design with secant piles could be used, which reduced costs, although no portion of the costs could be allotted to the cross passage ground treatment in the new location.

At the D Tunnel Emergency Exit, an active 42-inch sewer passed diagonally over the location of the cross passage, so an angled drill pattern for jet grouting was required. Secants could not be installed. A feasible alternative was to relocate the Safe Haven to a place away from the sewer, so the team analyzed the options. The D Tunnel Emergency Exit was located on the far side of the mainline tracks, at the end of the mandatory non-stop mining zone. The team felt that it would be better to locate the Safe Haven before the tracks, perhaps near the BC Tunnel Safe Haven. Operations for both could be done close by, minimizing the interruption to railroad operations in the adjacent AMTRAK yard. There had been extensive discussions between the Contractor and Owner to refine the design to optimize design and minimize risk.

Work at the Three Tunnel Emergency Exit shaft site had been delayed considerably due to access issues, and was more than likely going to delay tunnel operations. The SOE for the shaft would not be completed in time, but the Owner had a major plan brewing. The emergency exits structures were designed to be adjacent to ventilation shafts, signal rooms and traction power facilities. Following a review of the NFPA130 requirements and discussions with the FDNY and LIRR Operations and Safety departments, it was determined that the shaft spacing could be increased still be in compliance with NFPA130, and reduce the need for intermediate shafts and two structures, Three Tunnel and Tunnel D, that had been planned to incorporate safe havens were deleted. The Contractor maintained that the SOE for the Three Tunnel Shaft needed to be installed regardless, but steel beams in the secondary piles could be eliminated. The Owner and Contractor worked together to identify a modified secant pile pattern that reduced the size and optimized cost, since it was now required only for the Safe Havens. See Figure 18.

In order for installation of the Three-Tunnel safe haven piles to commence, various critical Amtrak cables crossing the site had to be relocated as part of extensive system modifications associated with the ESA Project. Delays of these relocations continued and site mobilization was delayed further. Another concern was that, even with the revised configuration, drillers were still going to have to work close to the busy mainline tracks which used overhead catenary traction power systems, and it was felt that the schedule for the work would be delayed further, as had been experienced in other locations near the railroad. TBM assembly was about to commence, and there was real concern over the impact. Something drastic was required. A series of 'brain-storming' sessions occurred where the Contractor and Owner came up with alternatives, but it was realized that even if access to the site were available immediately, tunnel operations would be delayed by many months. The thought of parking a TBM, or two TBMs, in close proximity to the mainline tracks was not deemed a realistic possibility.

The most feasible plan for the new Safe Havens was to re-align the YL Tunnel, and stack the Safe Havens on top of each other. All were in agreement with the new

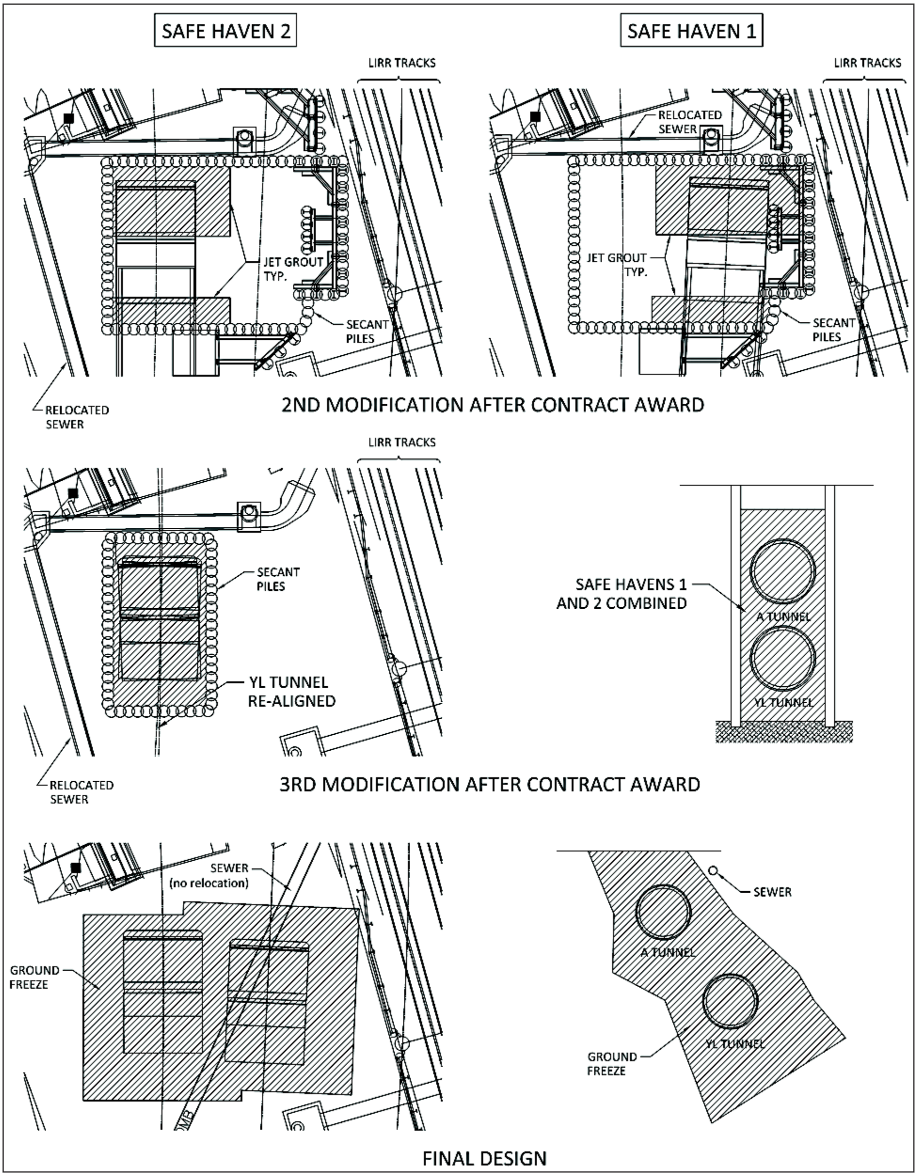

Figure 18. Safe Havens 1 and 2—new design schemes

plan, and with the YL Tunnel alignment away from the mainline tracks, the headache of working close to railroad territory was removed, and it appeared that the schedule impacts to tunneling would be minimized. See Figure 18.

There was however a catch with the new Safe Haven idea. The realigned YL Tunnel intersected a rock outcrop. So, not only would the TBM have to mine through the difficult mixed face zone, but now another one. Also, as part of the mobilization phase, a 42-inch sewer required diversion. Some of the work had started on the diversion, but issues related to asbestos had brought this work to a halt. Another solution for the Safe Haven was suggested, which did not require the sewer to be diverted, which could save

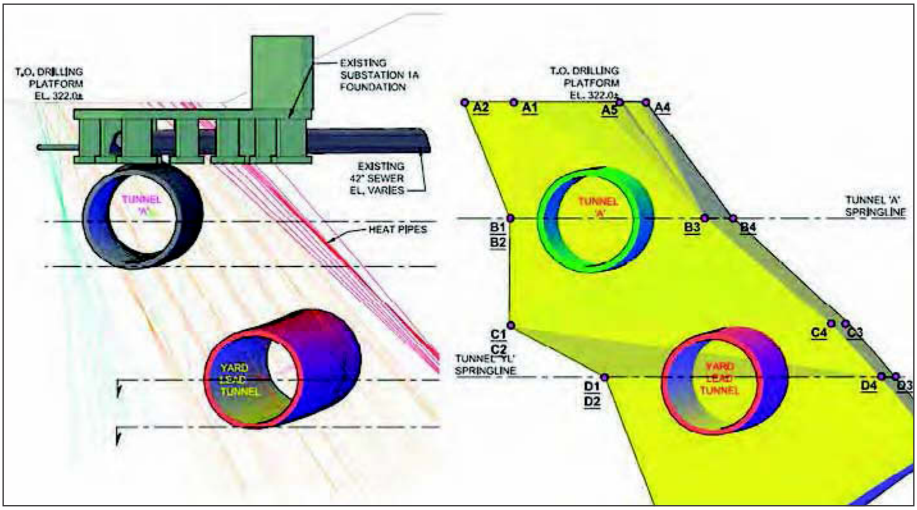

Figure 19. Safe Havens 1 and 2—freeze

considerable time and perhaps further minimize delays: freeze the ground into a single block crossing for both A and YL tunnels to form a frozen Safe Haven. It would need to extend from the groundwater surface, just above the A Tunnel TBM, to well below the bottom of the Yard Lead Tunnel, located 60-ft below the groundwater table. See Figure 18. The idea was dismissed when it was brought up in the brainstorming sessions on previous occasions. If such a block were to be made in time, there were many issues to resolve. But with limited options available, the Owner and the Contractor agreed to implement the concept, both parties knowing full well that such a thing had never before been attempted with a Slurry TBM. A lot would need to go right.

Safe Havens 1 and 2—Freeze

As can be imagined, there were plenty of issues to resolve for the Freeze Safe Haven. In terms of getting the block itself built, Moretrench of New Jersey were more than capable, and it appeared that if everything went right, the ground could be frozen sufficiently by the time the TBMs arrived and with minimal delays to TBM progress. The problematic issues were with the tunneling, and preparing plans and methods to park the TBM inside the block, preventing the TBM from freezing in the block whilst stationary, and then sealing the tunnel behind the TBM so that a free air intervention could take place. There were also time related issues to consider. The freeze, if not ready in time, would delay tunneling, and if ready too early, would need to be maintained (Figure 19).

TBM Arctic Grease Pumping and Distribution System

Moretrench were involved in a job in Boston where a steel box was jacked through frozen ground. To prevent the box from freezing to the ground, arctic grease was pumped around the skin. It seemed like a reasonable solution for the TBM, but the YL Tunnel TBM was getting close to launch, and time was running out. With Herrenknecht's help, the Contractor developed a grease injection scheme. See Figures 20 and 21. In eight locations along the shield skin, approximately 1m (3ft) apart, holes were drilled and grease ports were welded. At the front two locations (rows A and B), 24 ports would be installed per row, so that more grease could be pumped, whereas in the other locations (rows C, D, E, F, G and H), 16 ports would be installed. Rows A and B were located

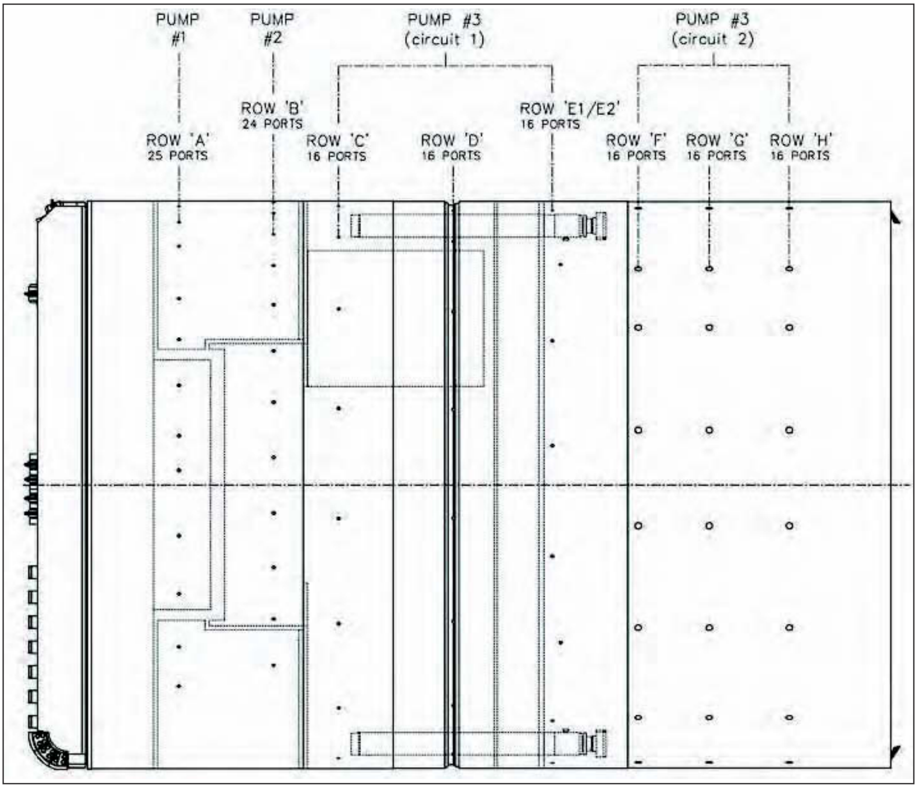

Figure 20. Arctic grease ports in shield

inside the Bubble Chamber of the shield, so were not accessible during mining operations. A 100mm (4in) hole was cored through the 50mm (2in) thick bulkhead in five locations to accommodate the installation of specially designed multi-port fittings to allow the 48 grease lines of rows A and B to pass through (See Figure 22). Rows C and D were very difficult to install as they were located between the articulation and thrust cylinders. Rows F, G and H were located in the ring build area, and needed to be flush with the tail skin. These would have to be plumbed up when needed, and removed for building the tunnel liner (See Figure 23). In terms of schedule for the YL Tunnel TBM, it took about four weeks to drill and weld all the ports, install the special bulkhead fittings, and install the plumbing for rows A and B inside the Bubble Chamber. Other TBM assembly and testing work was occurring concurrently, so the schedule was impacted by about two weeks. It was decided that all other work, which included plumbing the remaining ports and installing the grease pumps and distribution blocks, could occur later. For the A Tunnel TBM, installation of the grease ports had a bigger impact. Complete assembly of the TBM Shield and connection of the Trailing Gear could not occur until the TBM shield was jacked into the Launch Blocks. So, assembly of the A TBM was delayed about 4 weeks waiting for all the ports to be installed.

Tunnel Backfill Grout

The tunnel backfill grout was a two component system, which mixed sodium silicate and a cement based liquid at the injection point. This grout had never been used in a frozen environment, so tests were conducted to understand how the grout would behave. Test

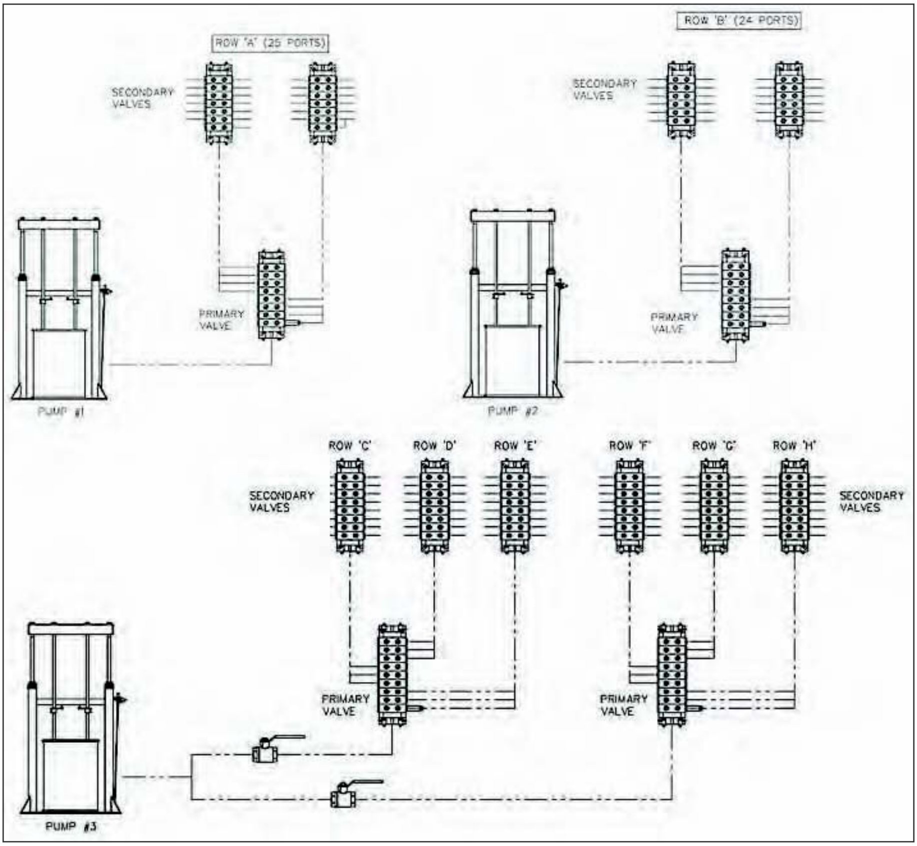

Figure 21. TBM arctic grease pumping and distribution system

samples were made by mixing the two components and allowing the grout to cure over time in the freezer of the construction office kitchen refrigerator. The temperature of the freezer was very close to the ground freeze temperature. When the grout was thawed out, the results were disappointing. The grout would turn to a thick slushy consistency, and it appeared that the two components, even though they had initially gelled, had not done much else after being subjected to the cold environment. Additional tests were conducted, using more cement and more sodium silicate, but there were similar unsatisfactory results. The two-component grout would not work. Most likely, if it were used, once the ground thawed out the backfill grout would turn to slush and not adequately support the tunnel liner. In discussion with other tunnel contractors who had mined through partially frozen ground successfully, the Contractor learned that a simple sand and cement mix would work, but this type of grout was not compatible with the tube pumps on the TBM, and concrete pumps would need to be acquired. Testing of a neat cement grout yielded promising results. It appeared that the heat of hydration of the neat cement grout was sufficient to cure the grout. Pumps on the TBM were also compatible. Additional tests were conducted to confirm results, and another problem had been solved.

Hayward Baker would be responsible for setting up and operating a grout batching plant. Two systems for transporting the grout to the heading were planned, just in case issues occurred. A 2-inch concrete slick-line was purchased, so that grout could

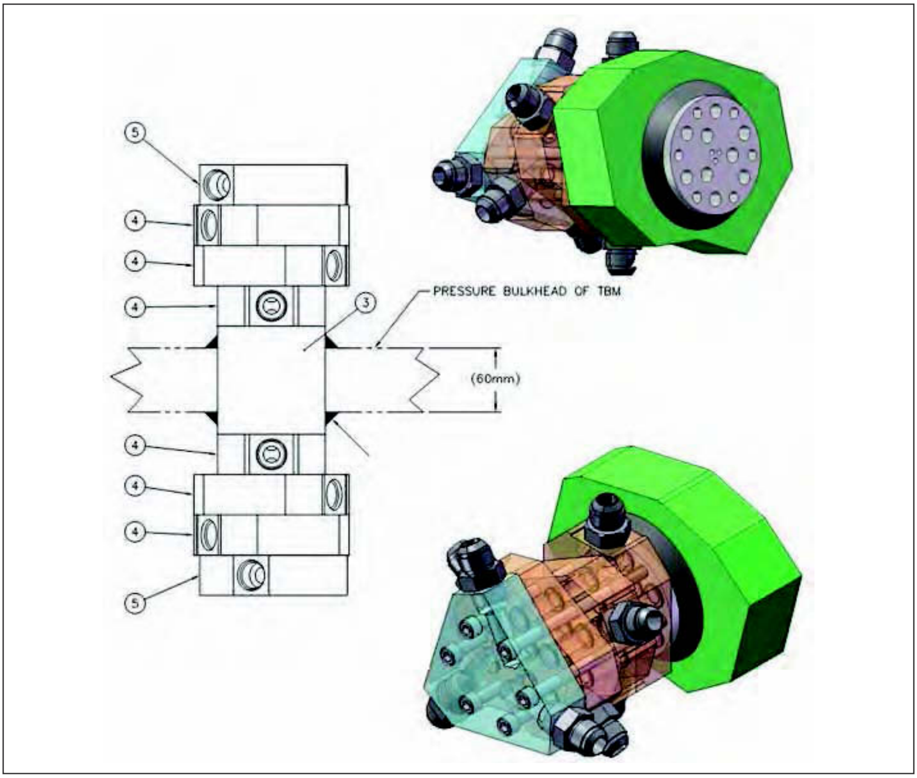

Figure 22. Special bulkhead fittings

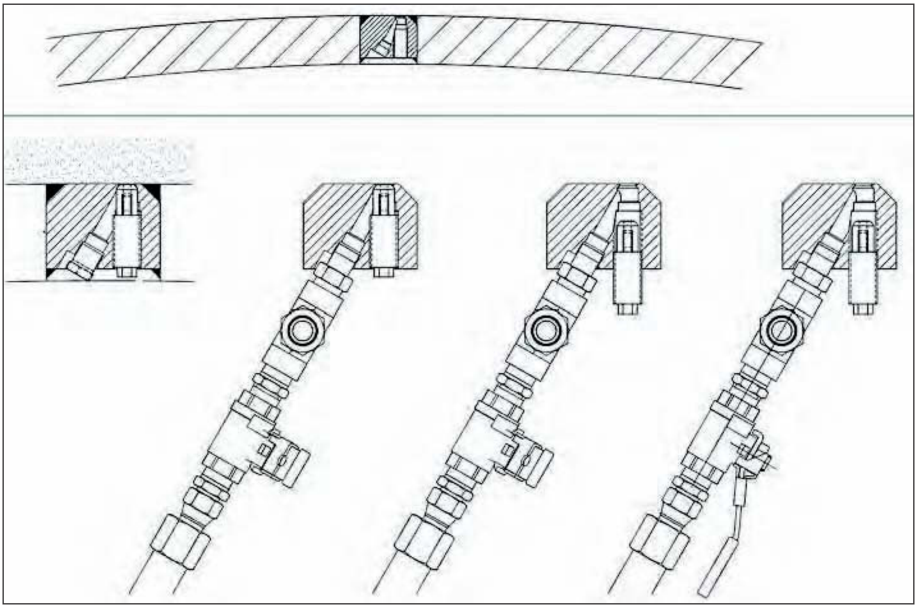

Figure 23. Special tail shield fittings

be pumped to the heading and, in addition, two grout cars, used on a previous contract, were refurbished and delivered to the job site. These would be able to transport the grout to the heading by rail. Further tests using admixtures were conducted to extend the life of the grout.

Plans for special tunnel segments were issued to the manufacturer, Technopref in Pennsylvania. Segments were fitted with 100mm (4in) threaded sockets, so that grout could be pumped at 16 locations around the ring. These could be used, if necessary, for remedial grouting as well. Pumping through the tail shield pipes would not be possible, as there was too much chance for blockage.

Sealing the Rear of the TBM

Concern still remained about whether the rear of the TBM could be adequately sealed with the neat cement grout. The worry was that the heat of hydration could melt the ground and create a path for water to travel through. Any flow of water would further erode the frozen ground and the seal would be lost. There was also concern that the neat cement grout would not completely fill the annular void as there was limited access to all the ports in the special concrete tunnel rings. On the Los Angeles NEIS project, in the Upper Reach, Traylor had fabricated and constructed steel ring beams in the tunnel, and sandwiched them between two tunnel liner rings, to house a Bullflex grout bag. The idea was very successful in sealing the annulus, and it seemed worth trying for the freeze. By pumping neat cement into the grout bag, there could be a better assurance that the grout would fill the annular void effectively, as it would be confined by the bag around the annular void. There still remained some doubt about whether the heat of hydration could cause problems. Since the Ring Beam would become a permanent fixture, it would need to be made from Stainless Steel. With only about three weeks left until the TBM would arrive at the freeze, the Contractor proposed ring beams for both tunnels and, with the Owner's approval, they were fabricated and delivered to site within two weeks, just before they were needed.

Tail Seal Grease

Samples of Condat WR89 tail seal grease were placed in the kitchen freezer, and another suspected issue was confirmed. The tail seal grease became very hard and stiff, and there was little hope for being able to pump it. Without tail seal grease, slurry, grout and even ground could leak into the ring build area. An alternative was required. Condat had no ready-made solutions, so the Contractor blended up some mixtures of Condat and Arctic grease. A combination of 4 parts Arctic grease to 1 part Condat grease appeared to have the same consistency at the freeze temperature as Condat WR89 had at room temperature. The Contractor considered getting the grease pre-blended, but there were no Vendors that could be found to help out, and time was running out. Instead, a pipe with criss-crossed bolts inside, was made to form a crude mixing nozzle.

Operational Considerations

An obvious concern mining a Slurry TBM into a freeze block was that the slurry could freeze. Thoughts of adding salt to the slurry were quickly dismissed when it was learned that this would coagulate the slurry, causing other issues. The only solution was to keep the circuit operational, not put the circuit into bypass mode, and keep slurry entering and exiting the excavation chamber continuously.

There was also the risk of the Cutterhead freezing in place if left stationary for too long. How long it would take for systems to actually freeze was unknown, but whilst exiting the frozen shaft on South Bay Ocean Outfall in San Diego, the Cutterhead got frozen in place after the TBM was stationary for too long. Workers had to enter the

Cutterhead to chip away the frozen ground to free it up. Again, the only solution was to rotate the Cutterhead on a regular basis, every 15 minutes for a couple of minutes.

Together, the Contractor and Owner worked on sequences and plans for mining into and out of the freeze. Grouting would need to switch from two-component type to neat cement at some point. Tail Seal grease would need to be changed, and special tunnel liners would need to be installed in the correct location. Also, there would need to be a point at which the overcut annulus around the shield, normally filled with slurry, would need to be primed with Arctic grease. All these steps would require good coordination.

Mining Into the Freeze

At the shaft site, Moretrench took approximately 9 weeks to drill the angled freeze holes and about 3 more weeks to install the plumbing and equipment for freezing. It then took another 15 weeks to reach –10° C (14° F), the desired temperature which designers had agreed to. Mining of the first section of the YL Tunnel was not the smoothest operation, but the TBM reached the shaft site three weeks before the freeze was ready. The TBM had stopped approximately 30m (100ft) before the shaft, as the Contractor wanted to have the opportunity to jump the TBM forward a few times, in case ground conditions in front of the TBM deteriorated over the waiting period. While the YL Tunnel TBM waited, the decision was made to enter the Cutterhead and change out the two gauge cutters to ensure there was a sufficient gap between the shield and the frozen ground. Paramedic staff from Life Support Technologies, of New York, assisted the Contractor, and the first hyperbaric operations began. Two days were spent changing cutters, and once complete, the rest of the wait time was spent completing the plumbing and setup of equipment for the Arctic grease system.

The A TBM began mining just as the YL Tunnel TBM stopped to wait for the freeze.

When the final design freeze temperature was reached, Moretrench circulated warm brine through the pipes that intersected the YL Tunnel alignment, and the pipes were pulled until the tips were 1.5m (5ft) above the crown of the TBM. The pipes were then reconnected to the freeze brine manifold. Pipes beyond the intended parking location of the TBM within the freeze were left in place, and would be pulled later, once the TBM started mining again. While Moretrench were pulling pipes, the Contractor began mining. It took a few days to mine within 3m (10ft) of the freeze and the rest of the week was spent testing the Arctic Grease distribution system, connecting up to the F, G and H port in the ring build area, and filling the overcut void around the shield with Arctic grease.

The following week, a program was followed that was jointly developed by the Contractor and Owner. On Monday, the three shifts prepared for cement grout operations, making sure the grout car could travel to the desired location, and that the pump connections and flow meters were all working properly. Trial tests were conducted with water and then grout to be sure the team was prepared. While the TBM mined forward into the freeze, over the next four days, the program was followed ring by ring, and approximately two 55-gallon drums of Arctic grease were pumped around the shield continuously for each 1.5m (5ft) push. The grease distribution system worked well, without hindering progress of the mining, although many of the forward A and B ports had become blocked, so grease injection was pumped only though the C, D and E ports. Special rings were built when the tail shield was within 1.5m (5ft) of the freeze, and two rings later, when it was time to start using the cement grout, the tail shield grout nozzles were thoroughly flushed and primed with Arctic grease to preserve them. The blended Condat/Arctic grease was also pumped, although this system had issues. If the Condat WR89 were to freeze inside the injection lines, then there would be no way to seal the tail shield, so to play it safe, pure Arctic grease was pumped to the tail seal brushes instead. The thought was that if this did not seal effectively, then more attention would be paid to solving the blending nozzle issues. There were no more issues however. Pumping of the neat cement grout slowed down the mining, as the grout was setting too quickly, and it was not possible to transport the complete quantity of grout for one push in one go (approximately 6 cubic yards). Instead, a number of batches (between 2 to 3 cubic yards) were transported to the heading, and the TBM had to stop advancing when the car required re-filling. Nonetheless, even when grout set up in one of the grout cars, the other car took its place, and the crews kept mining, matching the program ring for ring and the weeks activities remained on schedule.

The ring beam and Bullflex bag were installed on the back shifts Thursday night, and on Friday the TBM was pushed to the final parking location and the ring beam was grouted up. There was quite a bit of leakage through the mating joints of the ring beam once it initially exited the Tail Shield, but crews pumped the grout bag up with grout, and slowly but surely, the leakage stopped and the task was complete. As a final measure, the slurry was evacuated from the Cutterhead, and five totes of antifreeze were injected into the excavation chamber to mix with any slurry remnants. The TBM was left to sit for the three day long Labor Day weekend. It was felt that the three days of sitting would help the ground re-freeze where grouting had occurred behind the TBM.

Entering the Cutterhead

On the morning after the Labor Day Weekend, a small meeting took place with the Superintendents, Crew Bosses and Shift Engineer to make sure they all understood the possible risks moving forward. Compressed air loss over the weekend had been very low, which indicated that the seal behind the TBM was fairly good. Upon entering the Cutterhead, if any ground water inflow was encountered, workers were to stop any preparations for Cutterhead work, secure the Cutterhead entry doors and retreat to safety.

The air pressure was lowered, workers entered, and it was assessed that conditions were favorable. Safety Work platforms were installed in the Cutterhead, and crews were getting ready to start changing cutters, when a low flow of water was observed and, over a short period of time, the flow increased to the point where it appeared dangerous. Work stopped and the Cutterhead entry doors were secured. The work platforms were left in place inside the Cutterhead. Re-assessment of the situation was made, but conflicting indications made it difficult to understand the puzzling conditions. When valves next to entry door were opened, water was present, indicating that the entire excavation chamber had filled with water. It was not safe to open the entry door. The bubble chamber was fairly empty however, with little water in the invert, and no noticeable sign of filling. The puzzlement was that the bubble chamber and excavation chamber were normally connected at the bottom, but it was now blocked. Quickly, it was realized that the increasing pressure building up within the excavation chamber was creating a potential unsafe condition. The air lock door was closed, and air pressure was added to the bubble chamber to balance the ground pressure. Things had not gone well.

Scary thoughts were brewing. With the TBM now surrounded with flooded ground water, how long would it take for the water to freeze, and in turn lock the TBM shield in the ground? How long would it take to thaw the ground enough to free up the TBM? Getting the TBM stuck was always a big concern, and now there was a real possibility it could happen. The sadder predicament was that with time obviously being of the essence, the TBM could not even mine forward to escape the location, since attempts made to free the blockage by the Cutterhead, crusher or slurry circuit were unsuccessful. The Cutterhead had not been rotated as this would destroy the work platforms, possibly exacerbating the blockage and possibly causing other issues with the crusher and grill plate near the slurry suction inlet. An emergency meeting was held with the Owner, and various options were offered. Perhaps the leak was coming along the gap between the tunnel liner and the frozen ground, and the annulus tunnel liner could be re-grouted? But that would take time. Perhaps leaving the Cutterhead full of water, without flow, would allow the flow channel to freeze tight? But what other systems might also freeze? Perhaps, pumps could be installed into the Cutterhead to dewater and maintain a safe level so that the blockage could be freed up? This seemed possible, but this could cause more ground loss. Perhaps pumping some more Arctic grease around the shield could help? Perhaps working in Compressed Air would be an effective way to free the blockage?

Clearing the Blockage

It was agreed that more information was needed, so once the emergency meeting was over, the Contractor's crews forged on. Sufficient ports were opened to lower and maintain the water level in both chambers below spring-line. Bubble pressure was removed, and with the water level maintained at spring-line, bucket tests revealed that the water inflow was 570L/min (150gpm) in the excavation chamber and 100L/min (25gpm) in the Bubble Chamber. The flows appeared to be manageable. The Cutterhead door was opened, a pump was installed in the Excavation Chamber, and the water level began to fall. While progress was being made lowering the water level, an inspection was made of the surroundings. Approximately 0.6m (2ft) of ground around the forward upper part of the shield in front of the Cutterhead had eroded away. The eroded ground was most likely what had caused the blockage in the Cutterhead below. The noise of the leakage could be heard above the shield, as it carried cobbles and boulders with it, and a quick look back seemed to indicate that the leakage was possibly coming from above, through the frozen block, but it was too difficult to say with certainty. Temperature probe readings from Moretrench confirmed that the frozen mass of soil was still within acceptable temperature limits, and other investigations had proven there were small channels around the Tail Shield in places, which indicated there were leakage paths coming from behind the TBM. Needless to say, the origin of the leakage was unclear, and remained so, but as a result of the violent inrush of water, a large cavern had formed above the front portion of the shield and some eroded ground had piled on top of the shield like a small pyramid.

Eager to get a better understanding of the blockage below, and hopeful that the work platforms could be salvaged, an additional pump was called for, so that the water level could be pulled down quicker. At shift change, crews worked on installing the second pump. A communication breakdown occurred shortly after, and instead of hooking up the new pump, the crew decided to take the pump from within the Excavation Chamber and place it into the Bubble Chamber. The new crew did not understand that little, if any, flow was coming from the Bubble Chamber, and did not understand that the blockage had isolated the two chambers. During previous Cutterhead interventions, it was much easier to pump the water level down in the Bubble Chamber, as existing fixed work platforms inside that chamber made it easier to work. Word spread out of the tunnel quickly that the water inflow in to the Excavation Chamber had increased, and due to safety concerns, the Owner directed the Contractor to evacuate all workers from the Cutterhead, close entry doors and re-establish the Bubble Pressure. In turn, the void, or cavern as it was commonly referred to, which surrounded the front portion of the TBM shield, was allowed to fill with near freezing water.

With the Cutterhead once more isolated, there was a growing fear that the water and ground blocking up the chambers would begin to freeze, and with no decision on a path forward, there was no choice but to rotate the Cutterhead on a regular basis, to prevent it from locking up.

It would take 7 days for crews to clear the blockage, working 4-hour shifts, 24 hours a day, 7 days a week, in 2.3bar (34psi) of compressed air. Initially, there were grave concerns that compressed air operations could not work, as air loss was estimated to be 113m^3/min (4000cfm), which was very high, and near the capacity of the compressor station, tunnel pipes, and valves that were in place on the TBM. Additional compressors were added, and thick bentonite slurry was injected around the shield. In an average 4-hour shift, crews would work for 120 minutes filling approximately 100 bags, and spend 70 minutes decompressing with the aid of oxygen. Since union rules were strict about working no more than 4 hours, 15 minutes of contingency time was scheduled in, so that there would be no over-runs.

Once the TBM Slurry Circuit was operational again, some decisions had to be made. It was apparent that sealing the leak in the frozen ground would not be possible, as the location of the leak had not been pinpointed, and keeping the TBM in the frozen ground long enough to find out and implement a solution would risk it freezing in place. But a decision had to be made whether to change the cutters under compressed air while still in the frozen ground, or to mine out with the existing cutters. There would be no stopping to change cutters as the TBM was about to start mining under the LIRR tracks, which was a mandatory 24hr, 7 day a week mining zone. Although there were many who wanted to move the YL Tunnel TBM out of the freeze as soon as possible, an equal number wanted to change cutters like planned, as they were in bad shape.

Another consideration was that the A Tunnel TBM was very close to the Safe Haven, but it could not mine into the Safe Haven, as it was thought that the cavern which had formed around the Yard Lead Tunnel TBM could have extended up to A Tunnel. The team was divided about whether to try parking the TBM in the freeze again, or try their luck doing compressed air work before the Safe Haven, knowing full well that this might not be possible.

The wear of the disc cutters on the Yard Lead TBM was not excessive, but the TBM would have to mine under mainline tracks, a vehicular bridge, a 42-inch sewer and a main signal tower before it could reach a suitable spot to attempt a compressed air intervention. It was decided that the solution that presented the best potential outcome for the job, was to change cutters on the YL Tunnel TBM in the freeze, then mine the TBM out and grout the eroded cavern. Meanwhile, the A Tunnel TBM would stop 30m (100ft) or so before the frozen Safe Haven, attempt to change cutters in compressed air, and then mine without stopping, through the Safe Haven. There would be only 5m (17ft) of cover above the crown of the TBM, comprising silty sand, well graded sand and fill. The potential for excessive air loss was high. As a backup plan, if compressed air work was not possible, the Safe Haven would be used for the cutter change.

As it turns out, all went according to plan. It took about two weeks for crews to change cutters on the YL Tunnel TBM. As the TBM exited the Safe Haven, compressed air operations began on the A Tunnel TBM, and the Contractor and Owner coordinated to manage the process of systematically filling the cavern as the YL Tunnel TBM exited the Safe Haven. There were some nervous moments for the A Tunnel TBM when the face began to collapse hours into the first intervention, and the TBM had to mine another 1.5m (5ft), so an intervention could be tried again. Air leakage from compressed air operations also forced ground water into the sump of a Railroad MCC building, flooding the lower portion of the basement. Fortunately this possibility had been foreseen, and regular MCC inspections allowed the situation to be rapidly identified and resolved. Once it was determined that the sump pump was bad, the problem went away. After approximately 16 hours of work, air loss was too high and the face required re-caking for 8 hours. For both TBMs, a program was developed for mining through the freeze, which notified the crews when to change grout types, tail seal grease types, ring types, and when to pump Arctic grease around the shield.

Safe Haven 4—Yard Lead Emergency Exit

It was only another month until the YL Tunnel TBM was ready to enter the next Safe Haven at the Yard Lead Emergency Exit. This Safe Haven was a jet grout block surrounded with secant piles. Groundwater level was approximately 11m (35ft) above the invert. The TBM mined and parked inside the Safe Haven without incident, and the rear of the TBM was grouted up tightly with the two component grout. No Ring Beam was installed. Muck was evacuated from the Excavation Chamber and the TBM was allowed to sit for the weekend with compressed air inside the Cutterhead. On Monday, the air pressure was stepped down slowly, and there was no sign of any leakage. Workers entered the Cutterhead and began changing cutters. All was going to plan, but things went wrong.

About a day after work began, there was a blow out in the Cutterhead, and workers evacuated without any injury. The Cutterhead doors were sealed up, and the bubble pressure was restored. Sadly though, work decks and a pneumatic spader were left behind. And, during the night, a sink hole developed in front of where the TBM was parked. What had happened?

There were many theories, but simply, it was discovered that the TBM had been parked in the wrong location. It was 1.5m (5ft) past the intended stopping location, which meant there was only about 50mm (2in) of the secant pile wall left, when it blew. Later, it was discovered that the bubble pressure was not effectively supporting the face, as muck had burst in blocking the opening to the Excavation Chamber. Without sufficient support, the void developed by the blow in, eroded to the surface and formed the sink hole. So, the team was back to familiar territory.

It took approximately three weeks for crews to finally get the TBM operational again. Ground cover was approximately 7.6m (25ft), composed of silty gravels, with some sand and fill material. Air loss through the ground was so high it took over a week to provide conditions safe enough for workers to enter the Cutterhead. The sink hole had been filled from the surface with lean mix, but air was leaking around this plug. A pit was dug in front of the TBM, in the zone where most of the bubbles appeared through the saturated ground, but this did not do much to slow the air loss down either. Workers could not enter the Cutterhead even when using the thickest of bentonite slurry. The team was running out of ideas, so plans were drawn up to install more secants in front of the TBM. As almost a last ditch effort, sawdust was added to the Bentonite slurry to thicken it up to see if it would cake better. This provided a good enough cake for workers to spend 2 hours doing a hi-tech emergency patch job on the face. Workers were provided with straw, cement bags and extra sawdust to do a Sand-Hog style stucco job on the area of the face where the blow out occurred. Initially, the crew would let all the materials in combination suck up into the void along with the air flow, and slowly but surely, with the combination of cement, cement bag paper and straw, they were able to make a patch which worked. Upon their exit, the face was re-caked with the thick sawdust slurry, and then subsequent crews added to the patchwork and enhanced the seal. With this in place, there was hardly any air loss, as the rest of the ground surrounding the TBM was jet grout and secant piles. And so, as could be expected, there were heated discussions about whether to finish what was started and change cutters, or mine ahead and choose another location.

With almost no air loss and with stable face conditions, it was a difficult decision, but the Contractor changed the cutters. Motivated to get mining again, it only took the crews 5 working days to change all the cutters in 1.2bar (18psi) working pressure.

No More Safe Havens

With all the drama associated with trying to make the Safe Havens work, the Owner decided to eliminate the remaining two. The Safe Haven concept was a good one, but for various reasons, plans had not achieved the intended results. If the Safe Havens had all been constructed the same way and in a similar style as the one located at Yard lead Emergency Exit, then the story may have been different. Most would agree that if it were not for over mining the Safe Haven at Yard Lead Emergency Exit, the concept would have worked. Once the 'blow-out' was patched, there was very little air loss, proving that the rear of the TBM was sealed well. It also became apparent that Compressed Air work was possible, although there still remained the risk of whether the ground could hold the air. As it turns out, the risk the Owner took to eliminate the remaining two Safe Havens was the right choice, and workers were able to safely and efficiently conduct the necessary Cutterhead maintenance on both the subsequent tunnel drives.

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CONCLUSION

Contract CQ031 demonstrated that complex tunneling work, even with minimal cover, can be successfully accomplished in the middle of a dense urban area, with challenging soil conditions, and beneath a mainline railroad. The keys to its success were careful planning, a focus on safety, cooperation between the parties, continuous monitoring of instrumentation, and good communication with stakeholders.

PART 19

Risk Management

Chairs

Eric C. Wang
Jacobs Engineering Group, Inc.

Derek Zoldy
AECOM

DRILLING AND BLASTING OPEN CUTS IN THE UPPER EAST OF MANHATTAN—PART OF THE 86TH STREET STATION FOR THE 2ND AVENUE SUBWAY LINE

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Anil Parikh ▪ MTA Capital Construction Company

ABSTRACT

As part of the ongoing 2nd Avenue Subway program, which will relieve major congestion for subway riders on Manhattan's east side, MTA Capital Construction (MTA CC) contracted Skanska/Traylor Joint Venture (STJV) to excavate and concrete line the station cavern for the 86th St Station. Before excavation of the cavern could begin, STJV had to drill and blast a 36' wide × 190' long open cut on the south end of the station 70' below street level which would provide the access for the cavern portal. The excavation started east of 2nd Avenue next to a high rise apartment building and finished west of 2nd avenue underneath the basement of a building STJV demolished and adjacent to two historic/fragile brownstone buildings. To complicate matters, the excavation went below 2nd Avenue, under live vehicular and pedestrian traffic plus a myriad of utilities including gas, water, sewer, electrical and communication lines.

This paper will describe the means and methods employed to balance expedited drill, excavation and rock support production with minimal impact to the community and surrounding buildings. The obstacles that were overcome include the stringent vibration criteria, continuous public observation, low headroom when working with utilities, very tight surface staging, controlling environmental impacts (noise, dust, air overpressure) and safety challenges when working in one of the most densely populated and influential areas of Manhattan, the Upper East Side.

BACKGROUND

Early History

As the 1940s started there were three mass transit lines servicing the east side of Manhattan; the Lexington Avenue subway, the Third Ave Elevated train (El), and the Second Avenue El (see Figure 1). However, due to the changing nature of the area and to make room for construction the Second Avenue El and Third Avenue El were removed in 1942 and 1956, respectively.

Since then the population has steadily increased and no new mass transit has been constructed. This has lead to major overcrowding on the Lexington Avenue subway which was lead to longer trip times, less reliable service and a more unpleasant experience. In addition bus service has become overcrowded, less efficient and surface streets are congested with vehicles leading to a decrease in air quality.

These changes have not gone unnoticed, and since 1929 there have been plans to construct a new subway along 2nd avenue. The plans even got as far as the construction phase in the 1970s but the city's financial problems eventually led to the suspension of the project before completion.

Modern History

In 1995 the MTA New York City Transit department commissioned the Manhattan East Side Alternatives Study (MESA). The goal of the study was to determine current problems and come up with alternatives for improving them. Among the biggest problems were the overcrowding on the Lexington Avenue line, the severe traffic congestion, and lack of convenient access to mass transit for the east side population.

With the help of the public the MESA team came up with over 20 alternatives ranging from the do nothing alternative to improving bus service and lanes, to a full 2 tunnel subway. As the planning progressed the support of the community groups, public officials and the public in general gained for the full length 2nd Avenue subway line. The line would run from 125th street down to the financial district at Hanover Square. The project was broken down into four phases, the first of which would be from 96th to 63rd street as an extension of the existing Q line and the new line (T) would tie in as part of Phases 3 and 4 (see Figure 2).

In 2006 the Federal Transit Authority granted the MTA authorization to release the Final Design Contract. The first construction contract was awarded in March 2007 with the official groundbreaking in April 2007.

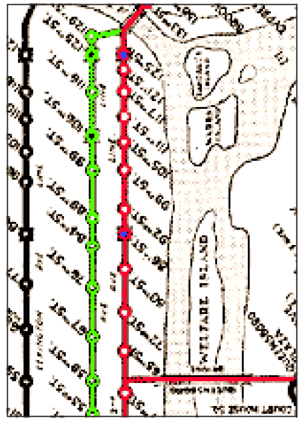

Figure 1. Mass transit lines in 1940

Figure 2. Planned route

SITE LAYOUT

The 86th street cavern was the last major excavation released for construction for the Phase I portion of the 2nd Avenue Subway. A joint venture of Skanska USA Civil Northeast and Traylor Brothers (STJV) was the low bidder and given the notice to proceed in August, 2011. The station will measure 60 ft × 70 ft × 950 ft long with the top of the cavern being about 35 feet below street level. Access to the main cavern would be provided by a construction shaft on the north side of the project at 87th street and a large open cut at the south end of the project at 83rd street. The open cut is an approximately 180 foot long × 40 foot wide excavation ranging from well west of 2nd avenue all the way to the building line on the east side of 2nd Avenue.

On the west side of 2nd Avenue are older brownstone buildings, around 4 stories in height, with rubble foundation walls. These buildings were defined as fragile or historic buildings. On the east side of the open cut is a 30 story residential building. Across the street on the southeast corner of 83rd and 2nd was a large diner, which constantly had patrons at all hours. It was very apparent that the project would have public eyes on it from all sides and even above.

At the beginning the open cut was divided into two sections called Ancillary 1 and the South Shaft (see Figure 3).

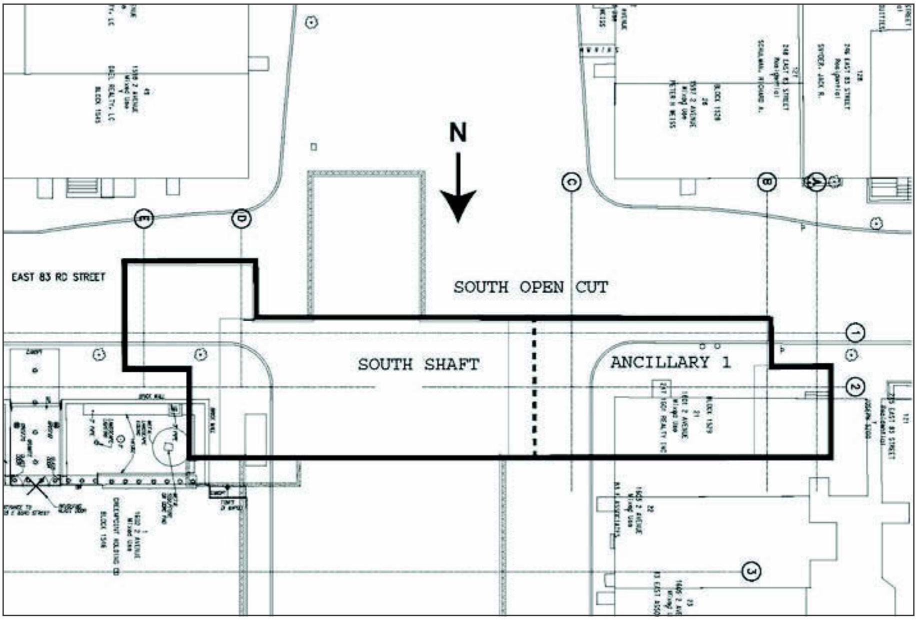

Figure 3. Site layout

SITE PREPARATION

Instrumentation

Before any excavation/demolition work could begin the contract required an extensive instrumentation program to be installed. Taking a closer look at the buildings surrounding the south open cut gives insight as to how comprehensive the coverage was. 1603 2nd Avenue is a four story brownstone adjacent to the building that was to be demolished by Ancillary 1. It had the following instrumentation installed:

- (15)—Deformation monitoring points (DMP's)
- (4)—Horizontal tilt sensors (HTS)
- (10)—Vertical tilt sensors (VTS)
- (2)—Vibration Monitors (VM)

On east side of the excavation is a 30 story building with the following instrumentation:

- (11)—Deformation monitoring points
- (4)—Horizontal tilt sensors
- (8)—Vertical tilt sensors
- (4)—Vibration Monitors

The deformation monitoring points are constantly scanned by Total Station monitors with manual checks done twice a week. The DMP's along with all the other instrumentation devices are linked to the internet and setup to automatically send out emails if any warning threshold or limiting value is reached. The DMP's have a limiting value of only 0.3". The tilt sensors have limiting values of 0.012 in/ft. In terms of the vibration criteria, the building at 1603 was classified as fragile and had a warning limit of only 0.5 in/sec and a limiting value of 0.8 in/sec. This can be contrasted with the limits set on the ore modern the 30 story building which had warning and limiting values of 1.5 in/sec and 1.92 in/sec, respectively. This is due in large part to the much more robust construction of the 30 story building.

South Shaft

When STJV took over the site, construction had already begun on the south shaft, which measures approximately 4,000 ft^2. A previous contractor had already excavated from elevation 160 to 130 in the area from the west edge of 2nd Avenue to the east edge of the excavation limits. This was a mixture of soft ground and rock excavation. They used a combination of piles and lagging for soft ground and rock bolting as their support of excavation (SOE). In order to minimize impact to 2nd Avenue, steel beams had been installed and decked over with precast concrete panels to cover the opening.

Upon taking over the site, STJV applied a layer of 4" fiber reinforced shotcrete over all exposed rock. This was done primarily for safety consideration to prevent any rocks from coming loose or falling down due to the vibrations that would come from future blasting. STJV also drilled line holes along the perimeter using an air track drill. This unit was preferable because of its low boom height, an advantage while working around the utilities.

A predominant feature that was noticed in this excavation was the large number of utilities running under 2nd Avenue (see Figure 4). They consisted of the following:

- 48" diameter water line
- 12" diameter water line
- 48" diameter sewer line
- 10" gas main
- 30" gas main
- Electrical ductbank
- ECS ductbank

These utilities ranged from 25 feet to 8 feet above the current grade of the excavation. Knowing these utilities would be in danger of being struck by the machinery or fly rock, STJV decided to encase the utilities in 3x10 wood / plywood.

Figure 4. South shaft utilities being protected

Figure 5. Excavating down to Elevation 130

Ancillary 1 Site Prep

The first step in preparing Ancillary 1 for excavation was demolishing an existing 4 story building on the corner of 83rd street and 2nd Avenue. STJV began removing this building by doing asbestos remediation, lead remediation, and disconnecting/rerouting all utilities. Once the building was isolated and safe STJV's subcontractor, Russo Demolition, took the building down floor by floor and trucked the debris out. After removal of the building, STJV excavated and poured a 3'-0" thick concrete SOE wall along the new perimeter adjacent to 83rd street and in the NW corner between the buildings. To account for the decking loads and other surcharges the SOE wall was pinned to the rock below with 1$\frac{3}{8}$" dwyidag rods, grouted into place and tensioned. Any remaining soft ground / fill from the basement level was removed using a combination of a CAT 321 in the hole feeding a CAT 322 on the surface which loaded out trucks.

Once the building was removed STJV was left with a 2700 ft^2 area that needed almost 20'-0" of rock removal to reach elevation 130 (Same as south shaft). Excavation of the soft ground to elevation 150 revealed that the adjacent building at 1601 2nd Avenue was not on structurally sound rock. STJV's in house engineering department came up with a plan to underpin the building with concrete blocks doweled into the stronger rock below. The instrumentation on the building was closely monitored as the sequential underpinning took place. In addition to the underpinning, a 1'-0" concrete wall was poured up against the rubble foundation wall to help protect it now that it was exposed. This work was done simultaneous with the excavation in order to minimize schedule impacts.

The final prep work to make the union between the Ancillary 1 area and the south shaft was to remove an abandoned vault under the west sidewalk of 2nd Avenue. STJV managed to do this by getting a permit to put pedestrian traffic into the right lane, demolishing the sidewalk and vault, installing a new steel beam across the excavation and extending the concrete decking over the sidewalk. This took three days and pedestrian traffic was restored back onto the newly installed deck panels. This allowed the "hole through" between the two areas.

After the temporary "hole through" the Ancillary 1 area was still at elevation 145, about 15' higher than the south shaft (see Figure 5). To match the elevations, STJV drilled 2" diameter holes in a 2' × 2' pattern throughout the area to soften it up and then brought in 321 with hammer to break the rock. Rock was stockpiled in the SW corner where a 322 reached over and loaded out the material. When the Ancillary 1 area was lowered to the same elevation as the south shaft, the South Open Cut was level and whole and blasting operations could commence with little to no interruptions.

Figure 6. Matting during blast

First Lift: Elevation 130 to Elevation 120

The first blast in the South Open Cut took place on April 11th, 2012. Because of the strict vibration considerations it was decided to make sure the first shot was a slash shot. Using the two 16" diameter burn holes drilled previously as relief, an 8' × 8' × 10' hole was chopped using a 308 excavator to provide relief. Drill holes were 1⅞" in diameter and were drilled 8' in depth in a 24" × 24" pattern. Due to the proximity of nearby utilities and the access hole above, the shot was triple matted with 8' × 8'–50 psf rubber blast mats (see Figure 6). The CAT 308 and 8'x8' mats were used because of the low headroom created by the utilities above. The utilities also required us to use a short boomed D3 hydraulic drill that was only capable of holding a 12' drill steel.

The first lift required 57 individual blasts, the last being on June 8th, 2012. A typical blasting sequence consisted of loading a shot, matting it, performing the blast, loading next shot, moving mats from 1st shot to 2nd and blasting again. During this lift STJV would get between 3–5 blasts per day. Special consideration had to be given to shots around the 30 story building and the fragile building at 1603 2nd Avenue. When working around these buildings STJV was sure to blast away from them, creating as much relief as possible before taking the shots directly adjacent to building.

Second Lift: Elevation 120 to Elevation 110

The first blast of the 2nd lift was done the same way as the first, only this time the 308 was swapped out with a CAT 321. Using the two 16" diameter burn holes, an 8' × 8' × 10' hole was chopped to provide relief. Production holes were drilled by two D3 hydraulic drills, this time at a 28" × 28" pattern. STJV continued using 8'x8' blasting mats but also added 10' × 14' mats to allow for larger blasts and quicker setting of mats. The larger spacing between holes and the bigger mats allowed the 2nd lift to be taken in only 44 shots and finished on July 16, 2012.

Third and Fourth Lifts: Elevation 110 to Elevation 90

The third and fourth lifts were done much the same was as lift 2. The big difference was at this point we had the space and headroom to allow for a second 321 to be placed in the hole to continue mucking operations (described later) and blasting simultaneously (see Figure 7). Due to our increased vector distance from the building foundations, blasting patterns were increased to 30" × 30" in some areas and the third lift was completed in 46 shots on July 30, 2012 and the fourth lift was taken in 35 shots, ending on August 21, 2012.

Mucking Operations

For the first two, lifts space and headroom greatly limited how many activities could go on simultaneously. Therefore it was typical to blast for 3 days, stockpile the rock, and then muck for two days. This had obvious schedule impacts but was cost effective, because we had the work elsewhere on the project to move workers around as needed. In order not to drive over any drilled holes muck was taken to the nearest exit point. On the east side that was to 3.3 CY skip pans lowered by a 45t crane that would haul and empty the boxes directly into dump trucks for removal from site (see Figure 8). On the west side, rock would be stockpiled in southwest and a 345 positioned on the corner pedestal would reach down and load out trucks/trailers (see Figure 9). By the third lift it was possible to muck and blast on the same shift, however it did not keep up with the increased rate and size of blasting so a 2nd shift was added with the 45t crane to expedite removal and keep blasting operations consistent. By this time the 345 required installation of an "extender arm" to increase the depth it could reach to in order to assist in the rock removal. Then by the fourth lift our electrical gantry crane was operational over the east portion of the open cut. This gantry could lower down 15 CY boxes, which allowed mucking operations to keep up with only one shift.

Figure 7. Multiple operations

Support of Excavation: Rock Bolting and Shotcrete

After each ten foot lift we were required to install rock dowels as part of our SOE design. Patterns ranged from 6'x6' patterns to 4'x4' depending on the rock conditions and the load above. Dowels were 10'-0" long #10 dwyidag bars. Holes were drilled using an Atlas Copco D3 hydraulic drill which could drill at the 10–15 degree downward angle required by design (see Figure 10). Slow set resin cartridges would be inserted into the hole and the dowels spun in using the D3 drill. The resin would setup and nuts and plates put on the tails of the bolts. It was important to keep up with the bolting for safety reasons and also because of the limited height range of the D3. Therefore it was decided on several occasions to bolt on Saturday's in order to not disrupt

Figure 8. Mucking with 321 and 45t crane

the drilling/blasting/mucking operations. For quality control 3% of all installed bolts were tested.

The second part of our SOE design was applying a 4" structural shotcrete layer. Because of the narrow size of the open cut a Meyco Oruga shotcrete robot was used (see Figure 11). The advantages were it was small enough to be lowered and raised by either the 345 or the 45t cherry picker, it has great maneuverability and enough range to cover the required areas. This robot does not have a hopper, so a Meyco Suprema pump was setup on the surface to receive the wet mixed shotcrete. Lines were run to get the shotcrete and accelerator from the pump to robot. A certified nozzleman would apply and shotcrete test panels were sprayed each day for testing by a third party lab.

COMMUNITY ISSUES

Dust Control

It is accepted in construction that with drilling and blasting comes a certain amount of dust. The challenge on 86th street was minimizing this impact to the community. Some solutions were easily identified, such as wet drilling with the Atlas Copco D3. Another was constantly maintaining a water hose while the 321 chipped tights, moved muck, or broke rock. Our biggest challenge was keeping away the impact of shotcreting and blasting. To protect the public, misting sprayers were setup at the corners of the opening to "knock" down any particulates in the air. The sprayers were engineered and fabricated in house, taking a 3 foot long cylinder and welding on air and water hookups to spray a fine mist over a wide area. These could easily be moved as the robot moved around site and did not create interference with the actual shotcrete or blasting operations. To ensure the engineered controls were adequate a Mini RAE 3000 was setup to measure

Figure 9. Mucking with 345

Figure 10. Rock bolting with D3

Figure 11. Shotcreting with Oruga

VOC with limits set at 5 PPM and a Dust Trak III to measure particulates (PM10) with an allowable limit of 0.100 mg/m^3.

Noise

Noise was another large concern. In respect to the public, the drilling was kept to one shift. Excavators and the 45t crane would still move, load out and break rock on second shift. However, concentrated efforts were made to keep the nosiest activities to a minimum on second shift, especially near the buildings. STJV also decked out portions of the west area to limit the noise. Sound blankets were installed along the entire perimeter of the work zone to protect pedestrians and façade panels were added to the surface mucking systems which surrounded the shaft opening in the roadway decking.

Safety

Worker safety was of utmost importance to the project team, but equally important was protecting the public. The site is right in the middle of one of the busiest areas in Manhattan. A difficult challenge was moving all our trucks in and out of the site. This required the use of STJV flaggers working with Traffic Enforcement Agents provided by the city. With a steady stream of vehicular and pedestrian traffic this took much coordination and focus to keep things organized.

Although every measure was taken to ensure fly rock would not escape the site, the possibility did exist. Working with the FDNY, protective limits were established to stop all vehicular and pedestrian traffic. A three whistle system was used to alert the public to any blast.

- 1 Whistle = 3 minute warning
- 2 Whistles = 1 minute warning
- 3 Whistles = All clear

This system, in addition to stopping all passersby, was paramount to ensure the public's confidence that STJV was looking out for them.

CONCLUSION

Due to the long history of overcrowding and congestion on New York's East Side, the 2nd Avenue Subway has been a long time coming. The NYC MTA and their teams made the decision to build what everyone knew would be a difficult project to complete. By forming a strong partnership with Skanska Traylor JV, the team has worked diligently through a litany of challenges; low headroom, crowded population, heavy traffic, difficult working constraints; to build a project that New York City can be proud of with the least impact on civilian life above. And in that goal, they succeeded.

PREDICTING AND CONTROLLING EXCAVATION VIBRATIONS IN URBAN AREAS USING THE DRILLING-AND-BLASTING METHOD

Oleg Sarantsin ▪ YIT Construction, Ltd.

Juha Kukkonen ▪ Sandvik Mining and Construction Oy

ABSTRACT

Blasting creates a stress wave in the rock which not only causes loosening and fracturing of the rock mass, but also transference in the intermediate particles, i.e., vibration. If the vibrations created by blasting are not controlled, damage to constructions and sensitive instruments nearby can be caused, as well as possible danger to people. Therefore, excavation needs to be planned in advance and performed in a way that no hazard or inconvenience for the surrounding environment is caused. This sets tighter limitations for vibration monitoring, blasting design and implementation in urban areas.

This paper concentrates on vibrations and their propagation caused by blasting in underground excavations. Some real data is studied and the results are compared to theoretical values in vibration prediction. The study reveals that one of the most important variables in hard rock excavations is the conductivity figure (rock mass quality index), which is based on measured variables such as the distance between a measuring point and a blasting site, the peak particle velocity and the maximum simultaneous charge. By planning and controlling blasts with the right tools, it is possible to optimize dependencies between allowed vibration limits and maximum simultaneous charges. This kind of monitoring of excavation vibration enables precise prediction of the conductivity figure and results in safe excavation in urban areas.

INTRODUCTION

Tunneling by drilling and blasting is the most commonly used excavation method in Scandinavian countries, where excavation usually faces hard rock. Every blast creates a vibration effect on the surrounding environment. Some of the vibrations can cause negative side effects, especially in urban environments when blasting close to buildings and sensitive equipment. These effects are impossible to avoid totally, but they can be minimized by using the right tools, techniques and working methods. A responsible contractor strives to minimize these effects by applying these right tools, techniques and working methods.

Urban environments set tight limitations for vibration threshold values. The only way to avoid exceeding the threshold values is to predict and monitor vibrations from every blast and to react to the results using real-time planning. Planning is composed of the drilling pattern, charging and ignition plan combined with vibration measurements. Good blast design and execution are essential to successful tunneling operations.

Transmission of Blast-Induced Vibrations

Blast-induced vibrations are relatively small but fast movements of the ground which are initiated by blasts and transmitted through the ground as seismic waves. These waves travel at speeds of 4,000–6,000 m/s in intermediate rock. For comparison, waves travel in clay intermediates from 400 to 1,800 m/s. Vibration is also transmitted from the ground to all structures and buildings near the blasting site. Vibrations can be modeled and measured using several measuring units, such as peak particle velocity (PPV) [mm/s], acceleration [mm^2/s], displacement [μm], frequency [Hz], etc. Blast-induced vibrations are usually high frequency, and are therefore known to abate very quickly in relation to distance. Vibration is reduced, on average, to one quarter when the distance is doubled in hard rock. Reduction is even faster in looser ground such as clay and silt. The reduction rate depends on the quality of the rock (i.e., the amount of fractures) and the moisture conditions. Every crack reflects, refracts and reduces the vibration. Water and moisture inside rock formations slow this reduction down, i.e., improve transmission.[1]

Controlling and Minimizing Vibration

Vibrations caused by blasts can be limited and minimized in several ways. The first action when approaching the threshold value of vibration is to analyze the drilling pattern, charging and ignition plan. The analysis is based on vibration measurements. Based on the analysis, the aim is to point out where the maximum charge per delay is located or, in the case of an excessive burden, how this could be reduced. The second option is to reduce the blast length or diameter of the hole, or ultimately to divide the blast into smaller sequences. Also, the choice of explosive and detonators and the quality of the charging work all have a great effect on the result. An optimized blasting result can be achieved when the burden is accurately planned in the drilling plan and the execution of the pattern is accurate. Excessive burdens increase vibrations rapidly. Too small a burden can result in excessive air blasts and throw. When drilling a small burden, the risk of drilling holes together side by side increases. It also increases the risk of holes igniting each other by slashing through the rock. The optimum blasting result can be achieved when detonators and explosives are selected and charged so that they all detonate as planned and there is enough time between the detonations of individual holes to prevent collective shock waves and to allow sufficient rock swell.

Equipment for Monitoring Vibrations

Requirements for vibration control increase in urban environments because of the risk of damaging nearby buildings and sensitive equipment. The positions of vibration monitoring instruments are planned carefully due to the requirements of the surrounding environment. All potential vibration-sensitive structures and equipment are analyzed. Sometimes it is necessary to use several monitoring instruments in the most critical places. In buildings which contain sensitive equipment such as computers, servers and laboratory equipment, monitoring instruments are placed both on the structure of the building and on the equipment. The damage risk for building structures is mostly measured by peak particle velocity (PPV) and that of equipment by acceleration. In both cases, factors such as frequency and displacement have to be investigated as well.

Nowadays, it is possible to measure vibration three-dimensionally (see Figure 1), i.e., all three axes of movement, with advanced measurement instruments, when controlling the damage risk for buildings and equipment. 3D measurement is required because it is difficult to pinpoint the vibration-critical component, and therefore also the vibration-critical direction. Due to the high amount of measurement points and blasts, all measured data is sent to a server where it can be accessed and analyzed by a web-based reporting analyzing tool. All necessary data is available in real time for all users with access. The system allows all units of vibration to be analyzed and, for example, to adjust the time history curve or to perform frequency and regression analyses in all three dimensions.

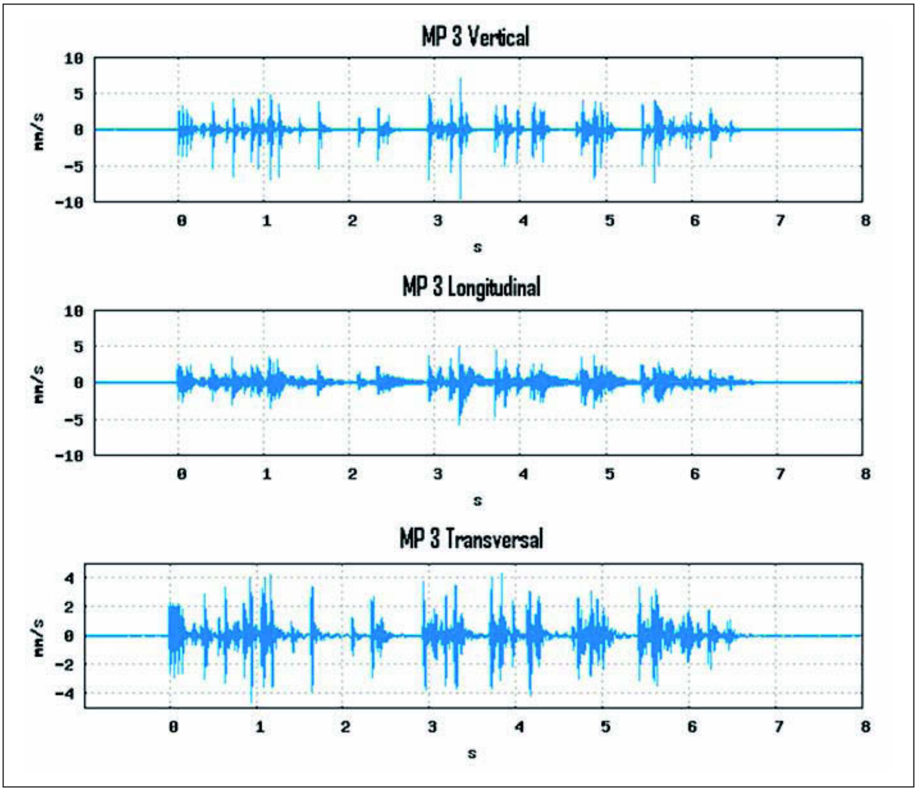

Figure 1. Three-dimensional vibration curve from a blast

The three-dimensional curve can be printed out or viewed for every blast in real time using the internet. This allows the possibility to analyze the blast, especially when approaching the critical measurement point and its threshold value. The three-dimensional curve from a blast shows the highest peak particle velocity in relation to the time history and it is essential information when planning the next blast.

IMPORTANCE OF PREPLANNING AND DRILLING ACCURACY

The overall productiveness of a tunneling or underground construction site is dependent on many different factors. As in any other project, good planning is half the task. One very important part of preplanning is the drilling pattern design. A thoroughly planned drilling pattern together with accurate drilling equipment makes the probability of a successful outcome of the project higher. The profile of the excavated tunnel or space can be controlled with blast management and correct placing of drill holes, i.e., the correct start and end points for the hole. The accuracy of the drilling equipment is dependent on its mechanical and technical features, but also on careful navigation and caution in the phases of the drilling cycle. The ability of the machine to drill the hole in a planned location, direction and length is actually a feature that created the foundation for the whole economical excavation.

The accuracy of drilling is essential when excavation advance is considered. Particularly, the accuracy of the cut holes, and most importantly the accuracy of the holes end locations, are fundamental. The blasting initiates from the end of the blast holes, and therefore it is of the essence that the burden and spacing between holes

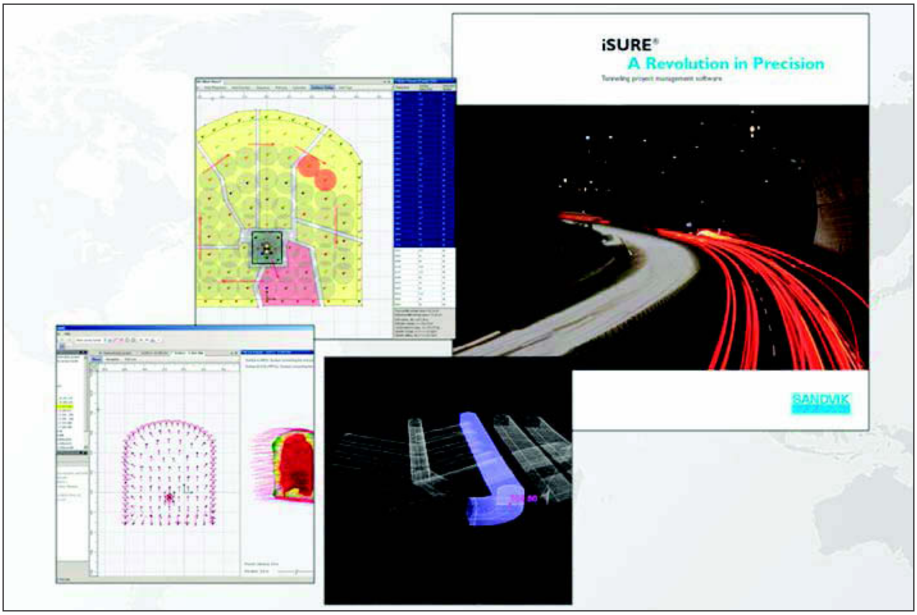

Figure 2. iSURE® tunnel management software

in the blast plane are as planned. If the drilling is not accurately executed, there is no possibility to correct the blast by charging.

The pull out of the blast, i.e., the portion of the drilled round length (in percent) that is actually loosened in blasting, is also a sum of many elements. Correct hole placement, accurately performed drilling and the proper blasting method have a great influence on the pull out, but also the selection of cut type and placement affect the final advance. As the blasting initiates from the cut, it is essential from the point of view of a successful excavation that the design, placement and accuracy of the cut are appropriate.

As mentioned, the most critical thing is the situation at the end of the round where the blasting initiates. Therefore, the burden and spacing between the holes need to be designed to meet the challenges of the blasting, and furthermore the accuracy of the drilling needs to meet the designed positions as planned. Even a small deviation can cause a charged cut hole to meet a reamed hole or make the burden too high. An exceedingly high burden can further cause breakage or plastic deformation in the cut, resulting in a shorter advance. On the other hand, when the burden and spacing between the holes are according to plan, the energy of the explosives is used correctly to break the rock between the holes and to move the rock mass to create open space for the next row of holes to be able to initiate, instead of distributing the energy incorrectly causing extra vibrations.

Tools for Controlled Blasting

Bearing in mind the importance of the accuracy of hole end locations, it is also important to say that the most optimal and logical place to design the drilling pattern is therefore at the end of the round, i.e., in the blast plane.

iSURE® (intelligent Sandvik Underground Rock Excavation) software (see Figure 2) is a tool for managing tunnel or underground excavation projects. The drilling and blasting pattern design is made where the excavation is most critical and where the blasting initiates that is in the blast plane. Taking the design at the end of the round also enables parameterization of the drill holes and burden calculation for optimized hole locations. One element of the drill plan design is to specify the explosives used in different parts of the pattern. The degree of charge, relative strength of the explosive and fracture zone are used for calculation purposes during the design process. Based on the information specified during the design process, the total consumption of explosives per round and charge per delay values can be illustrated, for example. The aim of this is to achieve more accurate drilling and, furthermore, better quality in excavation. A controlled situation at the end of the round enables better control of vibration and smoother blasting.

Detonators and extra delay (a.k.a. group or surface delay) detonators can be included in the design process in iSURE®. The software enables real-time monitoring of charge per delay values as the design advances. If selected, the information on the real delay times with or without extra surface delay, the number of detonators initiating at the specified delay time and the amount of explosives initiating simultaneously is available. As the excavation advances, the designer can easily revise the vibration measurement results and go back to the drilling and blasting pattern design to trace the cause of the increased vibrations and make modifications as needed.

In addition to real time charge per delay illustration, the design process is made easier for the user by enabling one to simulate the blast. Simultaneous detonators on a specified delay time can be illustrated and highlighted while the already-initiated delays are displayed faintly.

The explosives used in a tunneling or underground construction site should be selected so that the blast itself is effective taking the rock conditions into account, but also so that the remaining rock stays as intact as possible. Because of this, suitable explosives normally are such that the degree of charging can be controlled precisely, such as cartridge products and emulsion.

By using cartridge products, the degree of charging can be controlled accurately and usage is easy. On the other hand, the explosives need to be stored on site and there is a need to have facilities for storage. This might be problematic in some countries due to legislation and restrictions in tunneling in urban areas.

Use of site-sensitized emulsion explosives is becoming more common in tunneling and underground excavation. The emulsion is a mixture of an emulsion matrix and sensitizing additives. The mixing process is done at the charging platform in the tunnel face just before pumping the emulsion into the blast holes. The explosive capabilities are achieved normally approximately 30 minutes after mixing. Therefore, transportation and storage of emulsion is more practical since it is not considered an oxidative substance before sensitizing is completed.

When the blast design is made from the vibration minimization point of view, the focus is on minimizing the charge per delay, i.e., controlling the amount of explosives initiating simultaneously with the same delay time. It is still important to remember that the required specific charging value must not be decreased. This could lead to a situation where the energy of the blast holes is not sufficient to break the rock between the holes, thus creating even more vibrations. [1]

Detonator design plays a key role when excavation vibrations are strictly limited and the aim is to control vibrations. Non-electric detonators are commonly used, as they are simple and fast to install and also relatively inexpensive. The downside of non-electric initiation systems is the limited or exiguous amount of pre-set delay times and delay connectors. This sets higher requirements for the designer or time-consuming estimations to ensure the final delay timing design to keep the vibrations at a low level. These facts prepare the way for electronic detonators that allow the designer to determine extremely precise timing for each detonator with an accuracy of 1 ms in the range of 0–14,000 ms. The tolerance in time deviation of electronic detonators is less than

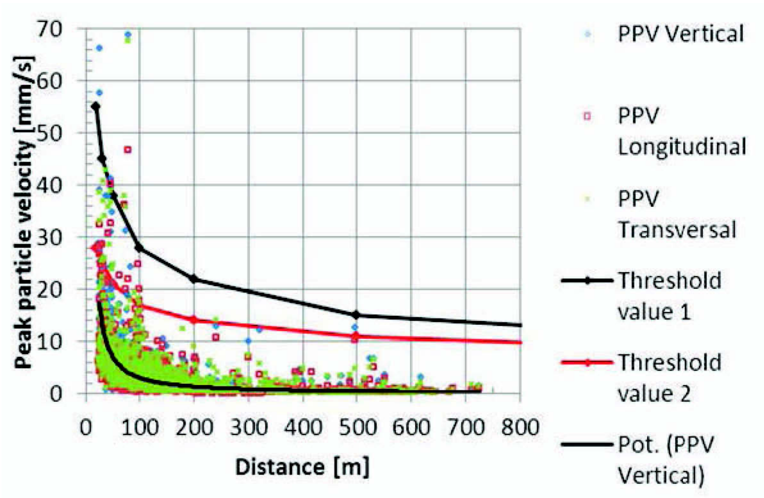

Figure 3. PPV dependence on geometric attenuation (distance)

0.02%, which is remarkably accurate. This guarantees simultaneous initiation of the desired holes and enables the designer to very accurately specify the collective effect of each blast hole to initiate.[1]

EMPIRICAL STUDY

When using the drilling and blasting method for excavation in a hard rock environment, it is essential to know what kind of bedrock is being faced. This is especially important when planning sensitive underground blasts in urban areas where vibration limitations and threshold values are strict. The vibration conductivity figure (k) is the main parameter for predicting of the vibrations caused by blasts.

An empirical study was carried out in 2012, in which vibrations caused by underground blasts were studied. The main focus was on vibrations and their propagation in a hard rock environment. The aim of the research was to determine the differences between theoretical and real measurements in vibration prediction. The aim of researching differences between theoretical dependencies in theory and real values was to achieve safer and more economical excavation by blasting.

The research was based on three underground tunneling projects in Helsinki. The urban environment was a common feature for all three projects. All the vibrations caused by blasts were measured with three-component vibration meters. A total of 2,233 measurements were analyzed and studied. The most important measured variables of vibration were peak particle velocity, frequency and acceleration. Based on the measurement results, the vibration conductivity figure was evaluated and compared to the values presented in the theory. The purpose of the results was to describe the vibration conductivity features of the bedrock and to find out what causes are behind abnormal measurement results.

In terms of vibration transmission, geometric attenuation is one of the affecting factors when examining high peak particle velocity (PPV) measurements compared to distance. Over short distances, vibration attenuation is higher than over long distances. Vibration is reduced on average to one quarter when the distance is doubled. From all measurements from distance of over 100 meters, excluding some exceptions, PPVs were under 10 mm/s. For distances less than 100 meters, PPVs decreased, modeling the power trend line. PPV attenuation results from geometric and material attenuation. Material attenuation depends on the conductivity figure of the local bedrock. Figure 3 shows the results of measured PPVs and their attenuation compared to distance.

The maximum conductivity figure is usually k = 400. The quality of the bedrock is the main attribute of the conductivity figure. Each blasting site has a characteristic conductivity figure which is determined after the first blast. For a reliable conductivity figure, at least 20 blasts and their measurements are needed. When the charge per delay and the distance between the measuring point and the blast are known and the PPV is measured, the conductivity figure can be calculated using the following formula:[2]

$$k = \sqrt{\frac{v^2 R^{1,5}}{Q_m}}$$

v = PPV [mm/s²]
R = distance [m]
Q_m = maximum charge per delay [kg]

When using the conductivity figure in vibration prediction, it is important to consider the fact that the conductivity figure can change during blasts. For example, cracking of the rock and clay pockets reduces the transmission of the vibration and also the conductivity figure.

CONCLUSIONS AND RECOMMENDATIONS

Tunneling with the drill and blast method in urban environments always has a small probability of causing damage to nearby buildings or sensitive equipment. The main damage risk is caused by vibration of the foundations because of the blast. Using the right tools, techniques, working methods and technology, these risks can be minimized and properly controlled.

Predicting vibrations is the first tool for planning dependencies between the maximum simultaneous charges and threshold values of vibration measurement points. The most important thing is to analyze the drilling pattern, charging and ignition plan combined with vibration measurements. The point of the analysis is to find out the points where the maximum charge per delay or burden could be reduced.

The latest technology in drilling equipment and in design software makes more accurate drilling and effective charging possible and thus enables safer blasting results to be achieved, even in very difficult excavation conditions in urban environments.

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TUNNEL MYTHS: A REVIEW OF CURRENT PRACTICES

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Gerhard Sauer ▪ Dr Sauer Corp.

ABSTRACT

Tunneling is one of the industry's solutions to improve our environment and consequently Quality of Life. Putting infrastructures and services underground definitely contributes to a more pleasant urban surface. But how to minimize its long-term construction impact and the "high risk/cost" myth? The authors are trying to identify preconceptions in this particular industry and offering more reliable approaches in terms of cost and schedule based on an overall control mechanism.

SUMMARY

Tunneling is one of the industry solutions to improve the quality of life of the citizens. Transferring underground infrastructures and services we can create free green areas on the surface and liberate it from current congestion. Tunneling means not only to bury the structures but how to do it on the most professional approach. Innovation is the solution and to fight against the existing inertia is the way. With today's technology (tool box) virtually any sizes of tunnel or cavern can safely be mined at reasonable cost and time. Voltaire's advice: "The better is the enemy of the good" calls for more competence and less "politics." Providing underground transportation and storage facilities improves the environment; we all agree. The paper tries to identify preconceptions and myths proposing alternatives which could provide a more reliable approach in terms of cost and schedule based on the overall control.

BASE OF ASSUMPTIONS

Internet is a revolutionary invention that is changing the today's world "One Way" mass information system, allowing the communication among people located in any hidden corner around the globe easily. But also this "Multiple Way" system still lacks the inconvenient, the correctness and accuracy of the info is not always verified and sometimes remains in the clouds forever. Tunneling is not excluded and wrong approach or misconceptions are divulgated, in the ever higher amount of conferences and (on line) technical magazines, worldwide. This is an attempt to call the reader's attention to some of those misconceptions with the hope that the tunneling community will understand this as a positive approach in order to offer to the citizens cost effective solutions to improve their quality of life but also minimizing the disturbance during the construction period, to the neighborhood and third parties (Figure 1).

Tunnels appear on media mainly negative, in case of collapses, fire or accident with losses of human life and much more dominant compared to other transportation facilities. We, the tunnelers, are technicians. The current state of the art technologies allow us to create undergrounds structures in all kind of ground safely in a reasonable time. Let's do it by minimizing the social cost and the taxpayer will look at our work with attraction. In this way, the authors have prepared a list of tunnel rules, still followed in many projects around the world, which can be avoided or at least reconsidered with help. These rules include design, construction and supervision, the three pillars on which a successful Project is based. If some of them are partly forgotten or misleading, it not only compromises the end product but also can lead to a tunnel collapse. Proper design, right construction and close supervision, all together, are the simply way to succeed in our field. Failure in one or two pillars will produce, intentionally or unintentionally, tragic consequences.

Figure 1. Alaskan Way viaduct replacement (Seattle, WA)

DESIGN

Starting from the design aspect, common rules applied are the following.

Cut and cover structures are cheaper than mined caverns. This is an assumption followed in many urban infrastructures (like Metro systems) to design stations, crossovers and operational structures. In reality the number of aspects to be analyzed before taking the right decision is considerable. It is true that surface methods are simple and cheap in open field areas without interference with sensitive utilities and only for shallow infrastructure. But this is usually not the case in urban environments were the excavation of an underground space requires the relocation of a substantial number of utilities like sewage, water, phone, light, gas, communication, etc. Every one of those utilities is owned by different organization and the removal and relocation could take years especially if the contractor cannot manage it at an early stage thus has no control in the selection of different/other possibilities. That means the project completion is controlled by third parties. Their interest may not always be the completion in time. This fact may sometime this alone may be enough to reject C/C as the selected working method.

Also limits in noise and vibration are every time more and more a contractual requirement and consequently all the activities executed above ground are restricted in time and resources. Adding to these facts the stakeholders' rejection to any construction activity in front of their own business (business loss) same applies to residences. All those makes C/C a reasonable method to be applied in open but not in urban areas. Even in the case of very poor ground below water table a considered analysis between C/C and mined excavation with ground improvement should be considered before taking the final decision. Mined methods also offer a competitive alternative approach with the added advantages of time and money savings. When factoring the additional cost attributed to environmental impact, utility relocation, extended construction time and volume of material, muck, concrete and steel; mining may be the preferred method.

Tunnel portal precut is required to achieve a minimum of one diameter overburden. This conservative approach is not based in any technical basis. Current state of the art allows protecting the tunnel arch by forepooling, spiling or splitting the full section in multiple drifts (pocket excavation) in order to dig the tunnel in a safe way. This solution does not require special equipment but just the same one used in the tunnel excavation. Consequently it is a cheaper and faster solution than excavate the portal trench, build the tunnel canopy with reinforced concrete and refill and restore the original landscape.

The minimum pillar in a twin tunnels must be two diameters. The reasoning of this rule comes from the basis that the pillar must be able to carry the rock load caused by the tunnel opening without affecting the parallel one. Despite is true that the pillar supports part of the redistributed loads, these loads can be supported by the tunnel

Figure 2. Narrow pillar on East Side Access (New York, NY)

lining as well, and thus the thickness of the pillar is not as relevant as it is the design of the tunnel lining and excavation sequence (Figure 2). The load in the pillar depends on (1) ratio of tunnel diameter to high of overburden and pillar thickness, (2) Young modulus, angle of friction and rheological properties of the surrounding rock/soil. High ratio inevitable redistribute the load around both tunnels. Low overburden/ratio can easily be addressed with lining thickness. This rule has an influence not only in tunnel design but in the access structures. Portal excavation can be minimized, twin viaducts can be converted in a single one able to allocate twin track, in case of railways, and in some cases of tunnels close to the seafront, reclamation works can be reduced substantially by this fact. The economy of the project can be significantly increased accepting this fact.

Final lining in tunnels must be done by heavily reinforced steel concrete. For some strange reason, this affirmation is applicable or not, country by country. It is a kind of tradition followed with devotion in new tunnel designs. It is not the geology, the overstressed ground or the anisotropy the justification to include rebars on final lining but the tradition. It is very common on the tunneling community listen sentences like:" In this country all the tunnels are steel reinforced" Assuming an average amount of steel of 7.5 pound/c.f. and a tunnel perimeter of 80 feet, road tunnel, the savings in case of avoid the non required steel will be some 2.000 US$ per yard (1 foot thick).

Bending moment in tunnel lining must be reinforced. A stabilized tunnel is surrounded by compress strength loads and in the case of ground anisotropy some flexural strength appears around the tunnel support. But the real loads able to cause a tunnel collapse are the shear strength, not related to bending moments. Consequently the amount of steel reinforce included on tunnel design by this fact is completely useless. The easiest way to avoid it is to make the shape Ovoid!

SEM is dangerous and prone to collapse and should be forbidden in urban areas. In tunneling conferences and publications is not uncommon to hear concerns about the safety of SEM mostly in urban tunnels. SEM must be regarded as a 3D concept to understand its way to control and distribute the ground loads on these complex shapes that are the underground structures.

The success of execution of the SEM is based on four premises: Thoughtful design by an experienced engineering team, Execution by a skilled contractor, Competent supervision, and Interpretation of monitoring results. SEM is an observational method (R. Peck), which means that monitoring (in-situ-measurements) of deformation in the ground and stress in the initial lining (shotcrete) is essential to the actual support means. A weak design or poor performing contractor can be improved always by proper supervision and interpretation, avoiding collapses and damages to third parties (see Table 1).

Table 1. Major tunnel losses from 1995 to 2005

O/C Y	Project	Type of Contract	Method	Type of Loss	Cause of Loss	€m
1994	Great Belt Link, Denmark		TBM	Ingress of water		32
1994	Munich, Germany		NATM	Collapse	Faulty design (soil)	2
1994	Heathrow Express Link, UK		NATM	Collapse	Faulty workmanship	150
1994	Tapei Metro, Taiwan		TBM	Ingress of water	Faulty workmanship	12
1995	Los Angeles Metro, USA		TBM	Collapse	Faulty workmanship	16
1995	Taipei Metro, Taiwan		TBM	Ingress of water	Faulty workmanship	30
1999	Hull Yorkshire Tunnel, UK	D&B	TBM	Collapse	Faulty design?	64
1999	Anatolian Highway, Turkey			E/Q	E/Q	121
2000	Taegu Metro, Korea		C & C	Collapse	Faulty design/ work	13
2000	TAV Bologne—Florence, Italy		NATM	Collapse		5
2002	Taiwan High Speed Railway	D&B	NATM	Collapse		11
2002	Autoroute A86 –Rueil, France		TBM	Fire		11
2003	Shanghai Metro		Freezing	Collapse	Faulty workmanship	69
2004	Singapore Metro, Singapore	D&B	C & C	Collapse	Faulty design/ work	t.b.a.
2005	Barcelona Metro, Spain		NATM	Collapse		t.b.a.
2005	Lausanne Metro, Switzerland			Collapse		t.b.a.
2005	Lane Cove Tunnel, Sydney		NATM	Collapse		t.b.a.
2005	Kaohsiung Metro, Taipei		TBM	Collapse	Faulty workmanship	t.b.a.
	18 major losses				TOTAL	>570

On 2006, at the 39th annual conference of the International Association of engineering insurers was presented the following table in which is relevant the number of collapses produced during tunnel construction in ten years by the two more developed techniques, NATM and TBM. Danger and risk are directly associated to poor performance on design, construction and supervision and nothing else.

Excavation sequences are not relevant and should be chosen by contractor. This rule is in direct opposition to the requirements of SEM which are the following:

- The excavated cross section should always be an ovoid shape.
- Installation of immediate and continuous smooth support around its perimeter (and, if required, smooth support at the face) is a significant factor in minimizing initial movement in the surrounding ground.

- It is also essential to structurally close the supporting ring (shotcrete) as quickly as possible within one tunnel diameter of the advancing excavation face.

The 3-dimensional stress redistribution around the tunnel depends on geometry and time. This must be carefully considered particularly where multiple openings are planned. It will govern the progress of tunneling with respect to stress redistribution interaction and the hardening of the shotcrete support, consequently it is not an issue to be left in the contractor's hands but to be coordinated among the parties in order to achieve the fastest excavation sequence but also the more stable and safe (Figure 3).

Bench and invert excavation is not relevant for ground movements. It is very common to see how in tunnels is excavated the arch heading, from portal to portal, with carefully supervision and monitoring of ground deformations but by the opposite, bench excavation is done based in large sections without support, late convergence measurements and leaving the invert closure as a final activity once the full bench has been excavated. A not negligible amount of tunnel collapses occurs during the bench and invert excavation. This is a fact that must be considered together with the previous rules (Figure 4).

The poorer the ground, the bigger elephant foot must be designed to support the arch. The elephant foot is a tool applied on the design of soft ground tunnels with the aim of support the loads of the arch heading, transferring them to the ground at the spring line level, instead to be transferred to the side walls support, closing into the invert (Figure 5). This solution looks very rational except by the fact that in the case of very poor ground, the bearing capacity of this ground is low and the over excavation for the elephant feet increase the size of the tunnel by more than 15% and to it's square the loads. Contraproductive!

During bench excavation, the loads are concentrated in the base of the elephant foot, the ground is weak and the risk of collapse is very high. On the other hand, if the ground characteristics are better and it is able to support the load, probably the foot is not required. Once again is the author's recommendation to provide a rounded shape avoiding sharp areas which are common stress

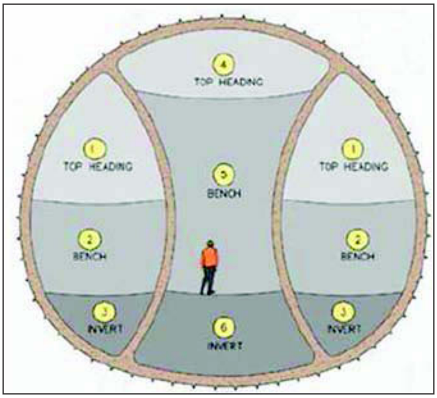

Figure 3. Excavation stages on large sections

Figure 4. Bench excavation in road tunnel

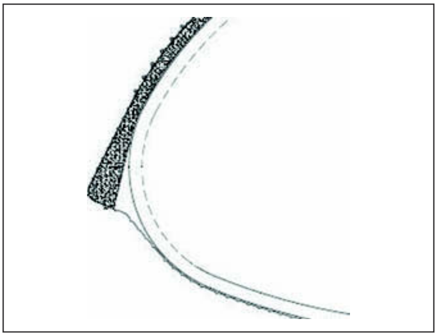

Figure 5. Elephant foot scheme

Figure 6. TH profiles installed in coal mine

concentration zones and consequently, prone to collapse.

Steel ribs are stronger than lattice girders. There is a substantial amount of tunnels in which the poorest ground is supported by heavy H-beams, 240 or even 300, spaced not more than 3 feet and covered by shotcrete or in the case of rock class 3 and 4 by TH-beams 29 or 36 with similar spacing and coverage. Steel ribs have been a traditional support on the mining industry, where shotcrete was not applied and the full loads were supported by those strong steel profiles (Figure 6).

Figure 7. Detail of connection for TH profiles

In order to be efficient, the beam is in touch with the ground by means of wood logs or boulders along the whole perimeter avoiding in this way punctual loads and consequently flexural stresses. In the case of TH profiles, they were originally designed (for the mining industry) to be able to slide one over the other and in this way allow the ground deformation without collapsing the tunnel. Once the deformation overcomes the required template for mining operations, the gallery is enlarged and new TH profiles are installed. In civil works, the TH profile is curved in the opposite side in order to keep the opening accessible for the sprayed shotcrete (Figure 7).

But in civil works, also the excavation is done, large enough to avoid the contact between the ground and the steel rib and only the shotcrete will connect the ground and the rib in order to be effective. This circumstance has the following disadvantages and concerns:

- Time consuming in overexcavation
- Time consuming in heavy rib erection and connecting the components close to the face not only reduces the progress but also is unsafe.
- Shotcrete discontinuity, full thickness between ribs and not more than one inch on rib station.

The effectiveness of this kind of support is very weak since the ribs do not assist in the tunnel reinforcement but only when the shotcrete achieve enough strength and in most of the cases, the shotcrete can bear the load by itself. In summary it is an expensive use of the mining technique. Lattice girders provide a real structural reinforcement to the shotcrete increasing its performances and giving continuity to the tunnel support.

Figure 8. Reinforced steel cage for precast concrete segment

Initial support is temporary and final lining must bear the full overburden. Again it is a merely opinion which differs from Client to Client. The fact is that the shotcrete ring remains as the MAIN support for most of the tunnels over its entire lifetime with very few exceptions. By the opposite, final lining bear the loads in those cases in which the primary lining is weak in design or construction performances.

Precast segments must be designed heavily steel reinforced just for handling. On TBM tunnels the lining is done by precast concrete segments, with the exception of gripper TBMs. These heavy pieces of concrete are calculated in accordance with the expected ground loads but not only. It is very important to analyze the forces applied to them during the demoulding, (early strength), storage and handling in the factory as well as the forces applied by the hydraulic jacks of the TBM during the excavation in single shield machines or during the regripping in double shield equipment. In most of the cases the higher restrictions comes from the handling in the factory. This is a fact that doesn't make any sense from the engineering point of view. It is not logical to include in the lining a high amount of reinforced steel where is not required and goes against the lifetime of the structure (Figure 8).

Actually, it is noticeable that in some tunnel projects, the designer solution is based in fiber reinforced concrete, mostly steel but also non metallic fibers. This is a new field that we encourage the designer to deeply investigate since it is a logical way to design, just to support the loads applied during the operation of the structure. Contractors must think how to elaborate the segments and designers must analyze the number of segments per ring in order to build them without this temporary requirement of reinforcement. Also we encourage the owner to be open to this new solution since will provide better and longer lifetime without any decrease on the final lining capabilities.

The more calculations and drawings the safer and better is the design. The insurance industry calculates the risk according to the spent man-hours on design. Based in this approach, design cost for tunnels and underground structures grows to 10% of the construction cost in some countries, but quantity can never replace quality and at the end the design cost is much higher than the strictly required. Tax payers are, again, the harmed ones.

Large bore is cheaper than twin bore. There is not a simple answer for this affirmation. It depends not on tunneling conditions but legal and operational ones. In the case of Metro tunnels the twin bores, single track, excavation diameters are on the range of 20 to 23 feet and use to be connected by cross passages every thousand feet. The required envelope depends on cars size, walkway dimensions and power source (third rail or catenaries), among others.

To join both tracks in one single bore must be taken into account the local, state or country regulations mainly related to safety issues. Consequently must be considered the exigency of dividing wall, pressurized passages, walkways dimensions and number, evacuation routes, etc…These "local" circumstances give us, at the end, a single bore in the range of 30 to 40 feet for the excavation diameter. It is obvious that the cost of one bore of 40 feet it is much higher than the one of 30s. Figure 9 gives us a construction cost comparison related to the excavation diameter and based in previous experiences. This comparison must be done for similar geological conditions. It is well known that the cost of tunnel excavation is in a very close relationship with the encountered geology.

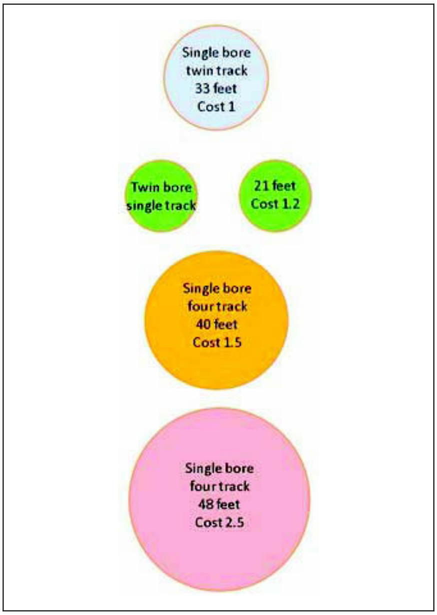

Figure 9. Construction cost vs. tunnel size

Tunnel excavation diameter is independent of the purpose. The question is: Why different designers show different excavation diameter for every single tunnel related to the same network? Again, the inner diameter refers to required envelope for operation but the excavation diameter comes from the inner diameter adding the lining thickness and, in the case of TBM tunnels, the gap related to the TBM design. Assuming that every TBM manufacturer has its own design, steel plates thickness, sandwich concept, tail brushes thickness and location, front to tail shield diameter evolution, etc…lets concentrate in the lining thickness. The precast segments are just concrete and concrete has as much performances as wished. Combination of compress strength and amount of reinforced steel if required can provide similar results independently of the thickness. Consequently the contract must be open to offer different concrete thickness than specified but with similar performances. In this way, contractor can bid lowest prices based in existing TBMs as well as tighter schedules for the benefit of the client and the contractor as well.

Alternate design is not allowed. Dual Design creates competition; it enhances identification with one's own work among Contractors. Conventional methods versus mechanical excavation are different options with pros and cons and both should get a fair chance.

CONSTRUCTION

Automatization is not possible in tunneling. The first EPB was built on 1974 by IHI. It was a 12 feet machine quite different to the current designs. Many things have changed since them, conveyor belts replace muck cars, much higher working pressure can be achieved in the front face and chemical additives have been introduced in to the market in order to deal with all kind of ground conditions. Despite these facts, there is one field which is a pending matter for all of us: automatization of tunnel excavation (Figure 10).

The largest TBM is currently the EPB for the Alaskan Way replacement project in Seattle. The machine is fitted with automatic erector and the cutting tools can be replaced in a free air instead the hyperbaric interventions, slow and dangerous. But still a substantial amount of skill people is required as a crew to operate the giant. Most of the industries, the fabrication process is automatic and robots are in charge of the most difficult and precise operations.

Figure 10. IHI first EPB

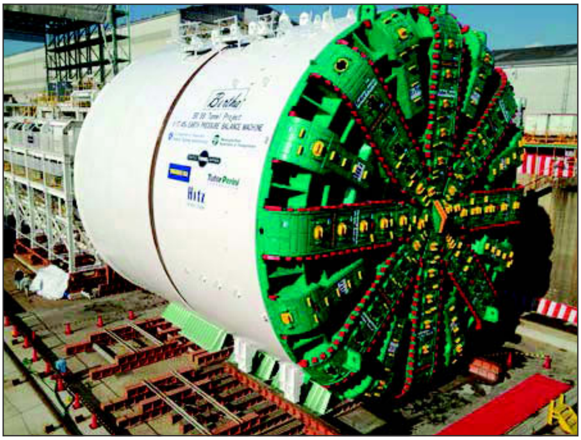

Figure 11. World largest TBM 57' 6"

Excavation and lining are much simpler operations that assembly a car. This approach is every time more and more common in Japan but not in western countries and these expensive machines are human behavior dependant (Figure 11).

A similar approach occurs in conventional mined tunnels. Drilling equipment and loader manufacturers use to have to different versions of the same machine in accordance of their utilization. For mining industry are robotized and for tunneling are operated for one skill labor or more (Unions conditions). It is time to concentrate efforts, i.e., R&D projects, to provide new equipment which minimizes the amount of personnel inside the tunnel where the working conditions are not as comfortable as they are in open air. The reduction of the human behavior impact will goes on the benefit of the construction quality and performance.

TBMs are for long tunnels and conventional for short tunnels. Construction method for tunneling is related mostly to the geology but not only. There are a lot of factors that can determine the final selection like:

- Number of intermediate adits. One long tunnel can be converted in several short tunnels just having intermediate access to the running tunnel and from where the excavation can be done simultaneously. A 12 miles tunnel, logically excavated by TBM due to the achievable progress rates can be excavated faster and more economical if we have the chance to get five intermediate access with a reasonable length in order to operate simultaneously in 5 × 2 + 2 portals=12 front faces of one mile each. In sound rock a monthly progress of 176 yards is achievable by conventional means, thus in ten months the tunnel is completed, much early than the TBMs delivery to the site.

- Environmental restrictions. Tunnels are built in the most varied scenarios from congested cities to protected natural parks. In this last case there are a number of restrictions that can eliminate the chance to apply the most technical way to approach the excavation. Blasting restrictions is one of the most common limitations but not limited to it. In this case of hard rock, TBM is an option despite the length of the tunnel. Also in very soft ground with environmental restrictions, like is the case of Miami port tunnel, a 0.5 miles tunnel must be built by a TBM.
- Urban restrictions. These restrictions use to be associated to noise and vibrations, consequently limited daily working time. Any selected method must fulfill the requirements independently of the tunnel length.
- Portal access. In linear projects such railway, road or water transfer ones, tunnel portal has limited space and in some cases is followed by a viaduct which limit the chances to operate a TBM.
- Import taxes. In some countries the import taxes for civil works equipment is so high that this condition can eliminate the option to excavate the tunnel with TBMs, considering that the purchase cost of a TBM is in a ratio of 10:1 with conventional equipment.

Upper tunnel must be excavated before the lowest one. The reasoning to follow this rule is based in the wrong assumption that once the arch above the upper tunnel is secured, another tunnel can be excavated below the first one easily. Indeed is the opposite, if the lowest tunnel is excavated in advance, the upper one will be excavated in a decompress ground but without damaging the previous tunnel meanwhile if the upper tunnel is excavated before, the excavation of the lower tunnel will create a substantial influence in its stability.

Waterproofing can be sprayed on in tunnels. This is not possible for each of the three physical reasons:

1. A tunnel remains under permanent strain changes caused by Earth crust tide, tectonic forces, temperature and moisture changes, earthquakes, etc.. This leads to unpredictable location and occurrences of cracks in the lining which can cause strain factors of theoretically infinite for a bounded layer over it.
2. Inflows of water, but also excessive moisture is in a high probability also unpredictable over short and long time and hinder the bound and curing of any sprayed on material.
3. The physical conditions during construction compromise the necessary application environment for a continuous reliable layer.

Pilot tunnel must be on the top of the tunnel section. The construction of a pilot tunnel as a way to investigate the ground ahead the front face is common practice for long tunnels and where the geotechnical information is not enough precise to allow the contractor to excavate the tunnel at once. Sometimes this pilot gallery is located at the top of the tunnel section and the enlargement will require the destruction of pilot tunnel support, disrupting the excavation process (Figure 12). If for some reason the pilot has to be in the tunnel section, it should be located close to the invert, but locating this pilot gallery parallel to the running tunnel has multiple advantages like:

Figure 12. Pilot and evacuation gallery in Dos Valires (Andorra)

- Excavation can be done simultaneously to the running tunnel and ahead
- Provides valuable info to be applicable in the running tunnel
- Connected by cross passages can be utilized as evacuation route during excavation and operation
- Different working methods can be applied to each tunnel as they have not interference

Ventilation time must take 30 minutes. It is common practice in drill and blast tunnels to allow half an hour as an standard time to evacuate the smoke and noxious gases produced by the blasting at the front face, independent from blasting material and amount. Every kind of explosive has different composition and generate different amount of noxious gases, the so called fume factor. Also, in accordance with the ground conditions will differ the pull length and the explosive load. Consequently the ventilation time must be optimized in accordance with this facts and try to remove those gases in the minimal time. For this reason a well selected fan in terms of power and the size of the ventilation pipe are relevant as the way to do this activity properly.

Innovation, but not in my project. In general, the tunneling community has considerable inertia on their assumptions. The usual Lessons learned are a good way to avoid mistakes that occurs in previous projects despite it is also a kind of conservative approach. Innovation means to do something for the first time by ourselves and it is not an invention. There is a lot of money involved in a tunnel project and nobody wants to take the risk of apply a new invention on it, but an innovation is something that has been already proved by others or in other fields and consequently minimizing the associated risk. A good sample is the automatization in mining equipment, mentioned above. The technology already exists and has been well proven on mines, then why not to apply in tunneling as well? Open minds and innovative approach are the key for the progress in tunneling.

SUPERVISION

The third pillar to succeed in a tunnel project is the supervision, not only the quality control and the deformation monitoring but the verification that the whole system is running properly. Site supervision quality which meets the highest possible international standards must be emphasized for the construction. To assure that this goal is achieved, the responsible designer must participate in supervision of his specified techniques. Destructive competition between third party field supervision and the designer should be eliminated; it only feeds lawyers and ruins the quality of the end product.

Collapses / Downfalls happen mainly on night shifts and/or on weekends. In most cases, they are a management problem. If specified and priced in the bid documents

Figure 13. TBM tunnel collapse produced by over excavation

their occurrence reduces substantially! Collapses are rarely a sole result of unexpected geological conditions (Figure 13).

Competence and Risk are Inverse Proportional. How to identify competence? Track record? There is a direct relationship between the competence of the Designer, Execution Group and the effort exhausted by the Client in identifying and putting his trust in his group of choice (Venturato)…and the Risk? Risk can be controlled by asking the right questions and accepting the true answers. The insurance industry has suffered huge losses by ignoring these simple facts. To do that, it is not necessary that the expert must be bold or gray hair, just competent. Contract Flexibility using unit prices and appropriate specs can minimize/avoid disputes and gridlocks on site. It leads to the most cost effective quality end product. The risk must not stay 100% on the contractor side. Also with this contract method are not required Board of experts to revise the revision of the revision, etc… It's crystal clear.

ENVIRONMENTAL

Environmental issues are not a pillar of the project success but usually a great constrain. Preservation of species cost a fortune and delays the starting of the new project for months and months or even years. Bird nest close to the tunnel portal, a specific kind of butterfly in the surroundings or an exclusive family of frogs are able to delay or cancel the project. Blasting use to be forbidden in restricted areas just to do not disturb the animal life but how can be measured?

The question is: Who must does the balance between the sustainable growth and the animal life disturbance? It is reasonable to spend more than 20 years in the excavation of a 5 miles twin tunnel just to limit the water inflow during construction in an evergreen rural area? How much it cost this solution to the tax payers? Are the citizens waiting for the new infrastructure for years less important than temporarily relocate some cows (for instance)? Why cannot be solved the problem in a most logical way supplying fresh water by tanks to the farmers or pumping the water inflow in the tunnel to the surface again, after treatment, during the limited construction period? This environmental problem is limited to the construction period since tunnel can be designed drain or tanked, so, the problem is limited in time. Or, can a frog be so disturbed by blasting that a half mile tunnel in hard rock which can be excavated in six months must be done by drill and split or similar methods in two and a half years? The reader can estimate easily the delta between both solutions.

Finally, it is reasonable to modify the tunnel lining during construction, increasing the thickness and consequently limiting the train speed from 186 to 50 miles/ hour during the whole tunnel lifetime just to limit the water inflow (designed drain tunnel) due to the complains of the closer village located in the slope of the range? (Namely The Water range).

CONCLUSIONS

The author's intention is to call the attention of the tunneling community about current practices that are far from the optimum solutions in terms of cost and schedule and also in some cases are disturbing to the neighborhood.

Our aim is to encourage our colleagues to develop sustainable solutions; tunneling technology allows doing it, during the design and construction stages to provide to our clients what they need but minimizing the disturbance to third parties.

Noise and vibration during construction as well as utilities relocation, business loss and disruptions can be avoided with the existing tunneling techniques. Let's apply them properly and logically.

As captioned above the success is strongly related to the three pillars: design, construction and supervision, as simple and as logical as that. Proper design, avoiding the wrong rules, right construction in accordance with the specified quality and adequate supervision based in transparent contractual clauses are the key.

Last but not least, environmental issues are really important but must be balance with the final cost to the citizens. Again tunnel techniques can contribute to it.

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PRACTICAL METHOD FOR PREDICTING DIAPHRAGM WALL SHAFT CONSTRUCTION-INDUCED SETTLEMENT CASE HISTORY—THE BLUE PLAINS TUNNEL PROJECT

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ABSTRACT

Over the years, simplified empirical methods for predicting ground movement induced by tunneling i have been developed. However, for prediction of ground movement due to shaft construction, there is very little published literature. Methods for predicting ground movement due to shaft construction using simplified methods are not only limited but also not reliable in many cases. While this type of movement is of less concern than that related to tunneling, it still remains an important because of the impacts associated with shafts close to structures in urban areas. Determining ground movements due to shaft excavation presents challenges due to factors such as prevailing ground and groundwater conditions, changes in groundwater levels, support system type and stiffness , and construction means and methods. This paper presents a case history of construction impact assessment due to diaphragm wall shaft construction for the Blue Plains Tunnel (BPT) project.

INTRODUCTION

One of the biggest challenges on the BPT project is the protection of numerous existing structures at the five shaft locations and along the tunnel alignment. The existing structures include buildings, facilities, underground structures, and various utilities. Some of the impacted structures are of particular concern due to their importance to the system operation and/or age. The contract document identified the majority of these structures and classified them under three categories based on impact, age, and importance. These categories are identified in the contract document as Tier 1, Tier 2a, and Tier 2b. Structures categorized as Tier 1 are the most critical structures and are required to have mitigation measures in place unless proven otherwise by 3D geomechanical and structural analysis.

This paper presents a case history of the construction impact assessment for the existing structures at the main launching shaft site, Blue Plains Tunnel Dewatering Shaft (BPT-DS), and Blue Plains Tunnel Screening Shaft (BPT-SS). At the time of writing this paper, the construction of the BPT-DS and BPT-SS are ongoing. The following sections provide description of the project, analysis performed, and the methodology for assessing the impact on existing structures due to shaft construction activities.

PROJECT BACKGROUND AND DESCRIPTION

The BPT project is a component of a larger scheme, called Long Term Control Plan (LTCP), to control combined sewer overflows (CSOs) to the District of Columbia's waterways (Anacostia and Potomac Rivers). The LTCP is designed to meet the CSO control objectives of DC Water and to meet water quality standards in the District of Columbia. The Tunnel is to be constructed from the Blue Plains Advanced Waste Water Treatment Plant (BPAWWTP) to the existing DC Water's Main Pumping Station (MPS) as shown on Figure 1.

The BPT project consists of:

- *Blue Plains Tunnel (BPT)*—approximately 24,000 ft. long, 23 ft. internal diameter (ID) tunnel.
- *Blue Plains Tunnel Screening Shaft (BPT-SS)—A screening* shaft for use in mining the BPT, located on BPAWWTP site.
- *Blue Plains Tunnel Dewatering Shaft (BPT-DS)*—a dewatering pumping station shaft, located on the BPAWWTP site.
- *Bolling Air Force Base Drop Shaft BAFB-DS)*—an overflow/drop shaft located within the Joint Base Anacostia Bolling (JBAB) site.
- *Poplar Point Drop/Junction Shaft (PP-JS)*—a combination drop/junction shaft on District of Columbia government land.
- *Surge Chamber and Approach Channel at PP-JS*—the approach channel will connect the future Main Outfall Sewer Diversion Chamber (MOS-DC). The MOS-DC will be built over the existing modified twin sewers (under different contract) for directing flow from the West Influent Sewer and East Influent Sewer to PP-JS. A surge chamber will be constructed where the approach channel connects with the vortex in PP-JS to facilitate handling the flow.
- *DC Water's Main Pumping Station Drop Shaft (MPS-DS)*—a drop shaft at MPS near 2nd Street and Tingey Street SE. This shaft will be used to convey diversions from CSO 13 and 14 (Division I) as well as CSO 9, 11A and 12 diversion chambers.

Figure 1. BPT site plan

BPAWWTP SHAFTS CONSTRUCTION

The two shafts at BPAWWTP were built directly adjacent to each other (in a figure eight arrangement) using the diaphragm wall installation process to safely install the main structural support. The design uses a dual-cell slurry wall shaft configuration that required a 5-ft-thick diaphragm wall (D-wall), except at the mid-wall where the D-wall thickness varies from 8 feet to 11feet., to enable the excavation of the shaft without the need for installation of the CIP liner as excavation proceeds. The BPT-DS has a required minimum internal diameter of 132 feet and a final planned depth of 166 feet (El. –149) to the top of the base slab. The BPT-SS has a minimum internal diameter of 76 feet with a depth of 150.5 feet (El. –133.5) to the top of the base slab.

The planned construction sequence of the BPT-DS and BBT-SS is as follows:

1. Installation of Cutoff wall panels for ground and groundwater control during at launching of TBM;
2. Installation of D-wall panels for both BPT-DS and BPT-SS
3. BPT-DS and BPT-SS excavation to the temporary slab at an approximate elevation of –120.5 and –135.83 ft. respectively
4. An opening in the wall separating the BPT-DS from the BPT-SS will be created to allow for the TBM trailing gear assembly and launch
5. Once the TBM is launched, the opening in the wall separating the BPT-DS from the BPT-SS will be reduced to 16 feet and a temporary bulkhead will be installed
6. TBM operations will proceed until the completion of the tunnel construction while BPT-DS excavation simultaneously continues as described below
7. Simultaneously with the TBM operations in BPT-SS, activate a water depressurization system targeted at the lower predominantly sandy layer (Potomac G-3A and G-3B groups)
8. Simultaneously with the TBM operations within the BPT-SS and once the water depressurization system is activated, BPT-DS excavation will resume to the bottom of the base at an approximate depth of 193 ft. (El. –174)
9. BPT-DS Waterproofing system and base slab installation will then take place
10. Deactivation of the water depressurization system once floatation/heave resistance is provided by the base slab and shaft structure as necessary
11. Waterproofing system installation and construction of the final lining within the BPT-DS will then take place
12. Once TBM operations are completed and equipment removed from the BPT-SS, BPT-SS excavation will resume to the bottom of the base slab at an approximate depth of 173 ft. (El. –155);
13. BPT-SS Base slab installation will then take place
14. Construction of the BPT-SS final lining

GROUND CONDITIONS

Geology Along the Tunnel Alignment

The BPT project site is located within the Atlantic Coastal Plain physiographic province. The Atlantic Coastal Plain comprises a wide belt of sedimentary deposits overlying crystalline bedrock. The natural deposits that underlie this region consist of Cretaceous-age formations, which are the oldest Coastal Plain sediments. Geologically recent alluvium is often present in the vicinity of historic and extant waterways. In some areas, the ground surface, as well as the course of tributary streams, has been altered by placement of artificial fills.

The Cretaceous-age sediments, known collectively in this setting as the Potomac Group, consist of dense sands and gravels with variable fractions of fines, and very stiff to hard over consolidated clays and silts. Although the clays and silts are typically very hard, the presence of slickensides (previous shear surfaces) often reduces the shear strength of the soil mass. Man-made fills overlie the natural materials in many portions of the site. These fills were placed principally to develop various areas of the project site, The fills in such areas typically consist of soils that were locally available at the time of placement, and as such, are sometimes difficult to differentiate from undisturbed natural soils.

Stratigraphy at BPAWWTP

The Geotechnical Baseline Report (GBR) indicated that the site is covered with fill underlain by Alluvium deposits. The alluvium is underlain by Potomac Group soils. Below is a summary of each soil formation. The information presented herein is based on the geotechnical baseline presented in the GBR.

- *Fill*—Fill within the site consists of all types of soils including locally derived soils and decomposed rock. The fill included construction debris, wood fragments, concrete fragments, cinder, and trash in some areas. The more recently placed fill within the demolished underground structures area is anticipated to be free of obstructions and to consist of excavated and locally derived materials. Fill is generally more granular than fine grained and is saturated below groundwater table. The GBR indicated that obstructions such as metal objects, boulders, and boulder-sized concrete fragments may be encountered within the fill materials. Fill material is expected to extend from ground surface to El. +4 to El. –16.
- *Alluvium*—the GBR indicated that Alluvium sand and gravel are expected below the fill to elevations ranging from El. –28 to El. –37. The alluvium sand consists of loose to very dense silty sand. Some lenses of cemented sand were encountered within the alluvium sand. The gravel deposits underlying the sand were described as gravel with sand, with layers and lenses of silty gravel, and silty sand. The alluvial Gravel is very dense.
- *Potomac Group*—the soil profile under the alluvium deposits primarily consists of high plasticity clay and silt, CH and MH (G1 of the Patapsco/Arundel Formation) with small interbedded finer materials to approximate elevations ranging from El. –203 to El. –218. The G1 soils are underlain by the Patuxent Formation consisting of predominantly granular materials (sand and gravel) with varying amounts of fines. These materials were encountered to the termination depth of the soil borings during the subsurface investigation at approximately elevation El. –335.

Groundwater Conditions

For the Blue Plains shaft site, groundwater is anticipated at elevations ranging from El. –9 to El. –4 in the fill and alluvium layers. The lower aquifer water head within the Patuxent Formation of the Potomac Group ranged from approximately El. –25 to El. –23.

EXSITING STRUCTURES AT BPAWWTP

Several structures exist at BPAWWTP site including facility buildings, 10 gravity thickener tanks (GTT), and various underground structures and utilities. Three of the existing structures are classified as Tier 1 structures in the contract documents. These structures are essential for DC Water operations and cannot be taken out of service during construction. The following paragraphs provide description of the Tier 1 structures as well as other structures present on the BPAWWTP.

Semi-Elliptical Overflow By-Pass (Tier 1 Structure)

According to 1935 contract drawings for Unit 2 (Project No. 9200), the semi-elliptical overflow by-pass has a reinforced concrete semi-elliptical roof with a concave reinforced concrete bottom. It is assumed that the conduit was built in 1935. The conduit is 8 feet wide at the base and 8 feet high at the centre with a lining thickness ranging from 8 inch to 12 inch. The invert elevations range from El. –3.36 to El. –4.84 with a 1% grade. No other information was available regarding conduit rehabilitation/maintenance. An attempt was made to inspect the conduit. However, it was deemed impossible to inspect due to the large volume of muck present.

Gravity Thickener Tanks No. 2 & 4 (Tier 1 Structures)

Gravity Thickener Tanks No. 2 and 4 are assumed to have been constructed in 1958. The tanks are connected to an underground service gallery, according to the 1958 as-built drawings. The two tanks and the underground gallery connecting them are made of reinforced concrete. The structures are founded on wooden piles. The pile tip elevations were estimated at El. –51 to –55. The internal diameter of the tanks is 65 feet with 1 foot-thick walls. The foundation levels for the tanks range from El. –6.25 at the center of the tank and the connecting gallery to El. +7.75 at the outer edges of the four-tank unit. The top of the tank structural wall is at El. +21.0. The tanks are located at 76 feet center to center.

A pre-construction condition survey was undertaken for the GTT's No. 2 & 4. Results from the pre-construction condition survey indicated that the structure exhibits a large number of very fine to fine cracks on the outside surfaces. These cracks appear to be due to thermal expansion and contraction. Thin layers of repair mortar have been applied on the outside faces of some of the Tanks. No major structural related damage was observed.

OTHER STRUCTURES

In addition to Tier 1 structures listed above, several other important structures are present on site. These include the following:

- *Laboratory Building*—two-story steel frame building with basement
- *Degritting and Grinding Building*—two-story steel frame building with a basement connected to an underground gallery (tunnel)
- *GT Tanks No. 1, 3, 5 through 10, and Thickener Control Roo*m—these tanks are similar to GTT's 2 & 4 but constructed at different times. Some of the tanks are founded on concrete piles
- *Underground Gallery Tunnel*
- *36″ Thickener Influent*—a 36 inch concrete pipe
- *BPAWWTP Seawall*—consisting of sheetpile walls
- *Electrical Ductbank*—an electrical ductbank hosting cables
- *24 inch Storm pipe*—a 24 inch concrete pipe

ANALYSIS METHODOLOGY

The analysis methodology included established procedures to provide a consistent basis for the assessment of damage risk to existing buildings, structures, and utilities.

The Construction Industry Research and Information Association (CIRIA) Report "*Building Response to Tunneling, Volume 1: Projects and Methods Case Studies from the Construction of the Jubilee Line Extension, London 2001*," recommends a staged assessment approach to be carried out in accordance with the methods described by Burland (2001).

The purpose of the staged assessment is to predict the possibility of damage to existing structures and to select the appropriate 'action' and 'maximum' levels for ground movement monitoring and any requirements for protective measures. The staged analysis approach included the following:

- *Stage 1 Assessment* is the first step to delineate the area where existing structures are potentially impacted.
- *Stage 2 Assessment* is the second phase of analysis using empirical methods to study the potential impact on the existing structure, and primarily used as a screening tool.
- *Stage 3 Assessment* includes further numerical analysis of structures with the potential for damage.

This approach also includes investigation of potential mitigation measures as needed. The following paragraphs explain our proposed staged approach. Figure 2 provides a graphical presentation of our staged approach.

Stage 1 Assessment

Stage 1 assessment includes a preliminary assessment in which the surface settlement contours for "Green field" conditions are determined using empirical analytical methods such as Attewell (1982) for settlement due to tunnelling and Empirical methods presented in the CIRIA Report No. 580, "*Embedded Retaining Walls: Guidance for Economic Design*." Based on the settlements established from these methods, settlement contours were developed in order to determine the "zone of influence." Based on past experience, it was agreed that structures falling outside 0.2 inches (5mm) settlement contour line are not impacted (i.e., 0.2 in. contour line is the limit for the "zone of influence"). Figure 3 presents settlement contours at BPAWWTP shaft site.

Stage 2 Assessment

Preliminary Analysis (Empirical Analysis)

The second stage assessment was conducted for structures identified to be in the zone of influence (based on Stage 1 Assessment). The second stage assessment makes use of the research by Burland et al. (1977) and the general methodology of Mair et al. (1996) that considers the building as an equivalent beam. The possible degree of damage is assessed according to the induced tensile strain in the structure. Table 1 presents the Building Damage Classification (After Burland et al., 1977 and Boscardin and Cording, 1989).

The structures adjacent to the proposed alignment were classified in terms of potential risk of damage based on categories proposed by Burland et al. (1977). The categories included: "*Negligible*," "*Very Slight*," "*Slight*," "*Moderate*," "*Severe*," and "*Very Severe*." These categories depend on typical damage of typical masonry structures along with typical values of maximum settlement and slope for damage risk assessment proposed by Rankin (1988). Boscardin and Cording (1989) showed that these categories of damage are related to the magnitude of the maximum tensile strain induced in the structure.

Buildings or structures possessing structural continuity such as those of steel and concrete frame or box construction are less likely to suffer damage than masonry or brick buildings, therefore the above classifications are considered conservative.

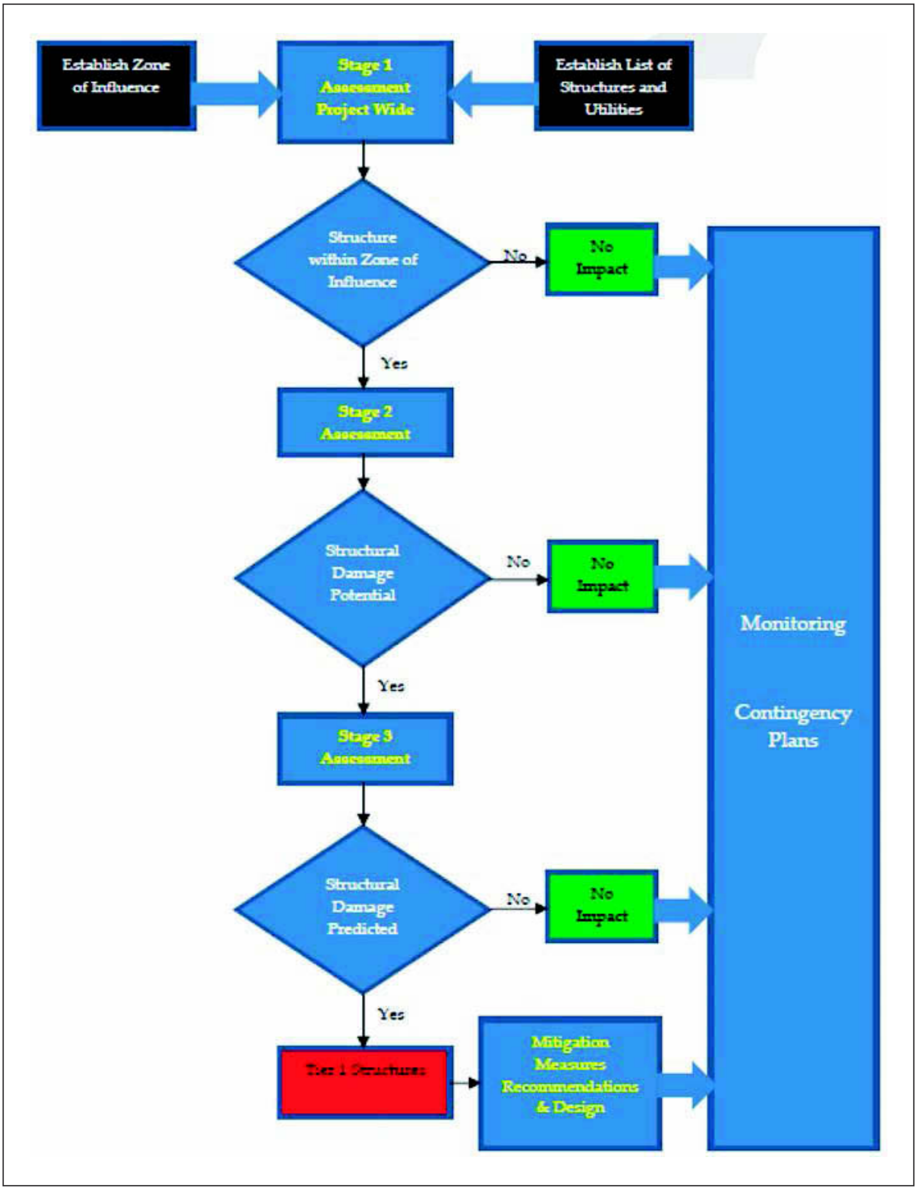

Figure 2. Protection of structures staged analysis approach flowchart

However, specific construction details of the existing structures should be carefully reviewed.

The presence of piles may be beneficial in limiting damage risk as piles cause some reduction to the surface settlement profile and zone of influence. According to Jacobsz et al. (2001) at volume losses at or less than 1.0% the observed pile settlements were less or very similar to the surface settlement for green field conditions. Pile capacity reduction and elongation due to tunneling was investigated using the empirical method described by Poulos & Deng (2004).

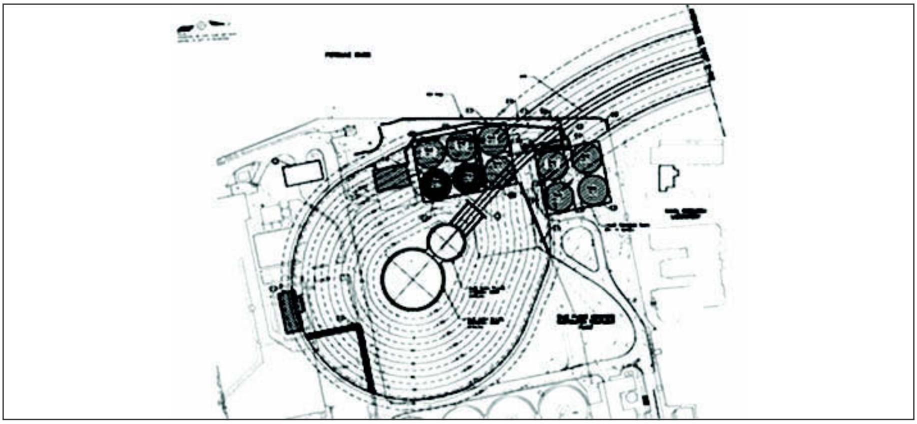

Figure 3. Settlement contours at BPAWWTP shaft site

A detailed evaluation (Stage 3 Assessment) was carried out on buildings that as a result of the Stage 1 or Stage 2 assessment, were classified as being at a damage risk of "*Slight*" or worse, and those structures whose foundations were likely to be adversely affected by the construction activities. In addition, all Tier 1 structures were required to undergo Stage 3 Assessment, as required by the contract documents.

Stage 3 Assessment

The Stage 3 Assessment was a refinement of the Stage 2 assessment and involved a more detailed evaluation of the structure considered. The evaluation considered particular features of the structure and excavation scheme, including the excavation sequence, dewatering, existing foundation type and depth, structural rigidity and continuity, dimension of the structure, and soil-structure interaction.

The analysis was performed by undertaking three dimensional (3-D) geomechnical numerical analysis models using PLAXIS 3D software packages (a finite element based software). The program was used to model ground behavior, simulating ground response, construction method, support type, and excavation sequence.

Deformations, stresses, strains of existing structures were determined and impact on the structures assessed in order to determine whether mitigation measures were needed. Structures likely to incur damage are subjected to evaluation of different mitigation measures to determine the best viable mitigation alternative.

STAGE 3 ASSESSMENT FOR GTT 2, GTT4, AND SEMI-ELLIPTICAL OVERFLOW BY-PASS

The numerical model developed for the BPAWWTP site was developed using Plaxis 3D. This software uses finite element analytical methods to model ground deformation due to construction activities. It is widely used in the industry with a track record of reliable results and includes some structural capabilities which allow the modeling of underground structure and foundation response to ground movements. In order to create a model that would produce representative results, a philosophy of investigating the most critical structure was adopted. This approach produces an efficient and practical model while keeping the complexity of the analysis to manageable levels.

GT Tank 2 and GT Tank 4 were both designated as Tier 1 structures. It was very difficult to judge which structure was more critical. In order to determine the most

Table 1. Building damage classification

Building Damage Classification (After Burland et al., 1977 and Boscardin and Cording, 1989)					Approximately Equivalent Ground Settlements and Slopes (after Rankin 1988)	
Risk Category	Description of Degree of Damage	Description of Typical Damage and Likely Forms of Repair for typical Masonry Buildings	Approx. Crack Width (mm)	Max Tensile Strain (%)	Maximum Slope of Ground	Max Settlement of Building (mm)
0	Negligible	Hairline Cracks		Less than 0.05		
1	Very Slight	Fine cracks easily treated during normal redecoration. Perhaps isolated slight fracture in building. Cracks in exterior brickwork visible upon close inspection.	0.1 to 1	0.05 to 0.075	Less than 1:500	Less than 10
2	Slight	Cracks easily filled. Redecoration probably required. Several slight fractures inside building. Exterior cracks visible: some repainting may be required for weather-tightness. Doors and windows may stick slightly.	1 to 5	0.075 to 0.15	1:500 to 1:200	10 to 50
3	Moderate	Cracks may require cutting out and patching. Recurrent cracks can be masked by suitable linings. Tuck pointing and possibly replacement of a small amount of exterior brickwork may be required. Doors and windows sticking. Utility services may be interrupted. Weather tightness often impaired.	5 to 15 or a number of cracks Greater than 3	0.15 to 0.3	1:200 to 1:50	50 to 75
4	Severe	Extensive repair involving removal and replacement of sections of walls, especially over doors and windows required. Windows and door frames distorted. Floor slopes noticeably. Walls lean or bulge noticeably. Some loss of bearing in beams. Utility services disrupted.	15 to 25 but also depends on number of cracks	Greater than 0.3	1:200 to 1:50	Greater than 75
5	Very Severe	Major repair required involving partial or complete reconstruction. Beams lose bearing; walls lean badly and require shoring. Windows broken by distortion. Danger of instability.	Usually greater than 25 but depends on number of cracks		Greater than 1:50	Greater than 75

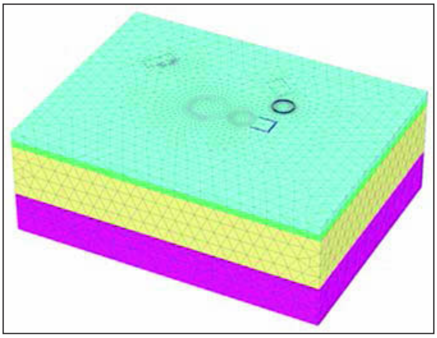

Figure 4. Plaxis 3D model mesh

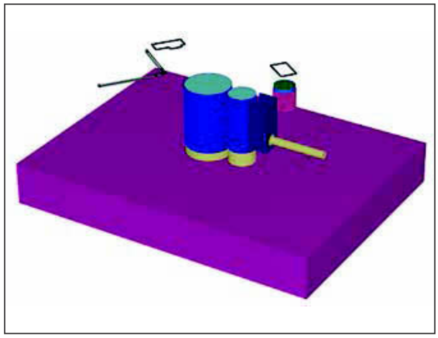

Figure 5. Plaxis 3D proposed and existing structures layout

critical tank structure, a green field model with the locations of both tanks was developed. No existing structures were included in this model (Model 1). Based on total and differential ground defamation results in three directions at the tank locations, it was determined that GT Tank 2 is the most critical structure. As for the semi-elliptical overflow by-pass, the green field ground movements were used to analyze the construction impact in lieu of including the structure in the Plaxis 3D model. This decision was made in order to simplify the Plaxis 3D model. A separate structural analysis was performed using the extracted green field ground movements.

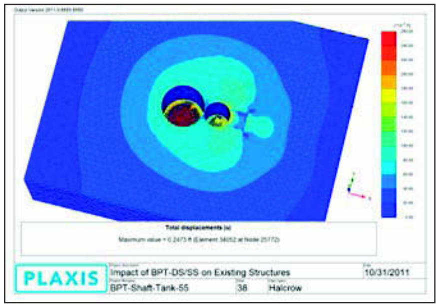

Figure 6. Plaxis 3D total greenfield displacement results at the end of construction

Another model (Model 2) was developed for GTT No. 2 with the GTT structure modeled in order to determine the impact on the structure. Structural checks were carried out to determine the capacities of the structural cross sections and the piles. Figures 4 through 8 present the model geometry and surface settlement.

EVALUATION

Stage 2 Assessment indicated that all structures within the zone of influence at BPAWWTP falls within Risk Category 1 or Risk Category 2 as presented in Table 1 (with a "Negligible" or "Very Slight" degree of damage). However, a Stage 3 Assessment was still required for Tier 1 structures for structures classified as Tier 1 structures.

Based on Stage 2 Assessment and Stage 3 Assessment results, it is evident that empirical method yielded more conservative results. For example, the maximum greenfield settlement due to tunneling only was estimated to be 0.61 inches from Stage 2 Assessment using empirical methods while the maximum greenfield settlement obtained from Stage 3 Assessment (numerical modeling) was determined to be 0.13 inches. Please refer to Figures 6 and 7 for settlement trough predicted by numerical analysis and empirical methods.

Structural analyses were performed on Tier 1 structures to determine whether the structural capacities of the structural members including pile foundation were exceeded. Based on the results of the structural analysis it was evident that the internal forces

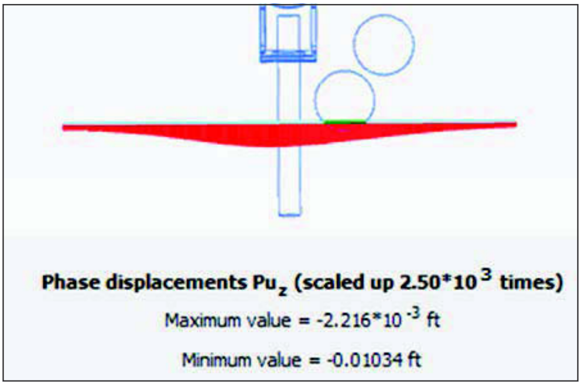

Figure 7. Ground surface greenfield net settlement over tunnel due to tunneling activities from 3D numerical modeling

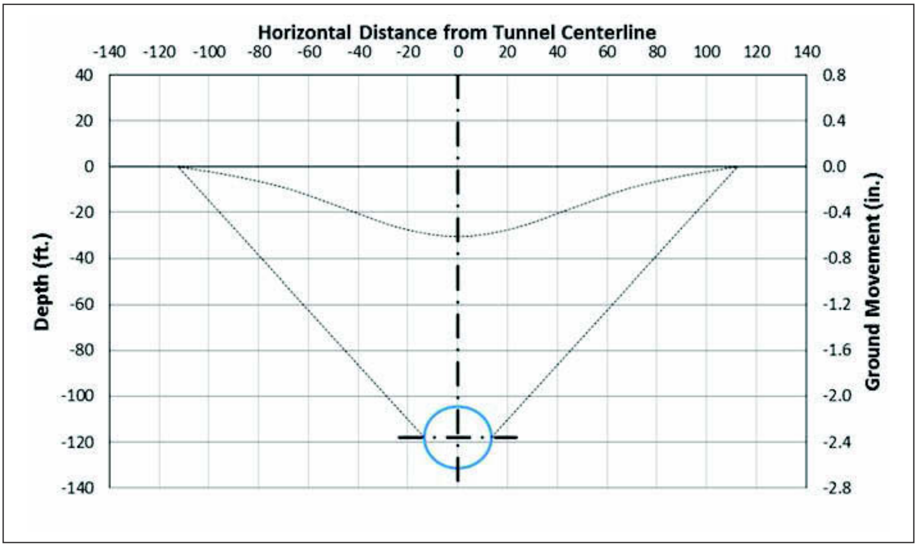

Figure 8. Ground surface greenfield net settlement over tunnel due to tunneling activities from empirical analysis (for comparison)

were within the structural capacities of the existing structural members. Furthermore, the increase in internal forces due to construction activities did not exceed 15% of the original internal forces.

SHAFT INSTRUMENTATION AND MONITORING SYSTEM

The instrumentation and monitoring system is targeted only on structures within the Blue Plains Tunnel (BPT) Project construction Zone of Influence. The layout of the instruments was carefully targeted to provide an efficient and cost effective system.

Existing Structures Instrumentation

For the purpose of monitoring the existing structures, following instruments types were used:

- *Single-Position Borehole Extensometers (SPBX)*—where possible, installed within ten (10) feet of the existing structure and anchored no more than five (5) feet below the base of the structure being monitored (footing, pile cap, lowest floor, etc.)
- *Tilt Meter*—installed to monitor the tilt of critical walls of existing structures
- *Structure Monitoring Points (SMP)*—installed on all sides of existing structures to monitor the vertical movement of the structure
- *Grid Crack Gauge*—grid crack gauge were installed on existing cracks of existing structures and were monitored prior to start of construction and will be monitored until the end of construction in the vicinity of the structure
- *Ground Monitoring Point (GMP)*—ground monitoring point were located at various points to monitor the surface and near surface ground movement in the vertical direction
- *Utility Monitoring Points (UMP)*—Utility monitoring points were installed on all structures that were expected to experience ground movements in excess of the Maximum Level specified by the Construction Impact Assessment Report (CIAR). Utility Monitoring Points were spaced at a maximum of 50 feet apart.

Shaft Instrumentation

To monitor the ground movement in the vicinity of the shaft area associated with excavation, drawdown of groundwater, and construction vibrations the following instruments were installed:

- Inclinometers—to monitor movement of soil for the full depth of excavation
- Structure Monitoring Points (SMP)—to monitor movements of the shaft walls
- Piezometers—to assess the effects on groundwater
- Seismograph—to monitor vibration in the vicinity of the excavation
- Earth Pressure Cell—to monitor total pressure (i.e., combination of effective soil stress and pore water pressure) on the diaphragm walls
- Rebar and Concrete Strain Gauges—designed to measure strain in rebars and concrete

FURTHER EVALUATION

The construction of the BPT-DS and BPT-SS are currently ongoing. The D-wall panels are completed and the shafts excavated to the TBM launch level. Ground deformation data are being collected. However, the data cannot be published until the construction is complete. Preliminary instrumentation readings indicated that actual ground deformations are within the ranges predicted by Stage 3 Assessment. It is recommended that further evaluation of the ground deformation recorded during construction is performed to confirm the results of the Stage 3 Assessment. Further work is needed to refine the Stage 2 Assessment empirical methods as they usually yield more conservative results.

CONCLUSIONS

The staged assessment approach was a useful tool in identifying and analyzing impacted structures associated with the construction of the BPT project. The existing empirical analysis methods for predicting greenfield ground deformations usually yield conservative results. It is the opinion of the authors of this paper that further refinement of these methods is needed. Stage 3 Assessment was more effective in predicting ground deformations and impact on existing structures. However, Stage 3 Assessment (numerical modeling) is time consuming and more expensive. More efficient empirical analysis methods are needed especially for shaft construction.

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PART 20

SEM/NATM

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LESSONS LEARNED FROM NATM DESIGN AND CONSTRUCTION OF THE CALDECOTT FOURTH BORE

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ABSTRACT

The Caldecott Fourth Bore is a 15.2 m wide (50 ft), 1,036 m long (3,399 ft) highway tunnel with seven cross passages located on State Route 24 in the San Francisco Bay Area, excavated through weak, highly fractured, and sheared sedimentary rock formations. This paper describes key observations and lessons learned from the design and construction of this NATM (New Austrian Tunneling Method) tunnel, including:

- Predicted versus observed ground behaviors and support performance based on direct observation and measured convergences
- Variations in tunnel production rate by support category
- Contractual considerations regarding support selection criteria
- Installation and performance of a 52 m long (170 ft) pipe canopy
- Effects of tunnel construction on slope stability

INTRODUCTION

Project Description

The existing Caldecott Tunnels consist of three bores along State Route 24 (SR 24) through the Berkeley Hills in Oakland, California. The California Department of Transportation (Caltrans) and the Contra Costa Transportation Authority (CCTA) proposed construction of a Fourth Bore that provides two additional traffic lanes to address congestion on SR 24 near the existing three Caldecott Tunnels. The length of the proposed Fourth Bore is 1,036 m (3,399 ft). The project includes short sections of cut-and-cover tunnel at each portal, seven cross-passageway tunnels between the Fourth Bore and the existing Third Bore, and a new Operations and Control Building.

The Fourth Bore provides two 3.7 m (12 ft) traffic lanes and two shoulder areas that are 3 m and 0.6 m (10 ft and 2 ft) wide. The horseshoe-shaped mined tunnel is 15 m (50 ft) wide and 9.8 m (32 ft) high. A typical cross section of the tunnel is shown in Figure 1. The tunnel includes a jet fan ventilation system; a wet standpipe fire protection system; and various operation and control systems, including closed-circuit television (CCTV) monitoring, heat and pollutant sensors, and traffic monitoring systems.

In accordance with general Caltrans practice for “important” facilities on lifeline routes such as SR 24, the seismic design for the Fourth Bore is based on the Safety Evaluation Earthquake (SEE) and a lower-level Functional Evaluation Earthquake (FEE). The project uses a 1,500-year return period for the SEE event and a 300-year

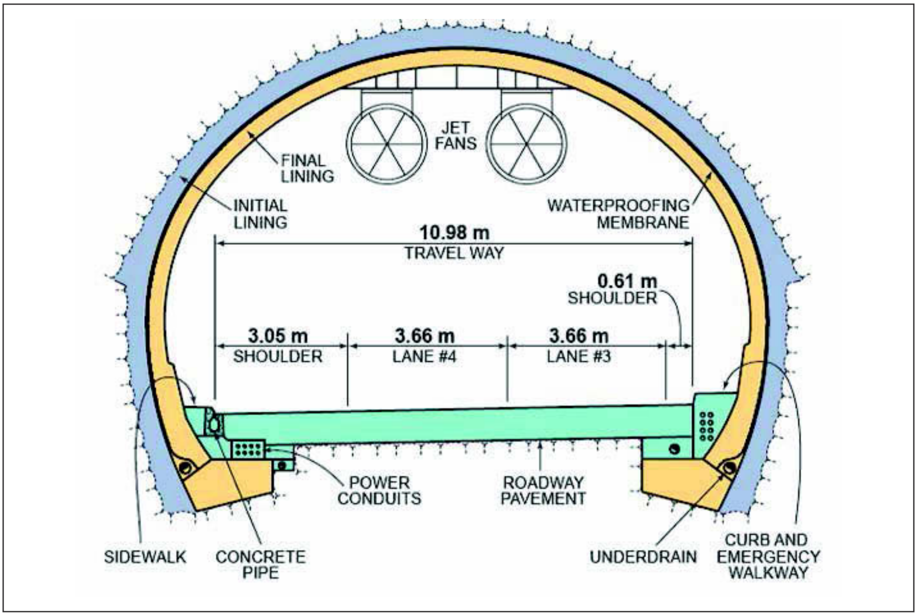

Figure 1. Typical cross section Caldecott fourth bore

return period for the FEE event. The performance requirements for the SEE are that the Fourth Bore will be open to emergency vehicle traffic within 72 hours following an SEE. Performance requirements for the FEE are that the Fourth Bore remains fully operational and experiences minimal, if any, damage.

Ground Conditions

The geology of the alignment is characterized by northwest-striking, steeply dipping, and locally overturned marine and nonmarine sedimentary rocks of the Middle to Late Miocene age. The western end of the alignment traverses marine shale and sandstone of the Sobrante Formation. The Sobrante Formation includes the First Shale, Portal Sandstone, and Shaly Sandstone geologic units as identified by Page (1950). The middle section of the alignment traverses chert, shale, and sandstone of the Claremont Formation. The Claremont Formation includes the Preliminary Chert, Second Sandstone, and Claremont Chert and Shale geologic units (Page, 1950). The eastern end of the alignment traverses nonmarine claystone, siltstone, sandstone, and conglomerate of the Orinda Formation. Major formations and geologic units within these formations are shown in Figure 2. A summary of key properties of these formations is presented below, and further details on site investigations and ground characterization are described in Thapa et al. (2008a,b, 2009).

The geological structure of the project area has been characterized as part of the western, locally overturned limb of a broad northwest-trending syncline, the axis of which lies east of the project area. The Fourth Bore alignment encountered four major inactive faults, which occur at the contacts between geologic units. These faults strike northwesterly and perpendicular to the tunnel alignment. In addition to the major faults, many other zones of weak ground were encountered, such as smaller-scale faults, shears, and crushed zones. The active Hayward fault, located 1.4 km (0.9 mi) west of the Caldecott Tunnel, is the closest regional fault to the project site. Engineering characteristics of ground conditions along the alignment are described later in this paper.

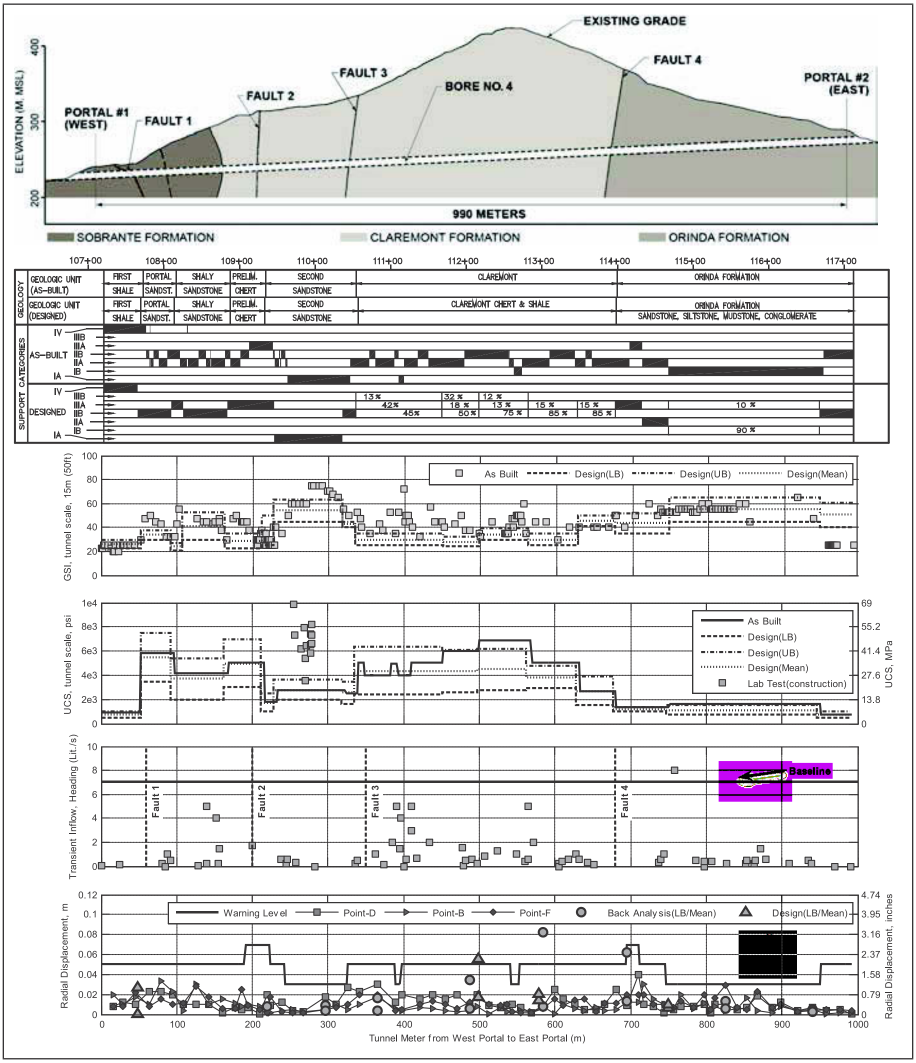

Figure 2. Longitudinal profile of design prognosis versus as-built ground conditions, support categories, and tunnel convergence (monitoring point location key shown looking at the heading for east and west portal drives)

Based on the occurrence of gas in the first three tunnels, the Fourth Bore was classified as a gassy tunnel by the California Occupational Safety and Health Administration, but was later reclassified to potentially gassy after breakthrough of the top heading.

OVERVIEW OF DESIGN AND CONSTRUCTION

A prescriptive approach to the specification of the excavation and initial support requirements was adopted to implement NATM construction of the Fourth Bore. Excavation and support requirements for each support category addressed the overall excavation and construction sequence—including restrictions on advance lengths, drift dimensions, arrangement and dimensions of support elements, as well as acceptable, alternative schemes where applicable. The global construction sequence consisted of a top heading and bench, or top heading, bench, and invert excavation sequence. The initial support system comprised fiber-reinforced shotcrete; drill and grout, as well as self-drill and grout rock dowels; lattice girders; invert arch; and drill and grout, as well as self-drill and grout spiles, pipe canopy, fiberglass face dowels, face sealing shotcrete, and a sloped core as face support as an alternative to face dowels. The appropriate combination of these support elements was prescribed based on ground conditions and predicted ground behavior.

Design and payment of the NATM tunnel excavation and support was organized into: (1) standard support consisting of four major support categories and three subtypes, each having a separate pay item; and (2) 20 additional support elements (tool box items) on a unit-price basis (including time-dependent costs such as impacts on advance rates) to address local ground conditions/behaviors, as required. Table 1 summarizes the key support elements for the four major support categories, and Figure 3 shows the arrangement of support elements and support installation requirements for one of the support categories.

Additional support measures were supplementary to the standard support measures. These additional measures were required to address observed or measured local ground conditions or behaviors and were installed when, for example, the measured convergence exceeded warning levels or when specific ground conditions or support system behaviors were observed, as defined in the contract documents. Estimated quantities of additional support measures included in the contract were based on an assessment of variations in expected ground conditions based on the results of the site investigation program. Additional support elements included spiling, rock dowels, additional shotcrete lining or face sealing shotcrete, face dowels, lattice girders, and an invert arch. Other additional pay items included the drilling of additional probe and drain holes.

Standard support categories and additional support measures were designed to address seven anticipated ground behaviors (Table 2). These predicted behaviors at defined locations along the alignment were established using numerical analyses to evaluate forces, moments, and rotations in the shotcrete lining; forces in the rock reinforcement support elements; the stresses, strains, and associated displacements in the ground around the tunnel; as well as evaluations of possible block failure modes. In addition, the recorded behavior in the existing three Caldecott Tunnel bores (Thapa et al., 2007, 2008a,b, 2009) confirmed possible ground behavior modes—including running ground (23 cubic meters [30 cubic yards]) with groundwater inflow of 5 L/sec (80 gpm), block failure, caving in of ground above the tunnel crown (extending to the ground surface in one instance), and slaking. Each standard support category was designed to support a defined ground condition that, along with the in situ conditions, resulted in a combination of these anticipated behaviors; it is this combination of ground conditions/in situ conditions and ground behaviors that defines a ground class (GC). GC and SC typically had a one-to-one correspondence. Support application criteria were based on encountered ground behaviors, conditions, measured lining deformations, and observed support performance. Daily meetings between the contractor's and engineer's tunneling experts were used to discuss monitoring data and support performance and to decide upon the required excavation and support measures for the next 24 hours. The support category as well as additional support measures were typically proposed by the contractor and approved by the engineer.

The contract was advertised to bidders by the California Department of Transportation in May 2009 and was awarded to the low bidder, Tutor Saliba Corporation (TSC), on November 20, 2009. Tunnel construction was preceded by portal excavation and support, which began concurrently on the east and west sides of the alignment in May 2010. Canopy pipes were installed at both portals prior to start of tunneling. Break-in occurred in August 2010 at the east portal, and in March 2011 at the west

Table 1. Summary of support categories (SC)

Support Category	Max. Advance Length, m (ft)	Systematic Presupport Measure	Face Support	Min. Shotcrete Thickness, mm (in.)	Avg. Radial Dowel Spacing, m (ft)	Temporary Shotcrete Invert Arch
I	1.8 (6.0)	None	SC IA: face dowels/ sealing fiber reinforced shotcrete (FRS) as required SC IB: systematic face dowels/sealing FRS	203 (8)	1.8 m (6)	None
II	1.4 (4.5)	SC IIA: none SC IIB: spiles	Face dowels/sealing FRS or sloping core/ sealing FRS	254 (10)	1.5 m (5)	None
III	1.0 (3.3)	Spiles	Sloping core/sealing FRS	305 (12)	1.2 m (4)	SC IIIA: none SC IIIB: top heading and bench
IV	1.0 (3.3)	Pipe canopy	Sloping core/sealing FRS	305 (12)	None	Top heading and bench

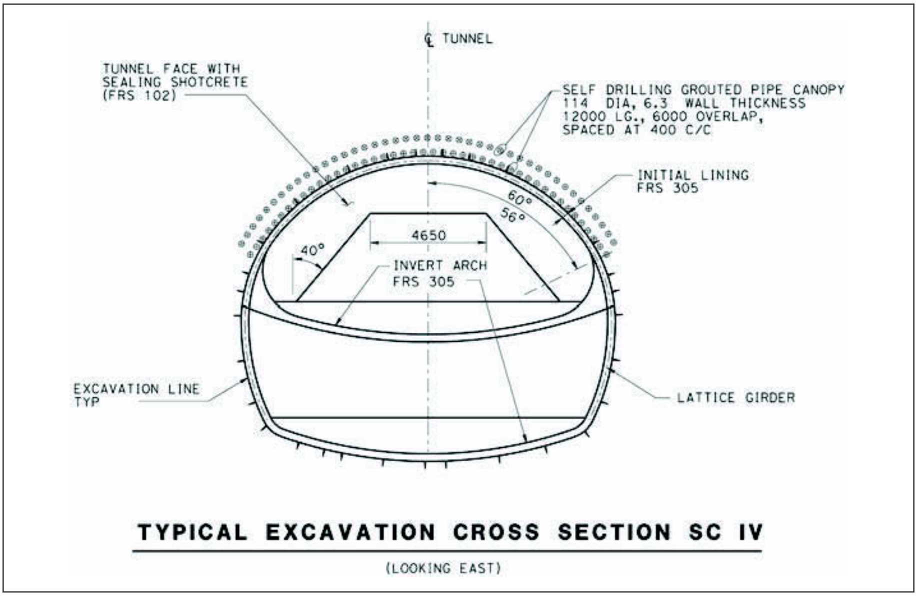

Figure 3. Example of support category requirements, typical excavation cross section for Support Category IV

portal. The contractor elected to drive the top heading from both ends of the alignment concurrently to expedite the schedule. Benching followed completion of the full top heading. Approximately 80% of the mined tunnel alignment (790 m [2,590 ft]) was excavated from the east portal, and the remaining 200 m (660 ft) was excavated from the west portal.

The east portal heading was advanced by TSC using a Wirth T3.20 roadheader, a Putzmeister shotcrete robot, an RDH two boom drill jumbo, RDH haul trucks and an RDH scissor lift, and a Sandvik load haul dump. Fiber reinforced shotcrete was pumped into the tunnel from the east portal as much as 700 m (2,100 ft) and was retarded for a maximum period of five hours. Beyond 600 m the shotcrete was trucked into the tunnel using a permissible haul truck with attached concrete mixing drum. Required excavation and support sequencing of the tunnel cross section within each advance length was performed in halves starting with excavation on the left side. While continuing with the excavation of the right side, the contractor concurrently started support installation on the left. The cycle finished with support application on the right side. DIBIT laser scanning of the excavated surface was performed immediately after excavation and again after shotcrete application to verify installed thickness and sufficient clearance from the design excavation lines. Production shotcrete testing was performed on a daily basis at the beginning of the project to verify achievement of early strength development and ultimate strength. Testing frequency was relaxed as construction proceeded, because the testing results consistently met or exceeded the specified requirements.

The west portal heading was advanced by Foxfire Constructors (FFC), a subcontractor to TSC. FFC used a Caterpillar 330C excavator with Alpine roadheader attachment or heavy hydraulic hammer attachments for excavation. Fiber reinforced shotcrete was pumped into the tunnel from the west portal for the full length of this heading. Other means and methods were similar to the east heading.

Table 2. Ground behaviors*

Behavior	Description of failure modes and manifestations in an unsupported tunnel
Block failure	Block failure is the discontinuity-controlled, gravity-induced failure of rock blocks that manifests as falling and sliding of blocks.
Raveling	Raveling is the progressive, discontinuity-controlled failure of small rock blocks within the general rock mass at or near the excavation surface. Raveling is manifested as successive fallout of small rock blocks and can ultimately result in a significant overbreak.
Shallow shear failure	Shallow shear failure results from overstressing of the ground within 0.25D to 0.5D of the tunnel perimeter (D = tunnel diameter) and may be enhanced by the potential for discontinuity and gravity-controlled failure modes. Shallow shear failure is manifested by moderate inward movement of the tunnel perimeter, including invert heave, and possibly by movement of rock into the tunnel opening along discontinuities.
Deep shear failure	Deep-seated shear failure results from overstressing of the ground beyond 0.25D to 0.5D from the tunnel perimeter. Deep-seated shear failure manifests as large radial convergence of the tunnel perimeter, including invert heave.
Slaking/ softening	Slaking is the deterioration and breakdown of intact rock upon exposure by excavation and manifests as slabbing of material from the crown and sidewalls. The severity of this behavior is assessed on the basis of slake durability tests performed according to ASTM Test Method D4644. Softening, which is dependent on wetting and exposure by excavation, is the reduction of intact rock strength at the invert or elsewhere and manifests as the development of a muddy or unstable invert or sloughing along segments of the tunnel perimeter elsewhere.
Swelling	Swelling occurs because of absorption of water by clay minerals in rock upon excavation-induced unloading. Swelling manifests as movement of the ground into the tunnel opening or additional tunnel support loading.
Crown instability due to low cover	Excessive crown geological overbreak and chimney-type failure will occur because of lack of confinement under low-cover reaches at portals. It manifests as block fallout and raveling above the crown.

* Modified from Austrian Society for Geomechanics, 2004.

Breakthrough of the top heading occurred at the end of November 2011 from the east portal heading after tunneling from the west side was completed to the breakthrough location roughly two weeks earlier. The breakthrough location at the west side was supported with a face supporting core, face sealing shotcrete, and spiles. When the tunnel heading approached from the east side, the same support measures as on the west side were installed to stabilize the decreasing rock pillar between the two headings. Breakthrough was completed successfully without any unexpected ground behaviors.

TSC's bench excavation sequence consisted of a center cut excavation followed by excavation of remaining side berms and installation of the tunnel sidewall support. TSC elected to perform the center cut bench excavation working eastward from the breakthrough point for the majority of this reach. Foxfire excavated the full face of the bench from the west portal towards the breakthrough point. Invert excavation and support followed benching, where required. Bench and invert excavation were completed in September 2012.

Final lining construction used a 15 m (50 ft) long form that was advanced uphill from the west to the east from April to October 2012. Typically, it took 8 to 10 hours to move, set, and place the concrete and another 8–10 hours for the concrete to set sufficiently to allow form removal, resulting in 4 to 5 form advances per week over a 6-day workweek.

KEY LESSONS

Predicted Versus Observed Ground Behaviors and Support Requirements

Encountered ground conditions and behaviors were mapped by the contractor's as well as the engineer's geologists on a daily basis during all phases of construction for each face. Probe holes were instrumented using an automatic data logger that recorded feed pressure, torque and advance rate and this information was interpreted to predict the ground conditions ahead of the tunnel face. Convergence monitoring was carried out across the tunnel arch and bench walls at instrumentation stations spaced approximately 15 m (50 ft) that were typically monitored within 100 m (300 ft) of the tunnel heading. All of this information was reviewed by the contractor's and the engineer's tunneling experts for ground classification and support selection at daily meetings.

Results of mapping are summarized (Figure 2) in terms of geological unit limits, Geological Strength Index (GSI; Marinos et al., 2005), and intact rock strength. Figure 2 shows that encountered ground conditions along the alignment were generally consistent with the design prognosis, with the exception of two reaches of the tunnel totaling 87 m (286 ft), or 9% of the alignment. These two reaches of differing site conditions occurred within the Second Sandstone between TM 241 and 322 (79 m [260 ft]) and within the Claremont Chert and Shale between TM 386 and 394 (8 m [26 ft]). In the Second Sandstone, the rock structure of the Second Sandstone encountered in the tunnel between TM 241 and 322 was blocky to massive, in contrast to the predicted blocky structure, and the intact rock strength was approximately 25% higher on average than indicated from strength tests performed during the design stage. The sandstone dikes in the Claremont Chert and Shale encountered in the tunnel between TM 386 and 394 exhibited a blocky to massive structure, in contrast to the predicted very blocky rock structure in the best rock mass in this formation.

As summarized in Figure 2, the groundwater inflows measured during construction were within the values of flush flow and sustained flow baselined in the Geotechnical Baseline Report—7 L/sec (110 gpm) and, 6 L/sec (95 gpm), respectively—at all but one location in the Orinda Formation. In this location the measured flow exceeded the predicted flush flow by approximately 1 L/sec (16 gpm) for a few hours.

Limited block failure, raveling, and shallow shear failure behaviors were observed to manifest during excavation and before the application of sealing shotcrete at the face and perimeter of the advanced heading in Support Categories II and III. Block failures typically involved sliding or toppling of blocks associated with steep bedding planes inclined into or away from the face. Raveling occurred over parts of the face and perimeter at locations of relatively higher fracturing and/or loosening. Raveling progressed above the crown spiles, in some instances requiring backfill grouting and longer 6 m (20 ft) spiles. Shallow shear failures primarily manifested as tunnel convergence, described below, and also as failure and movement of weak fractured zones at the face and heading perimeter. Large movements associated with deep shear failure were not observed. Slaking and softening behaviors were evident in the face and heading perimeter, as well as the invert prior to application of sealing shotcrete and mud mat, respectively. This was particularly the case within the Orinda Formation. Limited convergence of about 5 mm (0.2 in.), measured at a few locations in the Orinda Formation well behind the heading reach undergoing stress/strain redistributions, indicates possible manifestation of swelling behavior. However, we note that these movements behind the heading may also be due to other causes, such as the build-up of local hydrostatic pressure. The potential for swelling behavior was confirmed by swell testing of rock samples from the invert and was addressed in the design of the final lining invert. Observed behavior regarding crown instability due to low cover is discussed later in this paper. All of the anticipated behaviors were encountered to varying degrees, and were successfully addressed by the combination of excavation and support sequences specified in the contract.

Figure 2 shows that the measured tunnel convergences are generally consistent with values predicted during the design phase, as well as with back analyses made during construction based on encountered ground conditions. We note that there is variability of measured convergences around the tunnel perimeter at any location along the alignment, as well as in-between adjacent monitoring locations along the alignment, reflective of factors such as local variations in ground or groundwater conditions, differences in excavation and support installation details (such as overexcavation, shotcrete thickness, and cycle times), and the amount of time after excavation when the monitoring bolt was installed.

Figure 4 shows that the measured tunnel convergences are below the warning level except at a few locations, where they approached but never exceeded the warning level. Additional support measures were installed at some of these locations to address the higher deformations. Warning levels were defined by considering the anticipated movements, shotcrete lining capacity, and practical considerations such as measurement accuracy.

Figure 2 shows that the design prognosis for the four major support categories matches the installed support quantities along the majority of the alignment. The major difference between the design prognosis and installed support quantities is related primarily to the lesser quantity of SC III that was installed compared to the design prognosis (this difference is not associated with the differing site condition reaches discussed above). The reason for this difference is that, while ground conditions anticipated to require SC III based on GSI, UCS data, and ground cover (Figure 2) were encountered, SC III could nonetheless be avoided in these reaches because of the contractor's high strength fiber reinforced shotcrete. This was shown by back analysis results, which indicate that while an SC II shotcrete lining with contractually required minimum strengths would have been overstressed (Figure 4, left), normal forces and moments are within the allowable criteria using the much higher as-built shotcrete strength (Figure 4, right). The better than specified shotcrete strength allowed for support selection of a thinner shotcrete lining, while still maintaining the required lining performance. In addition to savings in material costs, savings were realized because there was less time required for shotcrete installation and there was a longer allowable advance length. Both effects produced higher production rates with SC II instead of SC III. The predicted total quantity of SC III was 257 m (843 ft), as compared to the installed quantity of 60 m (197 ft).

Figure 2 also shows a significant deviation of the installed quantity and locations of SC II subtypes SC IIA and SC IIB from the design prognosis. The difference between these SC II subtypes is the extent and pay mechanism for spiling. Spiling was an additional support measure in SC IIA, while SC IIB included systematic spiling (total of 54 spiles) over the entire arch. The design intent was that SC IIB would be utilized where spiling was necessary around the majority of the arch and that SC IIA would be utilized where spiling was required over a limited portion of the arch. The contractor's interpretation of the contract was to apply the pay item for additional spiles applicable to SC IIA unless the full number of 54 spiles, as prescribed for SC IIB, were required. Negotiations between the contractor and the engineer established a payment criterion that compensated the contractor for SC IIB when more than 37 spiles were required at a particular location, and compensated the contractor for SC IIA plus the unit price for the number of spiles when less than 37 spiles were required. This deviation from the design intent resulted in the following differences between the predicted support and as-installed support:

- The predicted total quantity of SC II was 412 m (1,351 ft), compared to the installed quantity of 568 m (1,863 ft).

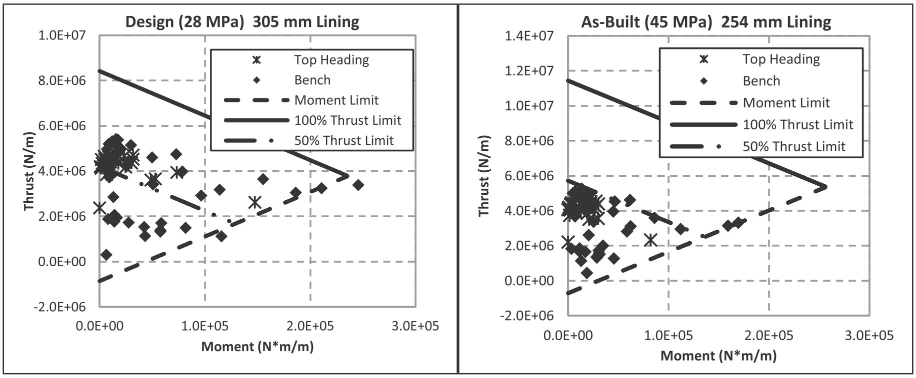

Figure 4. Shotcrete utilization at TM 690

- 35 m (115 ft) of SC IIA was predicted to be required, compared to the 380 m (1,246 ft) installed.
- 377 (1,236 ft) m of SCII B was predicted to be required, compared to the 188 m (616 ft) installed.

Contractual lessons related to the variation in quantities of SC IIA and SC IIB resulting from the application criteria for SC II are discussed further below.

Variations in Production Rates with Support Requirements

The majority of the tunnel (approximately 790 m [2,590 ft]) was excavated from the east portal, and the excavation of the top heading from the east portal took 15 months. The excavation of the west portal top heading was also completed within this timeframe. The length of the workweek varied between 5 and 6 days, with 20 to 24 working hours per day. The average production rate for the entire tunnel was 15.5 m (50.9 ft) per workweek, or 2.9 m (9.5 ft) per workday, based on the total length of tunnel and the time required for the east portal advance.

The best production of the top heading was achieved in Support Category IB with 11 advances of 1.8 m (5.9 ft) advance length, which is equivalent to 19.8 m (65.0 ft) per workweek. The highest production rates for other support categories (top heading) are:

- Support Category IA (in the Second Sandstone): 7 advances, with an increased advance length of 2.0 m (6.6 ft), or 14.0 m (45.9 ft) per workweek
- Support Category IIA: 14 advances, with a 1.4 m (4.6 ft) advance length, or 19.6 m (64.3 ft) per workweek
- Support Category IIB: 10 advances, with a 1.4 m advance length, or 14.0 m per workweek
- Support Category IIIA: 6 advances, with a 1.0 m (3.3 ft) advance length, or 6.0 m (19.7 ft) per workweek.

The weekly advance rates of the bench were subject to significant variation because of the different excavation sequences, the impact of footing excavations, longer haul routes, and impacts of off-cycle activities. Typical off-cycle activities were smoothing layer installation, excavation of the center cut ramp, and excavation and support of niches. Towards the end of the benching operation, the contractor focused on a concurrent bench/footing excavation, and weekly advance rates between 25 and 35 m (82 and 115 ft) were achieved. The peak weekly advance rate for the bench was approximately 45 m (148 ft). While typically the support and pre-support installation is critical in the top heading, the loading and hauling of excavated material were the critical activities in the benching operations.

The production rates achieved by the contractor confirm that performance (i.e., increased production) is improved by repetition of a systematic sequence of activities and that changes in the support measures generally slows down production.

It is also interesting to note that the “best” support categories (applicable to the best ground) with the longest allowable advance lengths did not result in the highest production rates. The highest production rates were achieved where all key activities of the cycle tied into each other with minimal friction and waiting time so that these activities progressed in parallel as much as possible. Longer advance lengths, for example, may lead to lower production rates as support installation activities “idle” during completion of the mucking operation, which can be significant with a long advance length for a large span tunnel.

Contractual Considerations and Unit Price Structure for Support Selection

The detailed and prescriptive design of the excavation and support sequence was developed to minimize the number of support categories and pay items with the goal to simplify the construction operations and avoid an overly complex and cumbersome contractual payment process. Standard support categories on the Fourth Bore were measured and paid on a per meter basis, with the pay item covering all associated excavation and support requirements. This approach was judged to be more conducive to promoting competitive and responsive bids. By comparison, the application of NATM in Europe typically allows selection of excavation sequence and initial support elements in combinations appropriate to variations in encountered ground behaviors so as to achieve the most efficient tunnel support system possible. The European approach often results in a highly variable excavation and support process that requires different pay items for each support element such that they can be combined as needed. However, using this approach can result in significant variations between estimated quantities that have to be addressed in the contract.

The experience with construction of the Fourth Bore indicates that the simplified contract structure minimizes the potential for misinterpretation of the contract as related to a multitude of support variations. As experience with NATM grows in the United States, it will be possible to develop designs with more flexibility that will require more sophisticated contractual payment structures.

The payment approach for each support category was successful except in the case of the spiling that is part of SC II, as described above. Based on the divergence of the contractor’s interpretation from the design intent and the variability in the number of spiles required per advance, it may have been more advantageous to remove a prescriptive design for the spiling from standard support measures and pay for the spiles as additional support (including time-dependent costs such as impacts on advance rates).

The daily meetings between a small group of the contractor’s and the engineer’s tunneling experts provided an efficient forum for jointly evaluating technical data—monitoring data, geological and hydrological conditions, support performance, and the expected conditions ahead of the face. Based on this joint review the group decided on the support class and support measures required for the day’s advances. A key lesson was to keep the meetings focused on solving the technical issues and to leave contractual disputes for another forum.

Experience with Installation of 52 m Long Canopy Pipes

The design drawings showed nine fans of 12 m (40 ft) long, 114 mm (4.5 in.) diameter canopy pipes to provide presupport for the first 52 m (170 ft) length of tunnel from the west portal, where the combination of low ground cover and disintegrated to very blocky disturbed seamy First Shale predisposed the tunnel to crown instability. The contractor proposed an alternative approach that entailed the use a single fan of 52 m (170 ft) long, 203 mm (8 in.) diameter canopy pipes installed using the Atlas Copco Symmetrix system and a Casagrande C8 drill, citing previous positive experience with this system. The canopy pipes were provided with one-way grout ports for grouting the pipe annulus.

The contractor's proposal was accepted after installation of two test pipes demonstrated that the maximum specified deviation of less than 396 mm (15 in.) along the entire pipe alignment and a maximum clear distance between any two adjacent pipes of less than 500 mm (20 in.) measured along the canopy arc could be achieved. The contractor agreed to install additional remedial pipes or spiling if the installed locations of the canopy pipes deviated from the design tolerances and cut out any pipes intruding into the tunnel excavation line.

The total of 51 canopy pipes including 8 remedial pipes were installed over a 3-month period, with construction difficulties resulting in significant delays especially during the installation of the initial pipes. Pipes that encroached into the initial lining were notched at the lattice girder locations or completely cut out, if the encroachment was more than 152 mm (6 in.) into the initial lining. At locations where pipes could not be installed for the full length or where deviated pipes created a gap between adjacent pipes of more than 500 mm (20 in.), remediation pipes were installed, if sufficient room for a remediation pipe installation was available. Where sufficient room was not available, local spiling was installed during the excavation and support of the section, if the encountered ground conditions dictated additional presupport measures. Once installed, the pipe canopy provided effective presupport for the ground.

Effects of Tunnel Construction Through Toe of a Weak Slope

The west portal (Portal No. 1) was cut into the side of a 43 m (140 ft) high slope with an average inclination of 33 degrees and consisted of colluvium underlain by the First Shale Formation. The First Shale is composed of a weak, black, silty shale that is highly fractured and crushed. The drained strength properties of the rock mass were determined using the Hoek-Brown procedure (Marinos et al., 2005), while the undrained rock mass strength was determined from laboratory testing of 60 to 145 mm (2.4–5.7 in.) diameter core samples. The rock mass properties are summarized below:

- Geological Strength Index (GSI): 26
- Unconfined compressive strength of intact rock (UCS): 5.3 MPa (800 psi)
- Hoek-Brown constant m_i: 7
- Mohr Coulomb friction angle: 30 degrees
- Cohesion: 65 kPa (1,350 psf).
- Undrained shear strength: S_u (kPa) = 7.35(z) + 38.5 (kPa) or S_u (psf) = 153(z) + 804 (psf)

A colluvium deposit, ranging in thickness from 0 to 10 m (0–32.8 ft), overlies the First Shale on the slope above the tunnel alignment. The colluvium deposit consists of stiff to very stiff clayey sand with gravel-sized rock fragments, sandstone blocks up to 0.3 m (1 ft) in dimension. No evidence of slip surfaces that would indicate past mass movement were identified during the investigation program that included eight borings into this deposit. Mohr Coulomb properties of the colluvium are a friction angle of 34 degrees and a cohesion of 10 kPa (200 psf). The groundwater table in the slope followed the slope contours at a depth of about 7 to 9 m (23–30 ft) below the ground surface.

Landslides frequently occurred on the slopes above the west portal cuts in the First Shale during construction of the First and Second Bores, at times measuring several thousand cubic yards. No slope instability occurred during construction of the Third Bore; however, the 6 m (20 ft) deep excavations adjacent to the slope to the north of the tunnel utilized soldier beam support.

The design of the portal structures (not discussed in this paper) and tunnel support system through the toe of the First Shale slope were developed to prevent any destabilization of the steep and potentially unstable slope. Measures employed included the use of pipe canopy presupport as discussed above; closing of the tunnel ring within 6 m (20 ft) and 10 m (33 ft) of the top heading and bench cut faces, respectively; limiting advance lengths to 1 m (3.3 ft) and 2 m (6.5 ft) for the top heading and bench, respectively; and the implementation of face support measures comprising a buttress as well as 102 mm (4 in.) of sealing shotcrete.

Stability of the slope was monitored during tunnel excavation by measurement of ground surface settlement and inclinometer readings. The tunnel section was also monitored for convergence, and the shotcrete lining was inspected daily to identify cracking. Monitoring during construction showed maximum ground surface settlements were about 35 mm (1.4 in.), and as shown in Figure 2, measured tunnel convergence was less than 25 mm (1 in.).

CONCLUSIONS

Encountered ground conditions and behaviors were generally consistent with design prognosis except for two locations, which amounted to 9% of the full tunnel length. Major support categories installed were also generally consistent with the design prognosis, with the exception of the less than anticipated use of Support Category III, which can be attributed to the installation of shotcrete that achieved almost double the specified strength. A significant deviation from bid quantities between subtypes of Support Category II (SC IIA versus IIB) occurred because of inclusion of spiling in the lump sum pricing structure used for Support Category IIB, and the contractor's interpretation of the conditions where each support subtype should be used. This variation from the bid quantity for the overall support category could have been avoided by paying for the spiles on a unit price basis.

One of the challenges on this project was construction of this 15 m (50 ft) span tunnel through the toe of a slope consisting of weak and broken shale. The use of ring closures, canopy pipes, and small advance lengths allowed for successful advance of the tunnel through this weak slope without adverse impacts to the slope. Canopy pipes at the west portal were installed in a single 52 m (170 ft) length per the contractor's preference, instead of nine fans of 12 m (40 ft) long pipes. Several difficulties were encountered during installation of the 52 m long canopy pipes, requiring removal of some pipes and installation of additional pipes. However, the system, once installed, provided the required presupport to the ground and allowed an uninterrupted excavation and support cycle through this difficult reach of tunnel.

Construction of the Fourth Bore demonstrated that production is enhanced by the consistent repetition of the same or similar excavation and support cycles. In general, too many changes between excavation and support activities and the implementation of off-cycle activities interrupts the overall cycle and results in negative impacts on production. Typical weekly advance rates for the Caldecott Fourth Bore were between 15 and 20 m (49 and 66 ft) for the top heading and 25 and 35 m (82 and 115 ft) for the bench.

The as-designed tunnel excavation and support sequence effectively controlled adverse ground behaviors that had been encountered during construction of the previous Caldecott Bores. Drainage ahead of the face, face dowels, a face supporting buttress, spiling, control of advance lengths, and sealing shotcrete were all critical to control block failure, raveling, and shallow shear behaviors at the face in various instances. Similarly, installation of a high-performance shotcrete lining and fully cement-grouted dowels immediately behind the face was effective in addressing anticipated behaviors behind the face in the fractured and weak ground conditions along the alignment. Daily meetings between the contractor and the engineer were key to application of the appropriate combination of excavation and support measures, which enabled advance of the heading in the most efficient manner possible and completion of tunnel excavation and support on schedule.

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LOAD SHARING IN TWO-PASS LINING SYSTEMS FOR NATM TUNNELS

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ABSTRACT

Generally, two-pass lining systems are used for transportation tunnels constructed using the New Austrian Tunneling Method (NATM). The initial lining can support a significant portion of the long-term ground loads and thus reduce the design loads on the permanent lining. This load-sharing phenomenon has traditionally been neglected, but considering load sharing in design can result in significant cost savings. This paper discusses the basic principles of load sharing in two-pass lining systems and proposes a rational method for design. Practical application of the load sharing design approach in a NATM tunnel project, the Caldecott Fourth Bore, is presented.

INTRODUCTION

Most transportation tunnels constructed using the New Austrian Tunneling Method (NATM) or Sequential Excavation Method (SEM) use a two-pass lining system. The initial lining typically consists of fiber reinforced or plain shotcrete, often augmented by some form of rock reinforcement (e.g., rock dowels), while the secondary lining (also called final or permanent lining) consists of either shotcrete or cast-in-place (CIP) reinforced concrete. Depending on ground and hydrogeologic conditions, a spray-on or sheet waterproofing membrane is often installed between the initial and secondary linings to ensure a watertight tunnel.

The initial lining and rock reinforcement are typically designed to carry the ground loads, control the ground deformations during tunnel construction, and provide a safe work environment. The secondary lining is designed to support the long-term ground loads, hydrostatic loads where applicable, and any additional loads resulting from finishes or anchored equipment. The secondary lining also provides a permanent lining for fire protection and accommodates seismic deformations if the tunnel is located in a seismically active region. With this design approach, the installed initial lining is not considered to be a component of the long-term load-carrying lining system. However, the initial lining does have the capacity to support a significant portion of the long-term ground loads during the design life of a tunnel. Neglecting the long-term load-carrying capacity of the initial lining results, in some cases, in an overly conservative design for the secondary lining. In order to achieve a cost-effective design, the concept of load sharing between the initial and secondary linings in the design of the secondary lining has been gaining increased acceptance by tunnel designers in recent years. This load-sharing approach has been applied to the design of many NATM tunnels worldwide.

This paper discusses the basic mechanism of load sharing in two-pass lining systems and proposes a rational approach for design. The practical application of the load-sharing design approach in a NATM tunnel project, the Caldecott Fourth Bore, is also presented.

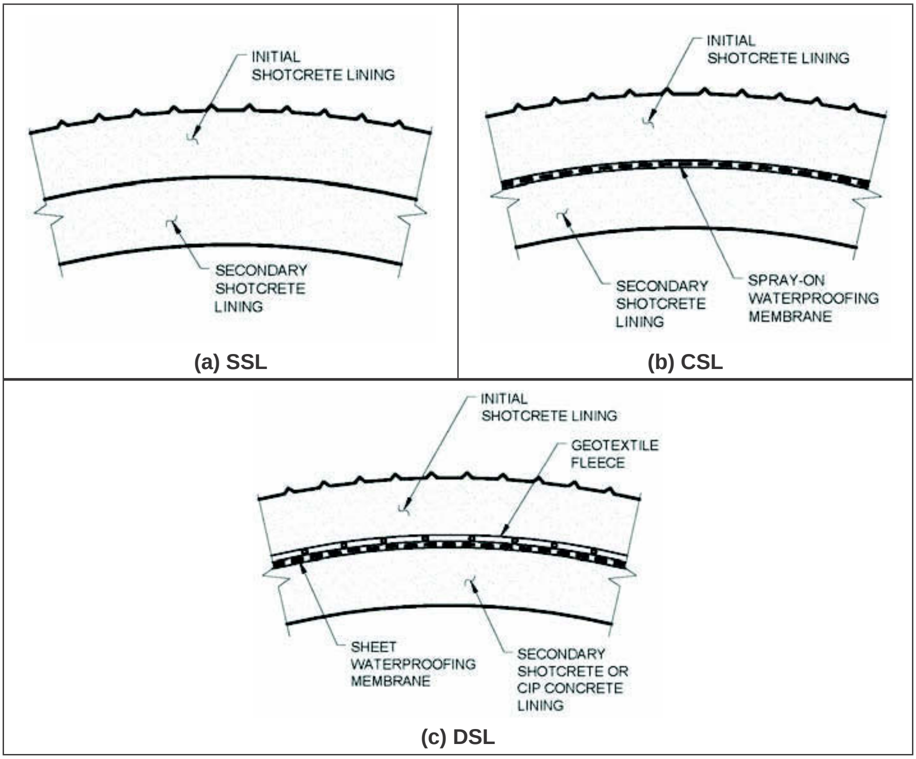


Figure 1. Two-pass lining systems: single shell lining, composite shell lining, and double shell lining

TYPICAL TWO-PASS LINING SYSTEMS IN NATM TUNNELS

Three types of two-pass lining systems (Figure 1) are widely used in NATM tunnels. These are briefly described below.

Single Shell Lining (SSL)

The SSL is defined herein as a combined lining system installed in two stages. The SSL consists of an initial shotcrete lining and a secondary shotcrete or CIP concrete lining and does not include a spray-on or sheet waterproofing membrane between the initial and secondary linings. The secondary shotcrete or CIP concrete lining is placed directly against the initial shotcrete lining, achieving an intimate bond to the initial lining because of the rough surface along the interface between the initial and secondary linings. Under loading, this combined SSL behaves as a single lining system, and no relative displacement occurs along the interface between the initial and secondary linings in either the radial or tangential direction. A bonded (no slip) interface condition with high stiffness is representative for the SSL system.

Composite Shell Lining (CSL)

The CSL consists of an initial shotcrete lining, a spray-on waterproofing membrane, and a secondary shotcrete lining. The use of a spray-on waterproofing membrane allows good adhesion to both the initial and secondary shotcrete linings and prevents any voids between the linings. Under loading, this lining system behaves as a composite shell lining system. Relative displacements in both the radial and tangential directions between the initial and secondary linings may occur as a result of deformations of the spray-on waterproofing membrane. These relative displacements depend on the thickness and stiffness of the spray-on waterproofing membrane. A bonded interface condition with low stiffness is considered a reasonable representation for the CSL system.

Double Shell Lining (DSL)

The DSL consists of an initial shotcrete lining, a sheet waterproofing membrane, and a secondary shotcrete or CIP concrete lining. The sheet waterproofing membrane is not bonded to either the initial shotcrete lining or the secondary shotcrete or CIP concrete lining. Voids along the interface may still exist, even though contact grouting is usually performed to fill these voids. Under loading, the initial and secondary linings behave as two separate shells and relative displacements may occur in both the radial and tangential directions along the interface, especially in the tangential (shear) direction. A full slip interface condition is usually assumed for the DSL system.

DISCUSSION OF THE LOAD-SHARING MECHANISM

The initial shotcrete lining for a NATM tunnel is designed and installed as temporary support for carrying ground loads induced by tunnel excavation. The initial shotcrete lining may degrade over time, particularly in aggressive environments due to the corrosion of steel reinforcement subject to high chloride conditions or deterioration of the shotcrete because of the presence of sulfates in the ground or groundwater (Nordstrom, 2005; Santhanam et al., 2003). This degradation of the initial shotcrete lining causes redistributions of stresses and strains or loads in the lining and adjacent ground, and possibly additional deformation of the lining. As a result of the degradation and additional deformation of the initial lining, the loads originally developed in the initial lining will redistribute to both the adjacent ground and secondary lining over the long term.

The magnitude of load transferred from the initial lining to the secondary lining, also called the load sharing, depends on many factors, including:

- Available bond and normal and shear stiffness of the interface
- Ground conditions such as rock mass strength/stiffness
- Relative stiffness between the initial and secondary linings
- In situ stress conditions such as the ratio of horizontal-to-vertical stresses (K_o)
- Tunnel shape (e.g., circular and horse-shoe shaped).

As discussed above, there are two interface conditions, bonded and full slip, which are associated with the three types of lining systems: SSL, CSL, and DSL. The SSL approach assumes a fully bonded interface, while the DSL approach assumes a full slip interface. The CSL approach assumes a bonded interface with low shear stiffness, with the absolute stiffness depending on the thickness and properties of the spray-on waterproofing membrane.

The loading mechanisms of these lining systems in terms of their load sharing are illustrated in Figure 2. With a fully bonded interface condition, shear forces and normal forces are developed along the interface when the initial lining deforms under loading. These shear forces transfer additional loads to the secondary lining, resulting in higher axial forces (thrusts) in the secondary lining. With a full slip interface condition, shear resistance does not develop along the interface, so relative displacements (slip) may occur between the initial and secondary linings and the transfer of load from the initial to the secondary lining is limited.

The magnitude of shear stresses that can develop at the interface within the SSL, CSL and DSL is described below:

$$\tau_{ss} >> \tau_{cs} \ldots\ldots \tau_{ds} = 0 \quad (1)$$

where τ_{ss}, τ_{cs}, and τ_{ds} are the shear stresses along the interface for SSL, CSL, and DSL systems, respectively.

METHODOLOGY FOR EVALUATING LOAD SHARING

Development of a closed-form analytical solution to estimate the magnitude of load sharing requires many simplifying assumptions; therefore, the use of numerical methods is the preferred approach for evaluating load-sharing behavior. This section discusses the approach for evaluating load sharing with two-dimensional numerical analyses using the FLAC software (Itasca, 2005).

The first step in performing a load-sharing evaluation is to estimate the magnitude of the loads that develop in the initial lining during tunnel excavation. The second step in the process is to model the installation of the secondary lining and the deterioration of the initial lining, which results in the transfer of ground loads to the secondary lining. The analyses assume:

- Only ground loads are considered to develop on the initial lining (i.e., groundwater loads are neglected). The initial lining is typically designed for ground loads only and often includes weep holes to prevent the buildup of hydrostatic pressures.
- The secondary lining is considered to carry the full hydrostatic pressure, where applicable, to the design of undrained tunnels. In reality, the hydrostatic pressure will cause additional deformations of the secondary lining, which could

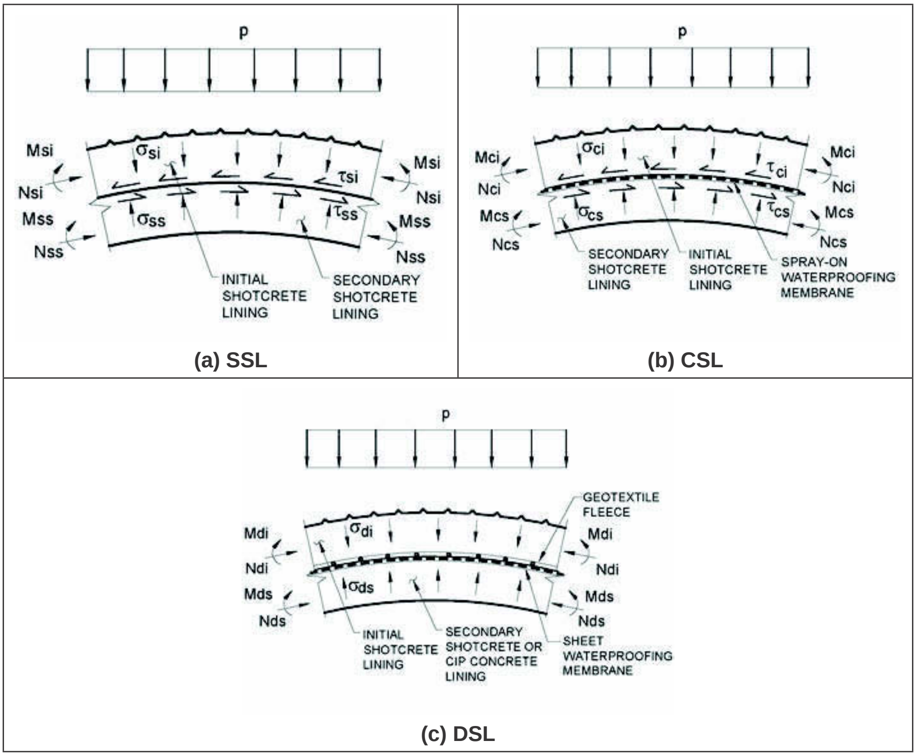


Figure 2. Load-sharing mechanism for SSL, CLS, and DSL lining systems

affect the magnitude of load sharing. For simplicity, the effect of hydrostatic pressure on the load sharing of ground loads is not considered in this paper.

- The degradation of the initial shotcrete lining and corresponding reduction in axial and bending stiffness is modeled by (Hoek, 2002):
  - reducing the cross-sectional area by 50 to 70%, and
  - reducing the moment of inertia by 100%.
- Rock dowels are often assumed to fully degrade in the load-sharing evaluation (Hoek, 2002). Generally, rock dowels installed during tunnel construction are not considered permanent and are subject to corrosion during the tunnel design life. Typically, this assumption has a minimal effect on the load sharing between the initial and secondary linings.

Modeling the load transfer from the initial lining to the secondary lining cannot be achieved simply by reducing properties of the structural elements that represent the initial lining. If these properties are simply changed after the model is in (mechanical) equilibrium, no load transfer will occur because constitutive models in FLAC operate in incremental fashion and incremental stresses are related to incremental strains. Therefore, changes in the initial lining properties when the lining-ground system is in equilibrium will have no effect on the system since the subsequent incremental strains are zero. To model the effect of changes in the initial lining properties, internal forces (thrusts, shears, and moments) in the lining must also be changed.

The internal forces in the initial lining are reduced using a FISH function in FLAC. To bound the solution, two separate analyses are recommended—one assuming the forces in the initial lining are reduced to zero (a conservative assumption that results in a maximum load transfer to the secondary lining); and the second assuming that the forces in the initial lining are reduced by 70%. In all of the analyses discussed in this paper, the forces in the initial lining are reduced to zero.

The specific modeling steps are as follows:

- ***Step I:*** Restore the saved FLAC file that contains internal forces (thrusts, shears, and moments) in the initial lining developed during and following tunnel excavations.
- ***Step II:*** Remove all structural elements that represent rock dowels, if included in the model.
- ***Step III:*** Install interface elements and structural elements that represent the secondary lining.
- ***Step IV:*** Reduce the initial lining properties (cross-sectional area and moment of inertia). These reduced properties are fixed during cycling.
- ***Step V:*** Reduce the internal forces (thrusts and shears) developed in the initial lining during tunnel excavation by 100%, and reduce moments by 100% prior to cycling. In some cases, the thrusts and shears in the initial lining are reduced by 70% prior to cycling. These reduced internal forces (moments, thrusts, and shears) will change during cycling based on the relative stiffness of each of the linings.
- ***Step VI:*** Cycle to equilibrium.

The magnitude of the load carried by the secondary lining is defined by the following expression:

$$LS = \frac{N_s}{N_i + N_s} \times 100\,(\%) \tag{2}$$

where *LS* is the magnitude of load sharing in percent; N_i is the axial force developed in the initial lining during the load-sharing analysis; and N_s is the axial force transferred to the secondary lining.

As discussed above, a portion of the ground load originally developed in the initial lining during tunnel excavations will often be transferred to the adjacent ground through the load redistribution process in the long term. This portion of ground load is not accounted for in estimating the magnitude of load sharing using Equation 2. Therefore, use of Equation 2 will result in a conservative assessment of the load carried by the secondary lining. However, it should be recognized that this behavior will not occur in all ground conditions, such as squeezing ground, where load transfer to the ground will not likely occur.

The use of the interface elements to model the interaction between the initial and secondary linings requires a realistic estimate of the interface normal and shear stiffnesses. These stiffnesses must be selected to allow realistic simulation of deformations and load transfer in the numerical model, but at the same time also avoid any numerical calculation issues due to incompatibility of adjacent material stiffnesses. In the analyses presented, the interface normal stiffness (K_n) is estimated (using Equation 3) based on the deformation modulus or stiffness of the medium/material adjacent to the interface.

$$K_n = \max\left(\frac{B + \frac{4}{3}G}{\Delta z}\right) \tag{3}$$

where K_n is the interface normal stiffness; *B* and *G* are the bulk and shear moduli, respectively, of the medium that controls the load transfer between two linings as discussed below; and Δz is the element size adjacent to the interface (Itasca, 2005).

For an SSL system, a reasonable estimate of the bulk and shear moduli in Equation 3 are those for the linings. For a CSL system, the load sharing depends on the thickness and stiffness of spray-on waterproofing membrane as well as the stiffness of the linings. The moduli for the membrane can be used in Equation 3 as an initial estimation for the interface normal stiffness and is considered a lower bound. The upper bound normal stiffness for a CSL system is estimated by using the lining moduli in Equation 3. A reasonable normal stiffness value can be determined by varying the normal stiffness value to calculate the deformations along the interface. An appropriate normal stiffness should produce the deformations that are equal to or less than the thickness of spray-on waterproofing membrane. Similarly for a DSL system, the upper bound normal stiffness is estimated by using the lining moduli in Equation 3. The lower bound value depends on the thickness and stiffness of sheet waterproofing membrane and geotextile fleece and whether there are any voids present along the interface, and can be several orders of magnitude lower than the upper bound value. The approach described above for estimating the normal stiffness for a CSL system can be employed for the DSL system.

If an interface is used to model an SSL system, the interface shear stiffness should be based on the properties of the lining, while for a CSL system the shear stiffness of the spray-on waterproofing membrane should be used to model the interface. For a DSL system with a full slip interface, the interface shear stiffness does not affect the load-sharing evaluation as discussed below.

It should also be noted that the load sharing discussed in this paper is based only on the loads related to axial forces or thrusts, which in a circular arch are directly related to the radial load applied. Load sharing in terms of shear forces or bending moments is not considered.

Table 1. Variables assumed in load sharing sensitivity study

Parameter	Value
Shear stiffness of interface (K_s), MPa/m (ksi/ft)	0, 22.6, 226, 2,260, 22,600, and 226,000 (0, 1, 10, 100, 1,000, and 10,000)
Normal stiffness of interface (K_n), MPa/m (ksi/ft)	0, 22.6, 226, 2,260, 22,600, and 226,000 (0, 1, 10, 100, 1,000, and 10,000)
Bonding condition	Bonded and full slip
Rock mass strength (UCS_{rm}), MPa (psi)	0.6, 2.4, 4.5 (90, 350, 650)
Tunnel shape	Circular and horse-shoe shaped
Tunnel diameter or width (D), m (ft)	3 and 15 (10 and 50)
In situ horizontal-to-vertical stress ratio (K_o)	0.5, 1.0, and 2.0
Secondary to initial lining stiffness ratio	2.0, 3.3, 4.0, and 6.7

FACTORS AFFECTING LOAD SHARING

Key factors that affect the magnitude of load sharing are investigated in a sensitivity study to demonstrate the variations in load sharing that can be expected. The factors analyzed include the following:

- Interface shear stiffness (bonded and full slip)
- Interface normal stiffness
- Rock mass strength/stiffness
- Relative stiffness between initial and secondary linings
- In situ stress conditions: ratio of horizontal to vertical stresses (K_o)
- Tunnel shape: circular and horse-shoe shaped

To simplify the analyses, the following assumptions are made in the sensitivity study:

- The ground (rock mass) is an isotropic and homogeneous continuum.
- The Hoek-Brown failure criterion is valid for modeling the behavior of the ground.
- Tunnel excavation is achieved in one step with a full face.

The range of tunnel size, ground cover, rock mass properties, relative lining stiffness, interface shear and normal stiffnesses, and mechanical properties assumed in the study are presented in Tables 1 and 2. Three rock mass strength values are used: 0.6, 2.4, and 4.5 MPa (90, 350, and 650 psi). The terms “weak,” “medium,” and “strong” correspond to these three rock mass types, respectively. These terms are defined to distinguish the rock mass types for discussion purposes. The quality of these rock mass types should not be equated to these terminologies as defined by ISRM (1981), where rock is classified in terms of intact rock strength.

Unless otherwise indicated, all analyses carried out in the sensitivity study are based on a horse-shoe shaped tunnel with a ground cover to tunnel width ratio equal to 3.

Effect of Interface Stiffness

The effect of the interface stiffness on the magnitude of loads transferred from the initial lining to the secondary lining is evaluated by analyzing cases with different shear stiffness (K_s) values while keeping other parameters unchanged. Six interface shear stiffness values are assumed: 0 (full slip), 1, 10, 100, 1,000, and 10,000 ksi/ft (see Table 1 for metric conversions for all ksi/ft to MPa/m). The range of these values is

Table 2. Rock mass properties

Parameter	Rock Mass Quality		
	Weak (UCS$_{rm}$ = 0.6 MPa [90 psi])	Medium (UCS$_{rm}$ = 2.4 MPa [350 psi])	Strong (UCS$_{rm}$ = 4.5 MPa [650 psi])
Unit weight, kN/m^3 (pcf)	22.0 (140)	24.4 (155)	24.4 (155)
Intact rock unconfined compressive strength (UCS), MPa (psi)	2.8 (400)	34.5 (5,000)	41.4 (6,000)
Geological Strength Index (GSI)	60	22	33
m_i	10	8	8
Deformation modulus, MPa (ksi)	366 (53)	690 (100)	2,069 (300)
Poisson's ratio	0.2	0.25	0.3

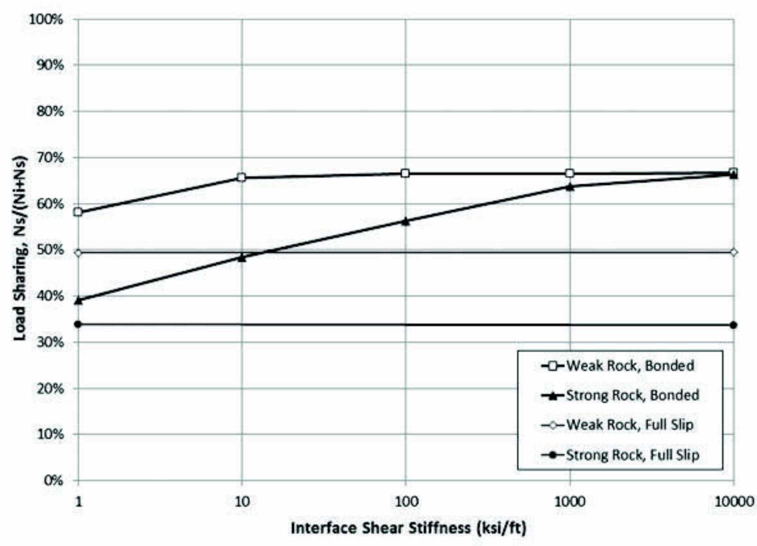
considered to bound typical variations of the lining interface shear stiffness for the three types of two-pass systems used in NATM tunnels. For example, for an SSL system, the interface shear stiffness is represented by the shear stiffness of hardened shotcrete or concrete lining (typically greater than 100 ksi/ft), while for a CSL system, the interface shear stiffness depends on the thickness and shear stiffness of spray-on waterproofing membrane and is low (typically less than 10 ksi/ft). For a DSL system with a full slip interface, the interface shear stiffness is zero and does not affect the load transfer from the initial lining to the secondary lining.

The effect of the interface normal stiffness is evaluated by varying the normal stiffness values while keeping other parameters constant. Six interface normal stiffness values are assumed: 0, 1, 10, 100, 1,000, and 10,000 ksi/ft.

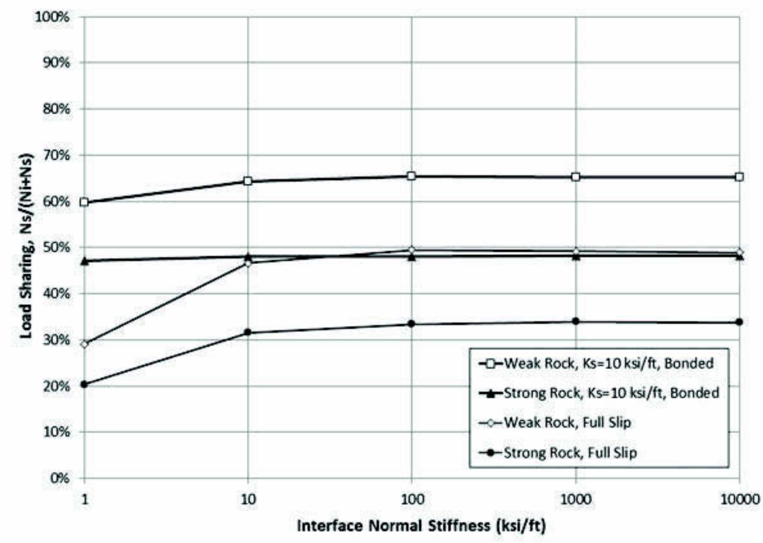
Figures 3a and 3b show the results of the analyses using different interface shear and normal stiffnesses, respectively. As illustrated on Figure 3a, the interface shear stiffness has a significant impact on the magnitude of load sharing when the interface is modeled as bonded, typical for an SSL or CSL system. Assuming a bonded interface, load sharing increases with increasing interface shear stiffness up to an interface shear stiffness of approximately 1,000 ksi/ft. The effect of interface shear stiffness is greater for stronger and stiffer rock masses because some of the load originally acting on the initial lining is shed into the ground when the initial lining degrades. This effect is more pronounced in stronger and stiffer rock with lower interface shear stiffness. In addition, in stiffer rock any lining deformations will lead to larger changes in normal stresses, which will result in larger changes in shear stresses. Since the thrusts developed in the secondary lining due to the degradation of the initial lining are affected significantly by the interface shear stresses, the load sharing for linings in stiffer rock is more sensitive to the interface shear stiffness. As shown in Figure 3a, the maximum magnitude of load sharing for a bonded condition is approximately 70%.

When the interface is modeled as full slip, typical for a DSL system, the magnitude of loads transferred to the secondary lining is not affected by the interface shear stiffness, and is controlled primarily by the stiffness of the adjacent rock mass (see Figure 3a). The higher a rock mass strength or stiffness, the lower the maximum loads that would be transferred to the secondary lining. Assuming a full slip interface, the magnitude of load sharing ranges from about 50% for weak rock to about 35% for strong rock.

Figure 3b shows the magnitude of load sharing as a function of the interface normal stiffness. As illustrated in Figure 3b, the magnitude of loads transferred to the secondary lining is not very sensitive to the variations of the interface normal stiffness, especially when the interface is modeled as bonded, typical for an SSL or CSL system, and the normal stiffness is high. For weak rock, the magnitude of load sharing



(a) Interface shear stiffness



(b) Interface normal stiffness

Figure 3. Axial force shared by secondary lining as a function of interface stiffness

increases slightly, in a range of about 5%, with increasing normal stiffness when K_n is low (less than 10 ksi/ft).

When the interface is modeled as full slip, the magnitude of load sharing increases by 10% for strong rock and 20% for weak rock, for low stiffness values (K_n<10 ksi/ft, see Figure 3b), but levels off at higher interface normal stiffness values. According to the results shown in Figure 3b, the load sharing is generally not very sensitive to the variation in interface normal stiffness. This finding will help to limit concerns about any potential uncertainties associated with estimation of an interface normal stiffness for use in analyses.

Effect of Rock Mass Strength and Stiffness

Three different rock mass strength/stiffness levels, as presented in Table 2, are used in evaluating the effect of the rock mass strength or stiffness on the magnitude of load sharing. Results of this evaluation are shown on Figure 4a. As indicated, the loads transferred to the secondary lining decrease with increasing rock mass strength/stiffness. The rate of decrease accelerates as the rock mass strength increases. This conclusion applies to the linings with either a bonded or full slip interface condition.

Effect of Relative Lining Stiffness

The effect of the relative stiffness of the initial and secondary linings on the magnitude of load sharing is shown in Figure 4b. The relative lining stiffness is defined as the ratio of the secondary lining axial stiffness to the initial lining axial stiffness. Four relative lining stiffness values—2.0, 3.3, 4.0, and 6.7—are used. Results of these analyses are presented in Figure 4b. As expected, the magnitude of loads transferred to the secondary lining increases as the relative lining stiffness increases because a stiffer secondary lining usually attracts more loads. This conclusion is applicable to the linings with either a bonded or full slip interface condition.

Effect of in situ Stress

The effect of the in situ horizontal-to-vertical stress ratio (K_o) on the magnitude of load sharing is evaluated for cases with three different K_o values: 0.5, 1.0, and 2.0. The results of this evaluation are presented in Figure 5a and indicate that the in situ stress condition (K_o) does not appear to significantly affect the magnitude of load sharing.

Effect of Tunnel Shape

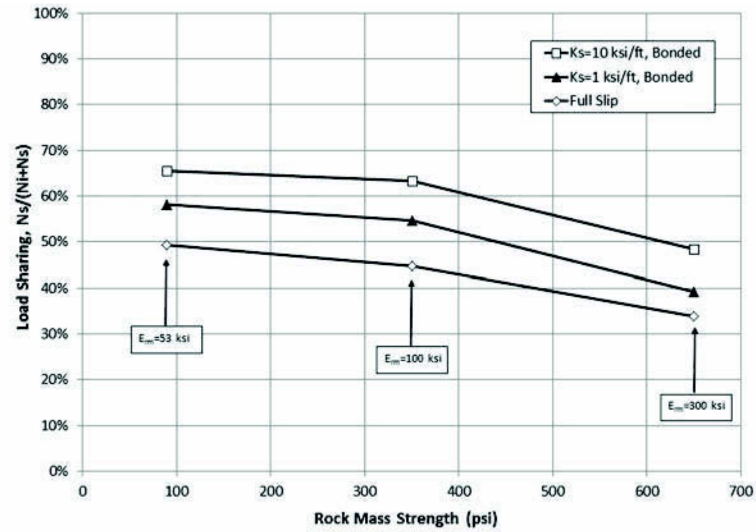
The effect of the tunnel shape on the magnitude of load sharing is evaluated by comparing the results for cases with circular and horse-shoe shaped tunnels. The results of this evaluation are presented in Figure 5b. In general, the tunnel shape does not appear to significantly affect the magnitude of load sharing when the interface shear stiffness is high (K_s > 10 ksi/ft). The tunnel shape may affect the magnitude of load sharing when the interface is modeled as bonded and its shear stiffness is low (less than 10 ksi/ft), which is typically associated with a CSL system. For a DSL system with a full slip interface, a horse-shoe-shaped tunnel is expected to attract slightly higher loads (about 5%) in the secondary lining than a circular tunnel.

APPLICATION

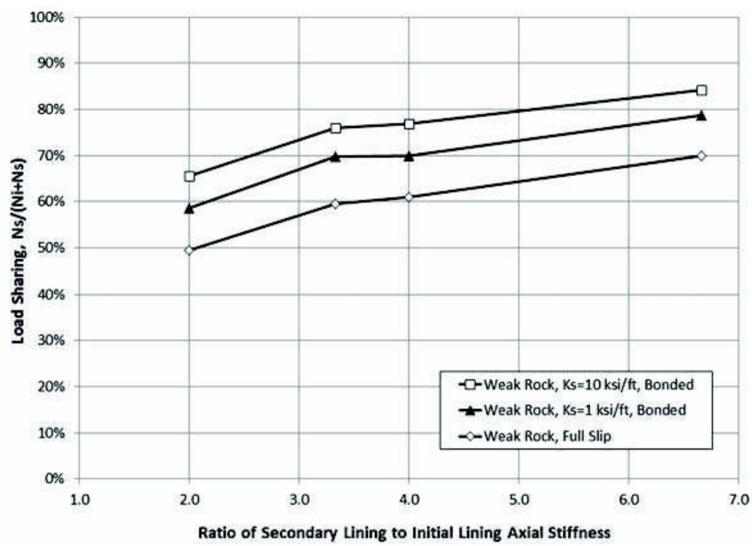
Application of the load-sharing method to the design of NATM tunnels can achieve a significant cost savings for construction of the secondary lining. This method has been employed in several NATM/SEM tunnel projects designed by Jacobs Associates, including the Caldecott Fourth Bore in Oakland, California; the Transbay Downtown Extension in San Francisco, California; and the Ottawa Light Rail Project in Ottawa, Canada. The application of the load-sharing method for the Caldecott Fourth Bore project is discussed in the subsequent section.

Caldecott Fourth Bore Tunnel

The Caldecott Fourth Bore tunnel is parallel to three existing bores along State Route 24 (SR 24) through the Berkeley Hills in Oakland, California. The Fourth Bore is 1,036 m



(a) Rock mass stiffness and strength

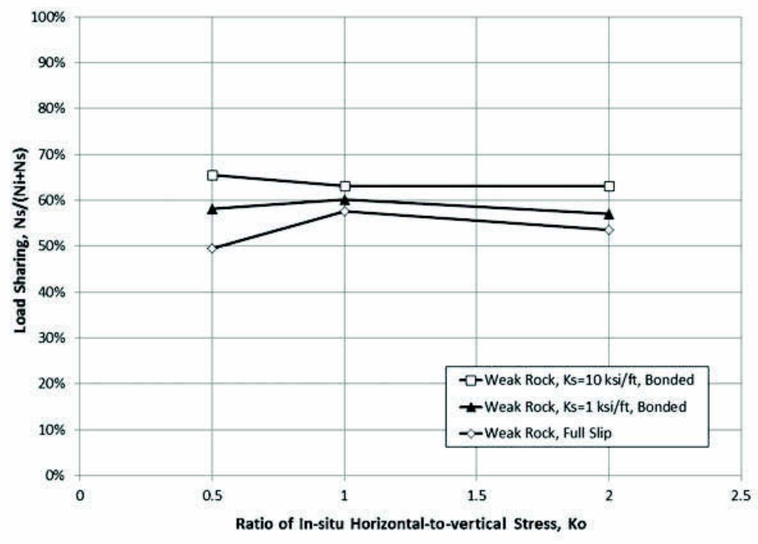


(b) Lining stiffness

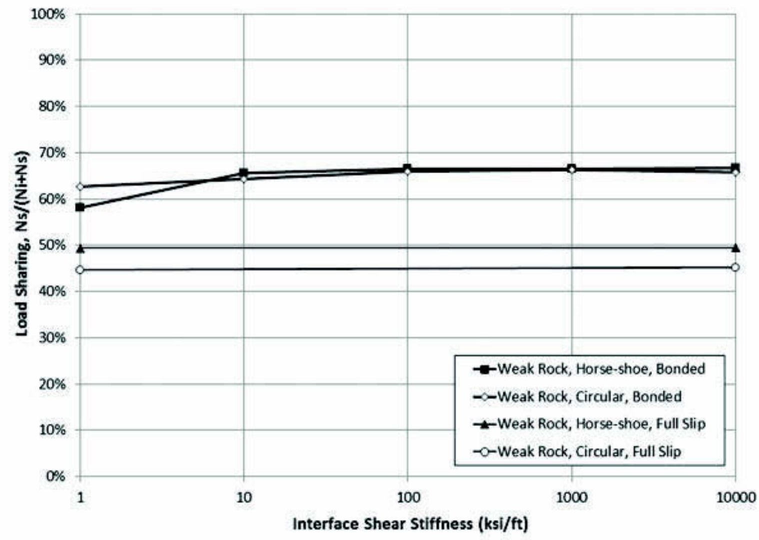
Figure 4. Axial force shared by secondary lining as a function of rock mass strength/ stiffness and lining stiffness

(3,399 ft), and the tunnel includes two 3.7 m (12 ft) traffic lanes, and two shoulders 0.6 and 3.0 m (2 and 10 ft) wide. The ground cover above the tunnel crown varies from about 7.6 to 152.4 m (25 to 500 ft). The horseshoe-shaped mined tunnel is 15.2 m (50 ft) wide and 9.8 m (32 ft) high. A typical cross section of the tunnel is shown in Figure 6.

The geology along the alignment is characterized by northwest-striking, steeply dipping, and locally overturned marine and nonmarine sedimentary rocks of the Middle to Late Miocene age including shale, chert, claystone, siltstone, sandstone, and



(a) In situ stress ratio



(b) Tunnel shape

Figure 5. Axial force shared by secondary lining as a function of in situ stress ratio and tunnel shape

conglomerate. The design mean rock mass properties for two of the rock mass types correspond to the "medium" and "strong" rock types defined in Table 2.

Ground support for the Fourth Bore tunnel consists of an initial shotcrete lining with rock dowels and a CIP reinforced concrete secondary (final) lining (Figure 6). A sheet waterproofing membrane with a geotextile backing layer for drainage is installed between the initial shotcrete lining and the secondary lining. The initial shotcrete lining

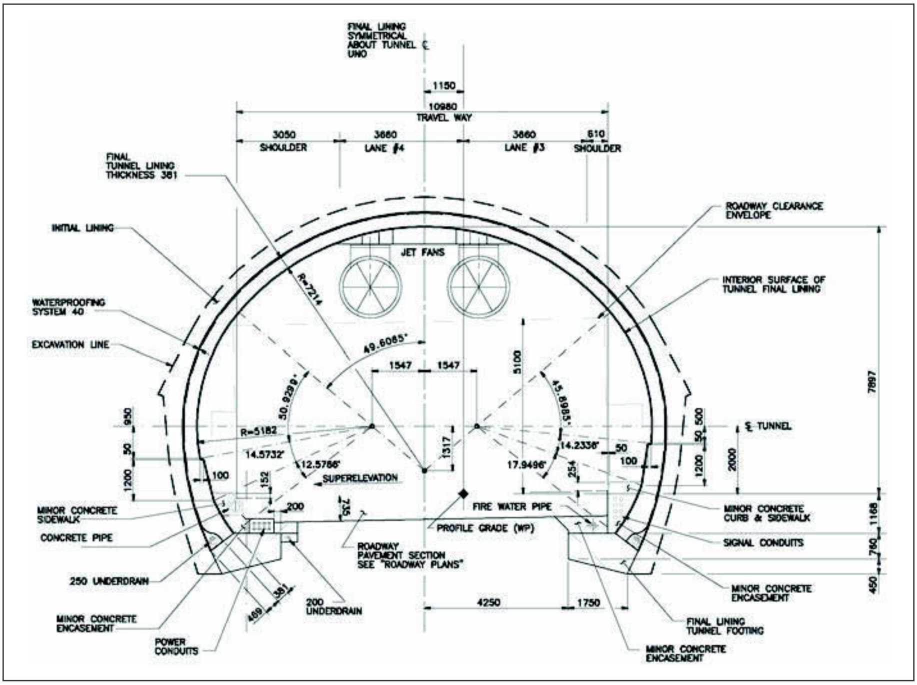


Figure 6. Typical cross section of Caldecott Fourth Bore Tunnel (all dimensions in mm)

has a thickness ranging from 203 to 305 mm (8 to 12 in.). The CIP secondary lining has a thickness of 381 mm (15 in.).

The initial shotcrete lining and the secondary concrete lining are designed as a DSL system. One of the fundamental design assumptions is that two essential components, the rock dowels and the shotcrete lining, are expected to deteriorate with time. In most of the tunnel, the first 51 mm (2 in.) of shotcrete lining applied as a flash-coat are considered sacrificial. In one particular shale formation, the first 102 mm (4 in.) of shotcrete are considered sacrificial because of a high sulfate concentration in the groundwater in this reach. The remaining shotcrete layers are also expected to deteriorate to some degree over time such that, in the long term, the initial shotcrete lining is assumed to have diminished axial stiffness and capacity and no flexural capacity.

Load-sharing analyses were carried out to estimate the magnitude of ground loads transferred to the secondary lining. Results indicated that the secondary lining will attract a maximum of 50% of the ground load originally supported by the initial lining. Given that the Caldecott Fourth Bore is part of a designated lifeline route, the secondary lining was conservatively designed to carry 67% of the ground load originally supported by the initial lining (Thapa et al., 2008). This load-sharing design approach resulted in a reduction of about 30% in the required thickness of the secondary lining. An additional benefit of the reduced secondary lining thickness was the reduction in forces in the more flexible lining when subject to seismic racking deformations.

CONCLUSIONS

Load sharing is considered important for design of tunnel linings. The magnitude of the load sharing between the initial and secondary linings installed as a two-pass lining system in NATM tunnels depends on several factors: the interface stiffness and bond capacity, strength/stiffness of the rock mass, and the relative stiffness of the initial and secondary linings. Other factors, such as, tunnel shape and in situ stress condition, are not expected to significantly affect the magnitude of load sharing. The findings from the investigation presented in this paper can be summarized as follows:

- For an SSL or CSL system, the interface shear stiffness has a significant effect on the magnitude of load sharing. This effect is more pronounced in stronger and stiffer rock. For a DSL system, the magnitude of loads transferred to the secondary lining is not affected by the interface shear stiffness, and is controlled primarily by the stiffness of the rock mass.
- Load sharing is generally not sensitive to the variation in interface normal stiffness, except for the DSL system assuming low interface normal stiffness.
- The magnitude of loads transferred from the initial lining to the secondary lining decreases with increasing rock mass strength/stiffness.
- The magnitude of loads transferred to the secondary lining increases as the relative stiffness of the secondary lining increases.
- The maximum ground load that the secondary lining is expected to support ranges from 50 to 70% of that originally carried by the initial lining. The use of load sharing in the design of a tunnel's secondary or final lining can lead to an efficient design solution and, consequently, result in a savings in project costs.

The results and conclusions presented in this paper represent the findings based on the assumptions and limited range of parameters used. Further investigations that include a broader range of parameters related to the interface stiffness, tunnel shape, lining stiffness, ground and other loading conditions, etc. are warranted in order for one to have a comprehensive understanding of load sharing and its implications for the tunnel secondary lining design.

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MASTERING KARST FEATURES AT THE BAUMLEITE NATM TUNNEL PROJECT IN THURINGIA, GERMANY

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ABSTRACT

Deutsche Bahn is currently building a double-track, high-speed railway between Ebensfeld and Erfurt in Central Germany. The 1,317-m-long Baumleite Tunnel had to be driven through limestone, where the sudden occurrence of karst phenomena was a definite possibility. The NATM drive was performed with continuous exploration drills. Several karst structures were in fact encountered, with the largest cavity having a volume of 500 cubic meters. This obstacle was mastered by combining exploration with backfilling, anchoring and grouting works. All activities were contractually dealt with using the so-called "flexible unit price model," which will also be described in the presentation.

PROJECT DESCRIPTION

General

Deutsche Bahn is currently building a new double-track, high-speed rail line between Ebensfeld and Erfurt in Central Germany as part of the "Verkehrsprojekte Deutsche Einheit Nr. 8.1." Due to the Thuringian Forest and its elevations, most of the rail line is built either on bridges or in tunnels. The 1,317-m-long Baumleite Tunnel is part of this project (total length 107 km (66 miles)) and is situated in Thuringia near the city of Schalkau.

Alpine BeMo Tunnelling GmbH (formerly Beton- und Monierbau Innsbruck GmbH) was awarded the contract by DB Netz AG for construction of the running tunnel including three cross-passages, emergency tunnels and a shaft (see Figure 1).

Geology

Baumleite Tunnel had to been driven through limestone with a cover of between 7 and 30 meters above crown. The layers of limestone are separated by thin layers of claystone. The exploratory drillings encountered many voids and much loose material, especially in the southern part of the project. The voids indicate karst structures, while the loose material might have been naturally backfilled into the voids. Some of the exploration drill holes were used to monitor the groundwater levels. In one place the water level showed fluctuations of 17 meters within several days, probably caused by the fissured rock (see Figure 2).

Excavation Method—NATM

The initial excavation and support were performed according to the New Austrian Tunnelling Method (NATM). The cross-section is excavated in three phases: the crown (74.9–81.5 m^2), the bench (48.4–50.6 m^2) and the invert (33.3–38.4 m^2). The client uses different excavation classes that differ in advance length and support measures.

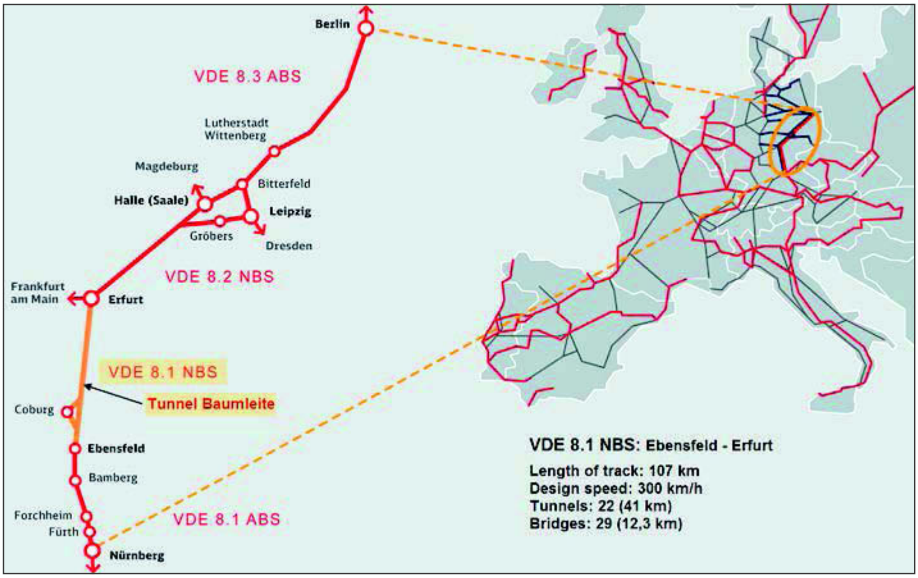

Figure 1. Overview of Neubaustrecke VDE 8 (Source: DB, n.d.)

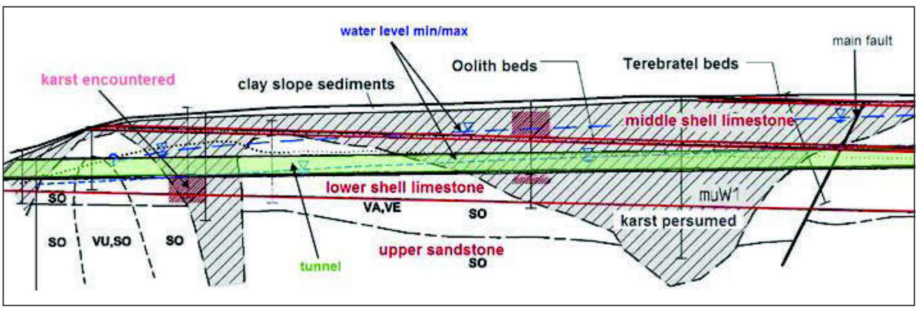

Figure 2. Geology of Baumleite Tunnel (Source: Arcadis, 2006)

The solid rock calls for mainly drilling and blasting, while in small areas mechanical excavation can be performed (backhoe). After mucking, the support is built in and consists of two layers of mesh, a lattice girder, shotcrete and anchors. Depending on the geology, the strength of the support varies and additional measures, such as spiles, are used to secure the top heading in the subsequent advance.

An example of NATM applied successfully in difficult hydro-geological conditions can be seen from the Egge Tunnel project (Fuegenschuh & Arnold, 2003).

At each advance the open rock and the finished shotcrete are recorded with a tunnel scanner to ensure correct profile and shotcrete thickness. A very important tool for ensuring safe working conditions and the right choice of advance rate and support is the recording of settlement. Normally measuring cross-sections are placed every 20 meters, in more critical cases at intervals of only 3 meters, and measurements are recorded daily, if necessary even more frequently. The data are processed by the onsite surveyor and then depicted in graphs. For the different excavation classes trigger levels are set by the designer. If the trigger levels are exceeded, additional support is needed in the affected area and the excavation class has to be adapted (see Figure 3).

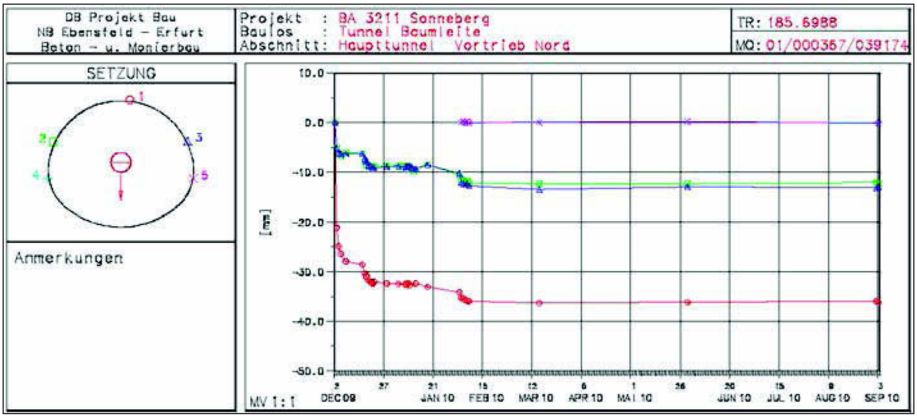

Figure 3. Graph of settlement at Station 367 (Source: ABT, 2010a)

After settlement of the outer lining has ceased, the inner lining can be built. Waterproofing is ensured by a 2-mm plastic membrane (FPO-PE) that on the shotcrete side is accompanied by a 1,000 g/m^2 fleece. The subsequent concrete inner lining is separated into the invert and the vault. Both parts are reinforced with an average steel quantity of 85 kg/m^3. The thickness of the invert ranges between 60 and 75 cm, that of the vault between 45 and 60 cm. In areas where the water level exceeds 30 meters the inner lining is also executed as a waterproof concrete construction by using a different concrete mix and a sealing joint construction (see Figure 4).

Contract

The contract is based on a unit price model. In the bill of quantities all works are described and subdivided into several positions and separate cost codes. The price of each cost code multiplied by the forecast quantity gives the volume for the respective cost code. The sum of all cost codes gives the total contract volume, which is the only criterion for award. The final accounting is drawn up on the basis of the quantities actually measured and performed during the project.

Construction Period

The bid invitation (call for bids) contains tables in which the bidder must enter the construction time he deems necessary to perform the contract within the time frame set by the client. A fixed time rate is used when the work is precisely described and changes are not expected, i.e., planning, site equipment, extra time for completion of tunneling works after the crown heading is broken through and the inner lining works and site clearance have been finished. A flexible time rate is used for the excavation time, karst exploration and, for steps required to cope with karst conditions. Table 1 shows the schedule for the heading, whereby the bidder had to complete the grey shaded areas. A similar table for the karst exploration and karst measures also had to be filled in and handed over to the client as part of the bid. Moreover, the hours calculated per unit had to be estimated and entered for each cost code containing tunneling works.

Accounting and Flexible Construction Period

For the different excavation classes a "meter-excavation" price is set that includes the excavation work and a predefined amount of support, such as anchors, spiles, mesh, lattice girders and shotcrete. If the amount of support needed during execution varies,

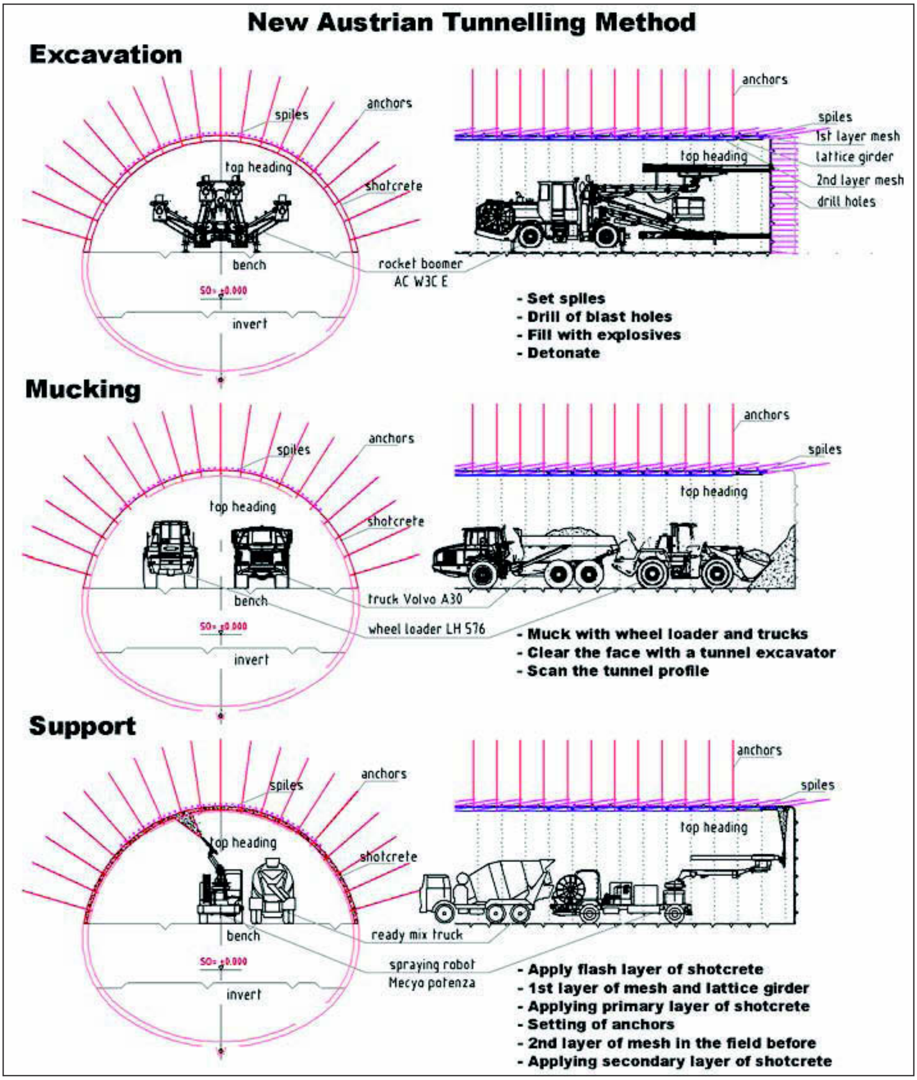

Figure 4. NATM sequence (Source: ABT, 2012)

the accounting will be adjusted. If less support is needed, an amount will be subtracted, and if more is needed the amount will be increased. If the shotcrete thickness of the lining is changed, e.g., from 25 to 30 cm, the additional excavation will be added according to the model.

Another big influence on the cost is the available project time. Construction time is normally calculated according to performance in each class and the actual distribution per class. The particular feature of this contract is the fact that all changes in driving performance cause changes in the available project time. As an example, Table 2 shows the result in the column “target time (days)” (23.46 days). In this contract the time influence of additional or less work is calculated. The basis is the target hours for the advance work (3,378.3). The number of available hours is calculated from the

Table 1. Bidding table for heading, simplified and with presumed performance rates (Source: DB, 2008)

Heading Class	Advance Length (m)	Distribution (m)	Performance (m/day)	Time (days)
K 4.3	1.3	70.00	7.8	8.97
K 4.4	1.0	70.00	6.0	11.67
K 6.1	1.3	151.00	6.5	23.23
K 6.2	1.0	275.00	5.0	55.00
K 6.3	1.0	448.00	4.0	112.00
K 6.4	1.0	200.29	3.5	57.23
K 6.5A	1.0	50.00	3.0	16.67
St 4.5a	2.0	200.29	18	11.13
St 4.5b	2.0	50.00	16	3.13
S 4.1	6.0	262.00	20	13.10
S 4.2	4.0	185.29	18	10.29
Sum				322.41

Table 2. Calculation flexible time (Source: DB, 2008)

Excavation Class/Support Measure	Unit (unit)	Performance (m/day)	Hours per Unit (hrs/unit)	Quantity (unit)	Target Time (days)	Hours (hrs)
		A	B	C	C/A	C × B
K 4.3	m	7.8	18.46	55	7.05	1,015.3
K 4.4	m	6.0	24.00	80	13.33	1920
K 6.1	m	6.5	22.15	20	3.08	443
Total target time and hours					23.46	3,378.3
Shotcrete 25 cm	m^2		0.23	–2,000		–460
Shotcrete 30 cm	m^2		0.31	2,000		620
Excavation class K4.4	m^3		0.22	100		22
SN Spiles 4 m	Stk		0.19	–125		–23.75
IBO Spiles 6 m	Stk		0.3	150		45
Shotcrete	m^3		4.0	10		40
Total actual hours = Total minus/plus and total target hours						3,621.55

variations in performance. The “flexible construction time” is calculated as “total target time (days)” multiplied by “total target hours” divided by “total actual hours,” in our example:

Flexible construction time = 23.46 × 3,621.55 / 3,378.30 = 23.46 × 1,072 = 25.15 d

The new construction time gives new costs for staff, machinery, indirect wages and indirect costs. These costs are thus billed according to excavation work plus all extra works affecting the construction time.

The same method is used for the karst structures in an effort to create a fair billing system for this basically unpredictable work. Figure 5 is a schematic illustration of a karst system. If such a situation is encountered, the main position will be billed and adjusted for all additional works. For bigger events various karst features can be mixed.

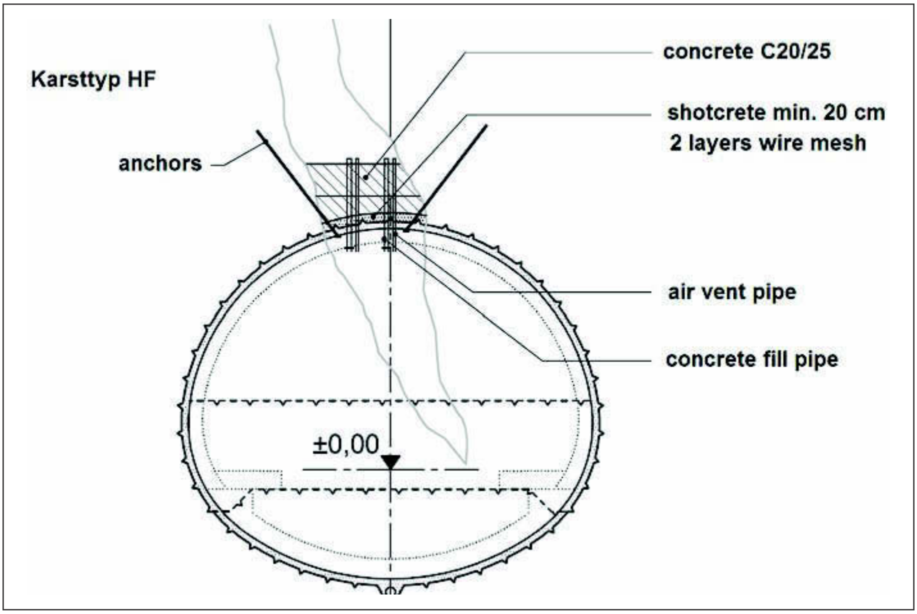

Figure 5. Karsttyp HF (Source: ILF, 2008)

Application of the Contract Model

During tunnelling the contractor and the client agree on the excavation class with a fixed advance length and on fixed (e.g., shotcrete thickness) and flexible (e.g., number of spiles) support measures. The result is a RESS (Required Excavation and Support Sheet), which is agreed upon and signed by both partners.

The RESS will not change as long as the geological conditions do not change. At the tunnel face the lead miner and a representative of the client agree on the flexible support measures needed for the geological structures within the borders given by the RESS. When karst features are encountered, the construction management and the client must decide how to deal with the karst using the karst types given in the contract.

PROJECT EXECUTION

Standard Advances and Karst "Exploration"

In addition to the RESS, a second important tool was the so-called karst exploration sheet. It normally consisted of five to six drill holes, diameter 50 mm, 12 m long, inclined 20° down to the horizontal and drilled through the base level of the respective excavation area to make sure no major voids were located under the forthcoming work area. These steps were repeated every 5 meters. The area above the tunnel excavation was continuously explored using the drill holes for the spiles, normally 35 to 40 holes, each 4 m long. The lateral areas of the tunnel excavation were explored using the drill holes for the anchors, normally seven to eight holes, each 4 to 6 m long. The blasting holes in the tunnel face were used to get further information on the karst features.

All these drillings were performed with the Rocket Boomer without making any equipment modifications. If drill holes had shown major voids, very soft areas or clearly visible karst structures, the exploration program could have been extended. To provide for this eventuality, the contract included a variety of additional drillings, such as drill holes up to diameter 150 mm and 15-m-long or rotary drillings.

Figure 6. Sealing the opening and the void at TM 580.01 (Source: ABT, 2010b)

Figure 7. Surveying the void at TM 580.01 (Source: ABT, 2010b)

Encountering a Karst Structure

Encountering and Exploration

The face at Station 580.01 in the top heading looked very good and stable, the spiles and the anchors had been drilled into solid ground. But when drilling the blast holes, voids were encountered on the right side of the tunnel face. The regular karst exploration system was put into place to check the area beneath the tunnel invert. The drillings confirmed a void or soft ground there. Work commenced normally, but no explosives were put into the drill holes close to the void. Detonation caused a small opening in the tunnel face, showing a large void behind it. The client and the construction management were informed immediately. Jointly they decided that the current advance had to be terminated and the area around the opening stabilized, meaning all loose material at the tunnel face would be removed and the whole area sealed with a layer of shotcrete. The void was surveyed and graphically analyzed. See Figures 6 and 7.

The top of the void looked very stable, but the main concern was the invert because it looked like the detonation might have caused loose material to collapse and fill the entrance to a lower, possibly much larger, void. From the face five rows of ten exploration drillings 12 m deep were made. Voids and loose material were encountered at the right side of Station 580.01. After that, a second exploration was performed at Station 580.01 in order to check the whole area around the tunnel excavation in all directions. The result was alarming because many of the drill holes at the opening showed either voids or loose material all the way to the end of each drilling. Due to this massive problem excavation was shifted to the bench, thus allowing time to develop a proper solution involving the designer and the structural engineer.

To confirm the void and obtain further information it was decided that at the face three rotary drillings, each 12 m deep, inclined 30° down to the horizontal, had to be performed. Depending on the circumstances, cores or loose material would be collected and, if a void was encountered, camera inspection would be made. The work was difficult because the material was very inconsistent. Two drillings showed solid

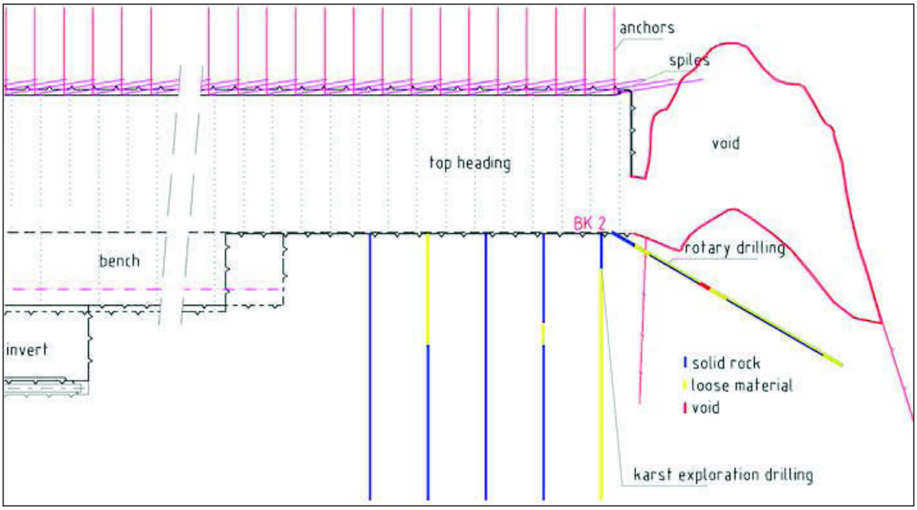

Figure 8. Exploring the void at TM 580.01 (Source: ABT, 2012)

ground for the first 2 meters, followed by loose material and voids to the end of each drilling. The third drilling showed loose material only in the middle section of the hole. Because the drilling ended in loose material, camera inspection failed. We concluded that we knew where the disturbed area began, but not where it ended. See Figure 8.

Processing the Karst Structure

To ensure a safe heading it was decided that the void had to be filled and the ground under the invert stabilized. At the tunnel face a concave wall was erected using wire mesh, rebar and shotcrete. Concrete fill pipes and air vent pipes were set in the wall to ensure a safe fill rate. The void was filled to the very top in stages with regular concrete, using a total of 304 m^3. See Figure 9.

To consolidate the loose material in the invert it was necessary to fill the whole area with stabilizing material. In addition, the area behind the lining at the void had to be checked. The agreed solution was to use grouted self-drilling anchors (IBO). The setup in the invert consisted of three rows, each having 12 IBO anchors between 4 and 10 m long inclined 45° to 60° down to the horizontal, to reach a wide range of the disturbed area. For the side and top three rows of 8 IBO anchors, 6 m long, radial spread, were used. The anchors were filled with a cement water value between 0.5 and 0.7 using an Obermann injection device. The stop criterion was either 2.000 kg of grouted material per anchor or a pressure of 5 bars. In a second and subsequent step all anchors that did not reach the stop criterion would be filled again. The radial anchors showed minor grout intake, namely a total of only 4.200 kg. This confirmed that this area was only slightly disturbed and now sealed. In the invert 17 of 36 anchors showed a pressure rise exceeding 5 bars in the first step; all the others were filled with 2.000 kg each. In the second step six additional anchors showed a pressure rise of more than 5 bars, the others showed no pressure build-up. These two steps called for a total of 86.700 kg of cement. The intense costs and great time needed caused us to abort this method. See Figure 10.

A new method was needed to fill the voids faster and with a faster-setting material. The solution was a reversed pipe umbrella ALWAG AT 114.3 × 6.3 mm with perforated pipes, 8 spread pipes, inclined 45° down to the horizontal. The first drilling was very problematic because the disturbed area prevented the drilling water from flushing out

Figure 9. Closing the void at TM 580.01 (Source: ABT, 2010b)

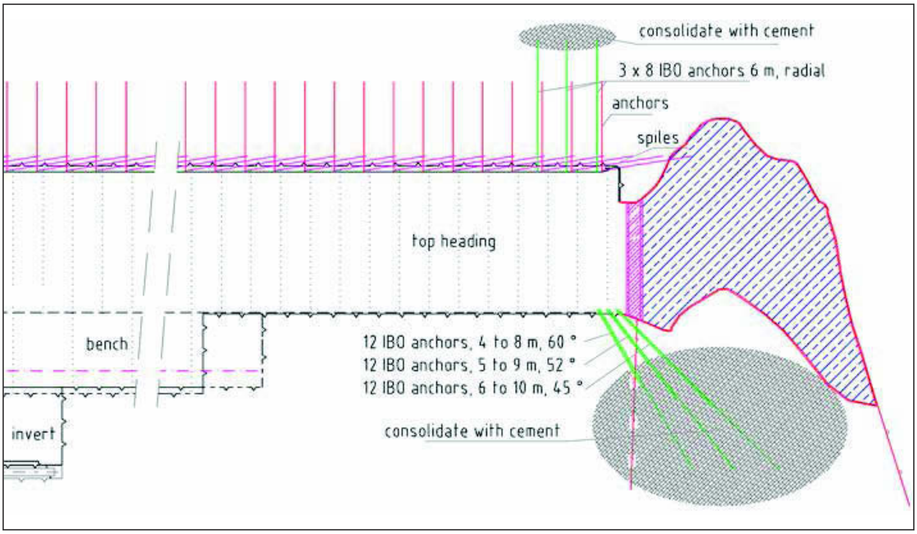

Figure 10. Consolidating material with cement (Source: ABT, 2012)

the drilled material. The crown consequently silted up, which made retrieval of the pilot crown very difficult. Thereafter, drilling progress was reduced and extra water added through the pipe, which improved the situation. After the drillings were successfully completed, a mortar mix (cement—sand 0/2 mm—water) was filled into the pipes. The mortar was batched in the batching plant and transported with a ready-mix truck to a Meyco Spraying Robojet, which in this case was used as a regular concrete pump. The fill rate was up to 9 m^3/h and a total of 140 m^3 mortar was used. See Figure 11.

Heading

To ensure a stable bedding for the lining, a crown surface was added 3.9 m in front of the face. After that the heading commenced at an advance of 1.0 m/round with crown surface, extended anchor length 10 m and 8.0-m-long spiles. After three advances the invert was stabilized with four reversed pipe umbrellas filled with 38 m^3 of mortar and

Figure 11. Filling the invert with mortar through a pipe umbrella (Source: ABT, 2010b)

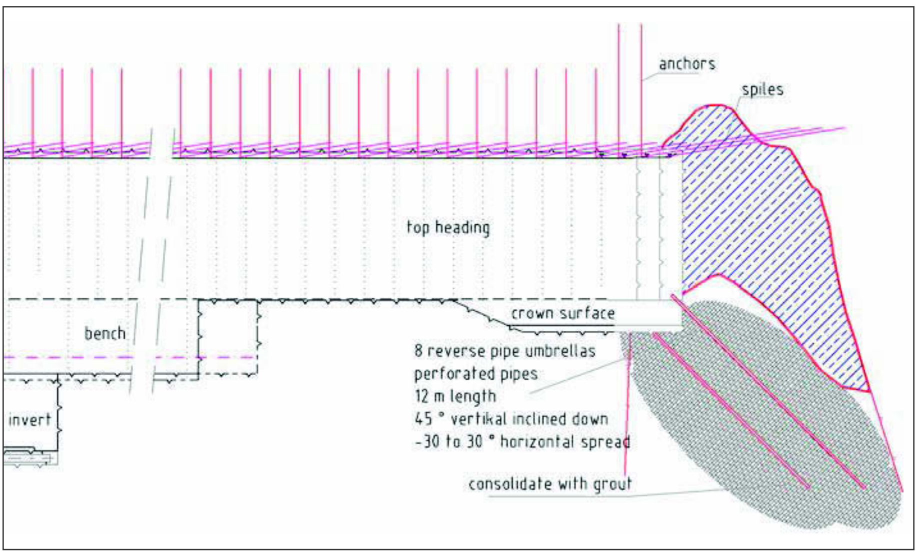

Figure 12. Consolidating material with grout and heading through the filled void (Source: ABT, 2012)

one row of anchors adding another 11,700 kg of cement. The subsequent advances were performed with no major difficulties, the ensuing stabilization of the invert showed no intake of grout material. At Station 592.01 tunnel driving continued according to plan. See Figure 12.

The advances in the bench and invert were expected to be difficult. However, due to the extensive filling work performed in the top heading the excavation progress was performed without noteworthy disturbances.

CONCLUSION

The flexible combination of NATM drive with provisions for karst exploration and support was the ideal way to handle the geology at Baumleite Tunnel. Despite the difficult circumstances involved with the karst structures, the problems were managed professionally and in a very constructive manner between the client and the contractor. The contract model chosen by the owner proved to be a good means of handling the actually encountered situations in a flexible way. However, close cooperation between all involved parties and quick agreement on steps to be taken were the key factors in the success of this project.

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SEQUENTIAL EXCAVATION UNDER NORTHERN BOULEVARD AND TWO SUBWAY STRUCTURES

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INTRODUCTION

Northern Boulevard Crossing (NBX) is the keystone portion of the massive $8.4+ billion East Side Access (ESA) Program by the Metropolitan Transit Authority Capital Construction (MTACC) to bring Long Island Railroad trains into and out of New York City's Grand Central Terminal. NBX constitutes the link between the Queens Tunnels and Manhattan Tunnels portion of ESA. The crossing consists of 120 FT of 2,000 square FT cross sectional area tunnel through soft ground, constructed utilizing Sequential Excavation Method (SEM) under the protection of a structural frozen arch.

NBX extends under an active five track wide subway box, a major six lane highway, as well as through the foundation piles of operating elevated subway line structure. All of the above presented a unique challenge to the owner and contractor. NBX is the first and only SEM tunnel project within the five Boroughs of New York and was constructed by a Joint Venture of Schiavone Construction Co., LLC and Kiewit Infrastructure Co. (SK)

Construction on NBX started in February 2010 with the SEM excavation starting in April of 2012 following access chamber excavation, grouting and ground freeze. Excavation was completed in November 2012 and was followed by PVC waterproofing installation and final lining concrete.

This paper describes lessons learned from the SEM excavation and support, description of the freeze operation is covered in separate papers.

GEOLOGICAL SETTING

The project site is located near the physiographic province boundary between the Manhattan Prong of the New England Upland and the Atlantic Coastal Plain. The site is underlain by Ordovician/Cambrian Age metamorphic bedrock, which is covered by Pleistocene glacial and interglacial deposits and post glacial deposits.

The glacial deposits can generally be divided into three groups, mixed glacial deposits (which include varved clays), glacial till, and outwash/reworked till deposits. Each group can be further subdivided into several strata.

Stratification is generally complex, and significant variations in the thickness and location of the individual units are common. Boundaries between strata are not clearly defined in many cases and considerable interlayering of the glacial materials, particular the mixed glacial deposits is observed. This heterogeneity is typical of glacial deposits found at the rear of terminal moraines. Under such environments, different processes of deposition occur during cyclical periods of advance and recession of the ice front. Prior deposits are sometimes reworked, and new materials are deposited.

SUBSURFACE CONDITIONS

The tunnel envelope of the NBX tunnel is predominantly within Stratum Four of the Mixed Glacial Deposits. Stratum Four consists of brown, grey to olive brown, medium stiff to hard non plastic to low plasticity silts and clays. The stratum was predominantly varved with fine micaceous sands and fine gravel. Gravel, cobbles and boulders were also observed. The Unified Soil Classification System (USCS) group symbols are generally ML to CL. Parts of the crown encountered Stratum Three consisting of grey to brown and olive brown, very loose to very dense coarse to fine micaceous sands and silts with gravels. The USCS group symbols for Stratum Three are generally SM or ML. Stratum Five and Six were encountered in the invert above the rock interface and consisted of brown, grey to reddish grey/brown, medium dense to very dense sands with silts and green to grey, very stiff silts and clays respectively. Both contained boulders.

The rock interface sloped up from south east to north west from an elevation below invert excavation line, to elevation 244.6 FT and thereby encroached the excavation by over four feet. Bedrock was predominantly fine to coarse grained, unweathered to moderately weathered, strong to very strong gneiss and schistose gneiss of the Hartland Formation. The Rock Quality Designation (RQD) were in most cases above 70%.

Groundwater in the project area ranged from elevation 303 FT to 309 FT. The crown excavation line was 280 FT.

TUNNEL DESIGN

The tunnel was designed by the General Engineering Consultant (GEC) for MTACC. The design incorporated a frozen soil arch that served as pre support and water cut-off. The frozen arch had to span the entire length of the tunnel from the slurry wall on the east side of Northern Boulevard to the slurry wall on the west. The frozen arch had to be socketed into the bedrock to isolate the soils below the arch from any water infiltration and allowing for drainage of the soils inside the arch. See Figure 1.

Full drainage of the soils were required prior to removal of the slurry walls temporary bracing in front of the tunnel portal, and to ensure stand up time of the soils during tunnel excavation.

Excavation Sequence

Originally the design consisted of three over three drifts. During final design the two center drifts were split into a top heading, bench and invert. Round lengths were four feet for all the upper drifts and eight feet for all lower drifts. See Figure 1 for numbering and layout of the drifts. The designed sequence of construction was as follows:

- Excavate and support Drift 1
- Start Drift 2 excavation and support after Drift 1 is 35 FT to 40 FT ahead of Drift 2
- Excavate up to 16 FT of Drift 5and place temporary invert after Drift 2 excavation is advanced at least 35 FT
- Commence Drift 3 upon completion of 16 FT long temporary invert at Drift 5
- Commence Drift 4 upon completion of Drift 2, and stagger behind Drift 3 by 35 FT to 40 FT
- Recommence Drift 5 excavation and support when Drift 3 and 4 are completed
- Commence Drift 6 excavation when Drift 5 is at least 35 FT ahead
- Remove temporary sidewalls

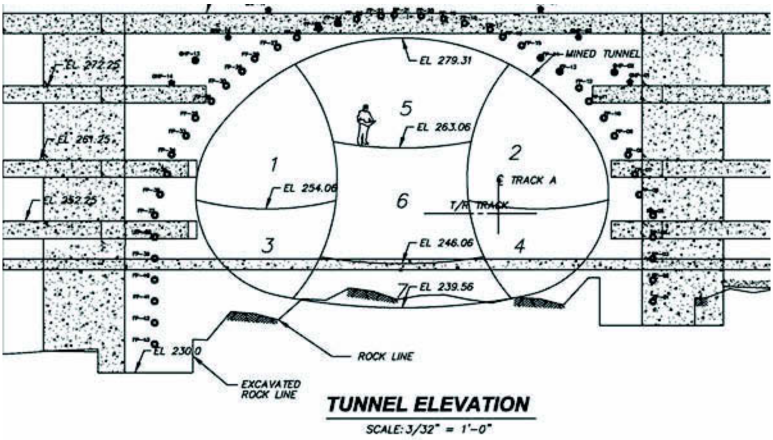

Figure 1. General layout and freeze pipes

- Commence Drift 7 excavation, remove the remaining temporary sidewalls and place invert
- Alternate between Drift 6 and 7

Ground Support

The permanent initial lining consisted of a three inch insulating shotcrete layer and 12 inch shotcrete reinforced with two layers of 4X4 D4XD4 Welded Wire Fabric (WWF). Lattice girders were required at four foot centers, equal to the round length. The temporary sidewalls had the same reinforcement with a 12 inch total shotcrete thickness. Temporary inverts between the upper and lower sidewall drifts were nine inches of shotcrete reinforced with one layer of WWF. Specified shotcrete strength was 100 psi in one hour, 500 psi at six hours, 1,800 psi at 24 hours, 3,500 psi at seven days and 5,000 psi at 28 days.

The tunnel design incorporated convergence measurements during construction to validate the design. 1.2" of settlement at the crown and 0.8" of convergence at spring line was anticipated.

LESSONS LEARNED

Freezing

The frozen arch was accomplished by circulating brine at –32°C through 45 horizontal freeze pipes. During the freeze development two "windows" were identified and corrected through Tube A Manchette (TAM) grouting and by balancing the groundwater elevation inside and outside of the frozen arch.

These additional measures prolonged the freeze duration before the excavation commenced and resulted in extended growth of the freeze inside the tunnel excavation profile.

A larger frozen area within the face of each drift provided for extended stand-up time but extended the excavation duration per round significantly.

Since the tunnel was excavated in the warm summer month, the freezing subcontractor chose to insulate the slurry wall around the brow of the tunnel together with installing freeze hoses directly on the slurry wall to maintain the interface between slurry wall and soil in a frozen state.

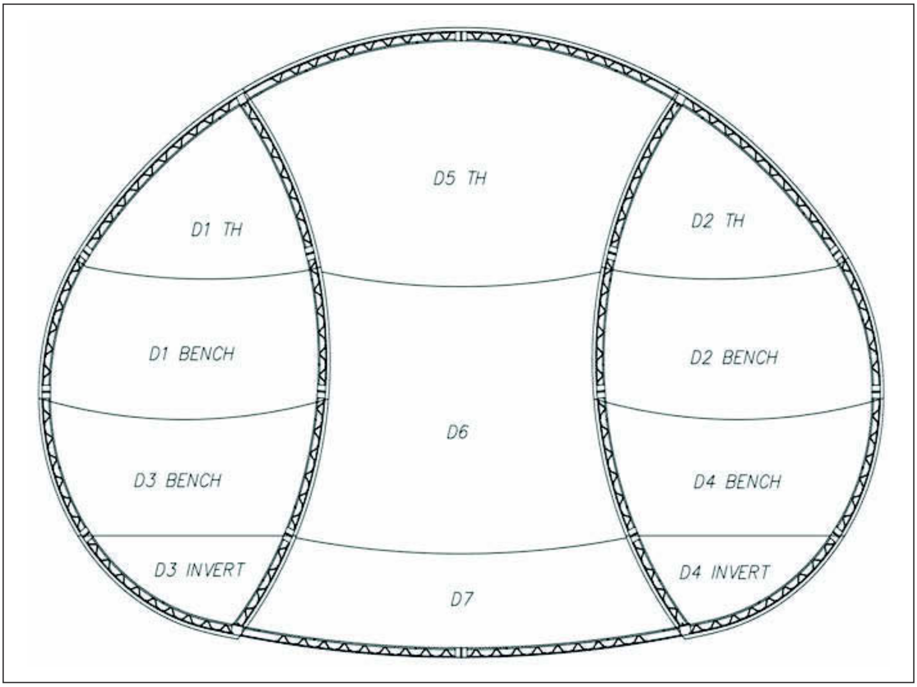

Figure 2. As-built drift lay-out

The freeze plant was operated 24 hours a day, seven days per week.

Dewatering

To drain the soils below the frozen arch following closure of the freeze, three deep longitudinal vacuum wells were installed in the lower part of the face inside the arch. Due to conflicts with structures and high rock elevations in front of the tunnel portal it was not possible install the wells below invert elevation. As the freeze developed the two outer wells were impacted by the freeze and did not function properly.

In order to maintain an equilibrium between the groundwater elevation inside and outside the frozen arch the wells were used to inject water during the final stages of the freeze development. The groundwater elevation in the core inside the arch dropped below the deepest well once the freeze was fully closed and the recharge turned off. It is suspected that the water drained at the rock to slurry wall interface or through fissures in the rock into the deep shafts used to access the lowest freeze pipes next to the slurry wall.

Through the drilling of several probe holes, prior to start of tunnel excavation, there was no indication of groundwater inside the core. However, the silty fine sands and silts in the lower part of the excavation would not drain sufficiently by gravity alone and self-drilling vacuum lances were implemented to increase the standup time of the soils. A flow of less than 0.9 GPM from three vacuum points was sufficient to eliminate any flowing soil conditions throughout the excavation.

Required Excavation and Support Sheet (RESS)

A meeting was held every day in the morning with representatives of the owner and contractor to discuss any safety or quality concerns, production the previous 24 hours, conditions encountered, performance of the instrumentation and plan for the upcoming 24 hours.

The owners design representative was present at the meetings and decisions were made with regard to alteration of sequence, additional ground support measures etc. as dictated by the actual conditions encountered.

Any changes were recorded and signed off on the RESS by designer, contractor and construction manager.

The process provided flexibility during construction and allowed for the tunnel to be excavated in the most expedient and technically correct manner.

Heading Sizes

After observing the soil conditions during the access shaft excavation there was a concern of the full face stand up time in the upper sidewall drifts. Since the designers finite element analysis also indicated three inches of face movement, S/K decided to implement a Top Heading and Bench approach. See Figure 2. Concerns over the miners safety working next to the vertical face in the lower sidewall drifts prompted a division of these headings into a bench and invert. The benches in the lower sidewall drifts were excavated the full length of the tunnel with the addition of a temporary shotcrete invert, prior to excavating the invert. Since the possibility of subdividing the headings was evaluated as a contingency measure in the planning phase, construction joints in the girders had been strategically placed to accommodate the division.

Sequence

Drift sequence and stagger between headings were altered during construction to accommodate the conditions encountered.

During turn under of Drift 1 Bench S/K encountered flowing silty fine sands; to mitigate any excavation delays vacuum wells were installed and Drift 2 was started ahead of schedule. This resulted in Drift 2 being the leading upper sidewall drift. After Drift 2 completion the first 16 FT of Drift 5 was completed as originally planned.

Drift 4 Bench then lead Drift 3 Bench. The Invert of Drift 4 started prior to the invert in Drift 3 but was slowed down due to rock excavation, therefore Drift 3 Invert passed the invert in Drift 4 and the crews commenced Drift 5 following Drift 3 completion prior to Drift 4 Invert completion.

Drift 6 & 7 were completed following the original excavation sequence apart from demolition of the temporary sidewalls. Sidewall removal took place in 30 FT long sections after the ring closure in Drift 7, versus the design sequence of removing them in eight FT sections before closing Drift 7 Invert.

Shotcreting

Compatibility testing and early set test were conducted on several accelerator sources prior to selecting a supplier. This was followed by yield tests and strength testing in the lab.

Since shooting shotcrete panels have nothing in common with applying shotcrete in a tunnel environment around lattice girders and WWF, it was agreed with the owner to eliminate shotcrete panels as the method of verifying the nozzle men's skills and instead construct a full-size mock up section of an upper sidewall drift. See Figure 3. This allowed S/K to evaluate the nozzle men's capabilities to maneuver the robot and the nozzle around the girders and see if they understood the sequence of application. One primary and one back-up nozzle man was approved for each shift.

Shotcrete panels were shot for verification of the mix performance both pre and during construction. Early strength testing was done using penetration needle and the powder actuated nail pull out test.

Figure 3. Mockup of Drift 1

All shotcrete was delivered by a ready mix supplier since it was not possible to set-up an on-site batching facility due to site restrictions. A retarding admixture was added to the shotcrete mix since the trucking time from the batch plant to the site varied from 20 minutes to over an hour, depending on traffic.

Shooting shotcrete on a frozen soil, especially overhead, provide certain challenges that had to be solved. The design required a 3" insulating layer be applied as flashcrete prior to lattice girder installation.

After application of the 3" flashcrete the heat of hydration from the flashcrete thawed the first inches of frozen soil, causing the unfrozen soil and the flashcrete to delaminate from the frozen soil and creating a safety concern. Additionally, on vertical walls where the flashcrete did not fall off, the freezing energy from the ground would refreeze the thawed soil and the flashcrete during lattice girder installation, hence resulting in the structural shotcrete being applied on a frozen surface and defeating the intent of the design.

Since there were no stability issues with the frozen soil requiring flashcrete, it was decided with the acceptance of the owners design representative, to apply the flashcrete layer together with the initial structural shotcrete layer. Additionally, the outside layer of wire mesh was stiffened with reinforcing bars to allow the shotcrete to build up without sagging due to deflection of the mesh.

Due to safety concerns over having crews working directly below freshly applied shotcrete while installing the inside layer of WWF, it was decided to apply all overhead shotcrete in two passes. Using this approach the inside WWF of a round was installed during the outside mesh and girder installation of the following round. Identically the second shotcrete pass of a round was applied immediately after placing the initial pass on the following round. Using this approach the next operation following shotcreting was excavation and, only the stick of the excavator was exposed to potential shotcrete fall-out.

Instrumentation

Automated monitoring of the overlying subway box, the city streets and overhead elevated rail structure together with groundwater elevations was administered by the owner and the results shared with S/K in the daily RESS meetings.

No movement was indicated by any of the instruments during tunnel excavation.

Emergency procedures were in place to notify subway operations prior to tunnel excavation in the event that excessive movement was detected.

S/K had survey crews on three shift who were responsible for checking line and grade of the excavation and lattice girders, in addition to performing convergence monitoring. Only the results from one crew was typically used to ensure the best repeatability in the results. The results were processed on-site by an S/K Engineer trained by the supplier of the convergence software package.

Convergence of the Sidewall Drifts

The same general behavior was observed in the two sidewall drifts. The majority of movement, both vertical and lateral, took place during the bench excavation of the lower sidewall drifts and did not fully stabilize until the invert was closed. The maximum vertical downward movement of 0.9 inches was observed in the quarter arch of the permanent shotcrete lining.

In one convergence array at Tunnel Station (TS) 70 the point below spring line in the permanent shotcrete lining continued to creep following invert closure of the sidewall drift and did not stabilize until excavation and support of Drift 5, the Center drift Top Heading was completed. The point stabilized at the maximum observed lateral movement of 0.9 inches of convergence. See Figure 4.

No movement was observed as a result of the excavation and support of the center drift bench and invert, Drifts 6 and 7, including removal of the temporary sidewalls. The magnitude of movement in the right sidewall drift was generally less than in the left.

This slight difference in convergence behavior between the right and left sidewall drifts could be a result of the stiffer tills sitting immediately on top of the rock in Drift 2 and 4, the shotcrete lining in Drift 4 Invert sits on rock and, due to freeze pipe geometry the frozen mass was larger in the right sidewall drifts excavation.

Drift 5, Center top drift. 0.3" was the maximum amount of vertical downward movement observed in the crown of the center drift. The movement occurred within five days after completing ground support at the measured section and then stabilized. No movement was observed during or after the removal of the temporary sidewalls.

Equipment

Due to the relatively high hourly crew cost in New York City, and to mitigate any schedule risk, back-up pieces of the main equipment was kept on-site at all times.

The main piece of excavation equipment was a 62,000 lbs. tunnel excavator with articulating stick. The excavator was outfitted with a quick connect that allowed for easy exchange of four different tools; grinding head, hydraulic hammer, aggressive bucket with tiger teeth and a bulk excavation bucket.

Mucking was done with a 3 CY track loader.

Shotcrete was applied using a small track mounted robot, except for the invert shotcrete in Drift 7, which was applied using a 2.5" hand nozzle suspended from the stick of the excavator. This solution was required to properly shoot the tight areas at the connection points to the lower sidewall drifts below the temporary sidewalls.

The shotcrete was pumped from the surface to the headings through a 4" slick line using shotcrete pumps with integrated accelerator dosing system. This allows for proper quality control of the accelerator dosing and ultimately the strength performance of the mix.

A dry shotcrete set-up with dry bagged materials was kept as back up for the two wet shotcrete pumps and in the event there was an issue with the wet shotcrete supply.

A 60 FT straight boom, track mounted manlift was used to aid in the lattice girder installation process in the upper drifts and a track mounted air powered drill was used for installation of vacuum well points.

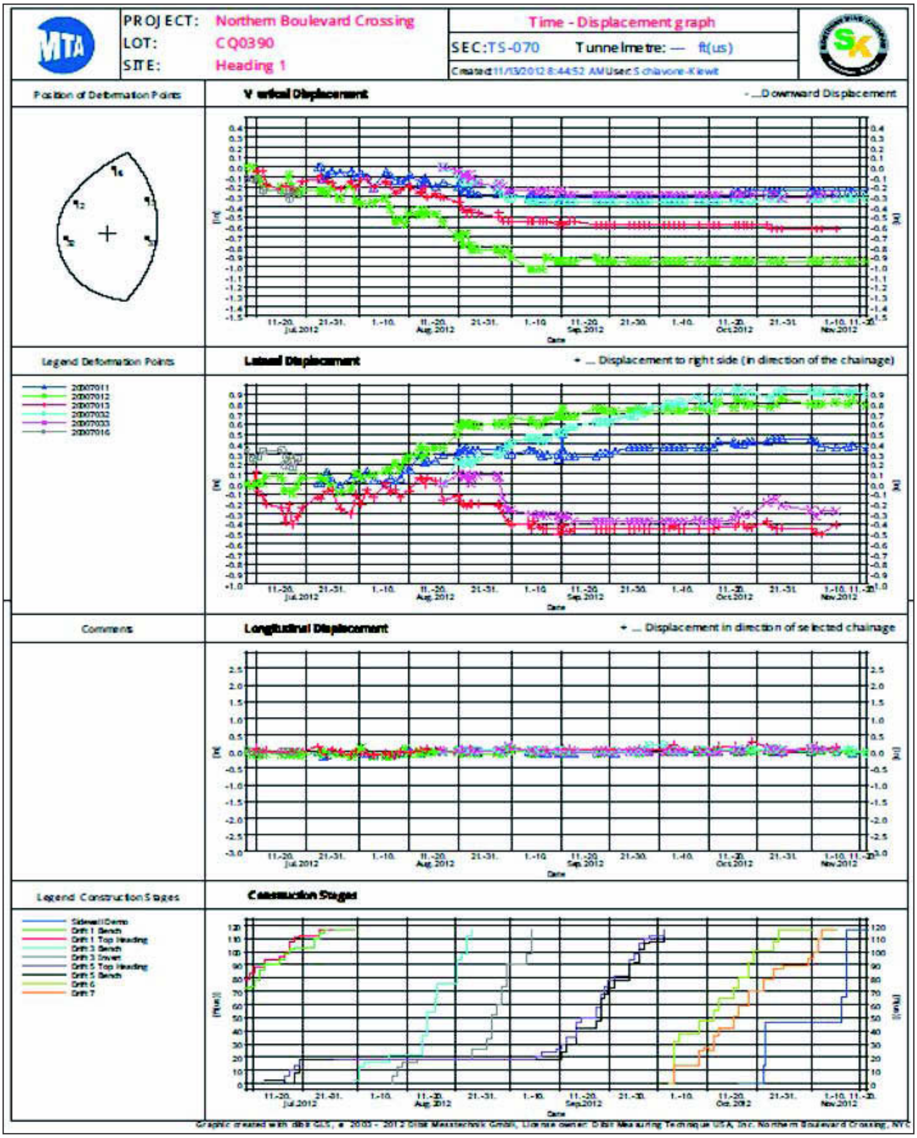

Figure 4. Convergence array at TS 70

To have sufficient reach to demolish the temporary sidewalls a 104,000 lbs. excavator was mobilized. The excavator was fitted with a 360 degree rotating shotcrete shear/pulverizer.

The tunnel operation was supported by a 300 Ton crawler crane and a 5 CY wheel loader to load out muck on the surface.

Crews

None of the miners in the local union had much experience with soft ground tunneling, let alone SEM. However, they had years of hard rock experience amongst them and operated as an experienced team. By having an experienced SEM superintendent on each shift the local New York miners quickly picked up the details and sequence of the work and delivered a quality product. Six miners worked the headings with two operators. The total crew count including mechanics and support was over 20 per shift.

SUMMARY

The Northern Boulevard crossing was successfully excavated and supported through soft soils under major traffic arteries without interruption, utilizing ground freezing, dewatering in conjunction with SEM.

PART 21

Shafts

Chairs

Richard McLane
Trayor Brothers, Inc.

Bob Stier
Kiewit

LAKE MEAD INTAKE NO. 3 TUNNEL INTAKE STRUCTURE AND TREMIE CONCRETE PLACEMENT

Jim Nickerson ▪ Vegas Tunnel Constructors

Jim McDonald ▪ Vegas Tunnel Constructors

James Grayson ▪ Vegas Tunnel Constructors

ABSTRACT

The Lake Mead Intake No. 3 Tunnel and Shafts Project includes construction of a 183 m deep vertical access shaft, an approximately 4.8 km long, TBM-mined, 6.1 m inside diameter segmentally lined tunnel, and a composite concrete and stainless steel intake structure that is placed in 100 m depth of water at the bottom of Lake Mead. This paper discusses the operations related to the setting of the intake structure into the excavated shaft located in the bottom of the lake. The work consisted of placing a guiding frame into the excavated hole to seat the intake structure in the proper position, surveying of the intake location, and placement of tremie concrete around the submerged intake structure.

INTRODUCTION

While the majority of the construction for the Southern Nevada Water Authority's (SNWA) Intake No. 3 project occurs underground, a considerable portion of the project was marine work. The construction and placement of the intake structure was entirely off-shore utilizing two large barges, several small barges, and several work boats. This portion of the work involved unique challenges combining the complexities of standard construction techniques with those associated with building on water. The marine work involved constructing the intake structure on a barge, excavation of the lake bed using shaped charges, and clearing the excavation with an air lift and clam shell bucket from a crane barge. The intake structure was then lowered into the shaft excavation. For additional reference, intake design and excavation has been presented before in the 2011 RETC Proceedings.

The marine work for the Lake Mead Intake #3 project is significant due to the activities that were accomplished for the first time on this project. These activities included the use of shaped charges to sequentially excavate the bed rock in the lake bottom, the completion of work in deep water without the use of deep water divers, and the placement of a large mass of tremie concrete utilizing a dobber tremie system, all in water with a depth of 105 m to 110 m.

PROJECT DESCRIPTION

The Lake Mead Intake No. 3 Tunnel and Shafts project is being constructed by Vegas Tunnel Constructors (VTC), a joint venture created by the parent company Impregilo S.p.A. and its subsidiary S.A. Healy, Company. The intake is to increase reliability of the municipal water supply to the Las Vegas metropolitan area.

Las Vegas receives the majority of its water from Lake Mead and currently accesses the water through two intakes. As of December 2010, after an extended period of drought in the Western United States, Lake Mead had dropped to an elevation of 330 m above mean sea level, compared to the high water level at El. 372 m. If the lake falls below elevation 320 m, the No. 1 intake will become inoperable. The risk of losing access to the lake's water has prompted the necessity for constructing a deeper, third intake.

Intake No. 3 will consist of a 183 m deep shaft constructed near the Alfred Merritt Smith Water Treatment Facility (AMSWTF), an approximately 4.8 km long tunnel, and an intake riser located in the bottom of the lake.

Marine Work Schedule

Since the intake portion of the project is critical to the overall success of the project, the marine work was kept off of the critical path to ensure work associated with the intake tunnel would not be at risk of delay. As a broad overview of the schedule, construction of the concrete portion of the intake structure was completed in December, 2010, and fabrication of the stainless steel riser portion was finished in January 2011. The marine excavation was completed in November, 2011. Then the intake structure was fully assembled and set in place during February, 2012. Placement of the tremie concrete was finished in March, 2012.

Some major milestones of the marine work included:

- Start overburden excavation with crane barge and airlift: Jan 2010
- Start rock excavation using shaped charges: Aug 2010
- Start construction of intake structure: May 2010
- Guide frame positioning: 5 Jan 2012
- Intake positioning and lowering: 17 Feb 2012
- Completion of tremie concrete: 12 Mar 2012

INTAKE STRUCTURE PLACEMENT

The placement of the intake structure consisted of several significant phases of work, including setting the guiding frame into the shaft excavation, lowering of the intake structure, and placement of tremie concrete around the intake structure.

Intake Structure

The design engineering of the intake structure was performed by ARUP with constructability design input by VTC. The final design took into consideration VTC's desire to limit the weight of the structure to make construction more practical. The final structure was about 30 m in height which included a 15 m tall concrete section with a 15 m tall stainless steel riser for the upper section. The finished intake was approximately 1,300 tonnes. The concrete section is about 1,200 tonnes and the stainless section is about 100 tonnes. The concrete portion was cast with 55 mpa concrete and is conventionally reinforced, and the stainless steel section was fabricated from 316L stainless steel with no less than 2.5% molybdenum for corrosion resistance.

The concrete portion of the intake structure was constructed by Contri Construction entirely offshore adjacent to the staging area on barges and other marine equipment provided by Rasmussen Equipment. The construction of the concrete section involved custom formwork, placement of fiberglass reinforcement bars for the TBM eye, and steel reinforcement bars. The intake concrete was placed in five lifts while being suspended over the lake in the moon pool of a barge with a strand jack system supplied by Bigge Crane and Rigging Company. To help maintain buoyancy of the barge and reduce loading, the intake was sequentially lowered during construction of each subsequent section. The stainless steel portion of the intake structure was fabricated by Brown-Minneapolis Tank near Olympia, Washington. It was then transported to the project site where it was erected on top of the previously cast concrete section. The complete intake structure was then transported to its final location in Lake Mead over the end of the tunnel and was lowered into place with the strand jack system on the barge.

Guiding Frame Design

In order to precisely and accurately set the intake structure, a steel guiding frame was used since the sheer size and weight of the intake would make placement of the intake structure extremely difficult. Therefore, it was planned to install a steel guiding frame prior to lowering and setting of the intake structure.

The guiding frame was to be lighter and more maneuverable than the intake structure, making it much easier to set it into the correct position. The guiding frame was designed to support the weight of the intake and ensure proper orientation of the intake. In discussions with a specialist designer (STAC Italy), the following points were considered for the guiding frame:

- The structure needed to be auto-leveling: The structure was equipped with several systems of level control, including spherical high precision bubble levels, with 0.2° of precision and a gyrocompass system (very precise data about true North, pitch and roll) to control the orientation and level once in the deep water, and with 3 independent hydraulic jacks (60 tonne each) with the ability to adjust the position of the frame.
- The structure needed to be robust enough to support the 1,300 tonne structure: The structure utilized heavy steel beams fully welded at each joint (total final weight of the guiding frame was about 70 tonnes). After the guiding structure was set into place, about 500 m^3 of tremie concrete was placed to prevent displacement during setting of the intake structure and to test the tremie concrete delivery equipment.
- The structure needed to be equipped by a system capable of guiding the intake to its final position: The guiding structure was equipped with 4 towers (located on each corner of the guiding frame), equipped with angled plates to move the intake so that it would seat properly on the guiding frame when lowered. The towers stand about 1.2 m above the guiding frame base. To help position the intake in the required position, the system was equipped with 4 cables (one at each tower) that were maneuvered by 4 independent 5 tonne winches. The cables were lowered through sheaves located in the 4 towers of the guiding frame and brought back to the barge on the surface. These cables guided the structure to the frame and allowed movement of the intake as it was seated.
- The structure needed to be equipped with a survey system capable of determining the frame's exact final position: To satisfy this very critical necessity, and due to the fact that we were unable to find an advanced system capable of guaranteeing the precision we needed (to locate the X, Y and Z of the structure within ±10 cm), 4 cigar buoys were attached to the frame using a special reduced elongation cable. The cigar buoys were constructed from 356 mm diameter Schedule 10 steel pipe 6 m long. The wall thickness of the pipe was 5 mm, which gave a net buoyancy force of 300 kg. The buoys were connected to the guiding frame with 107 m of 16 mm Nilspin wire rope. The total length of the cables and buoys were measured, the coordinates (X and Y) of the starting points of the cables were surveyed and, as a consequence, the axis of the intake structure could be determined.

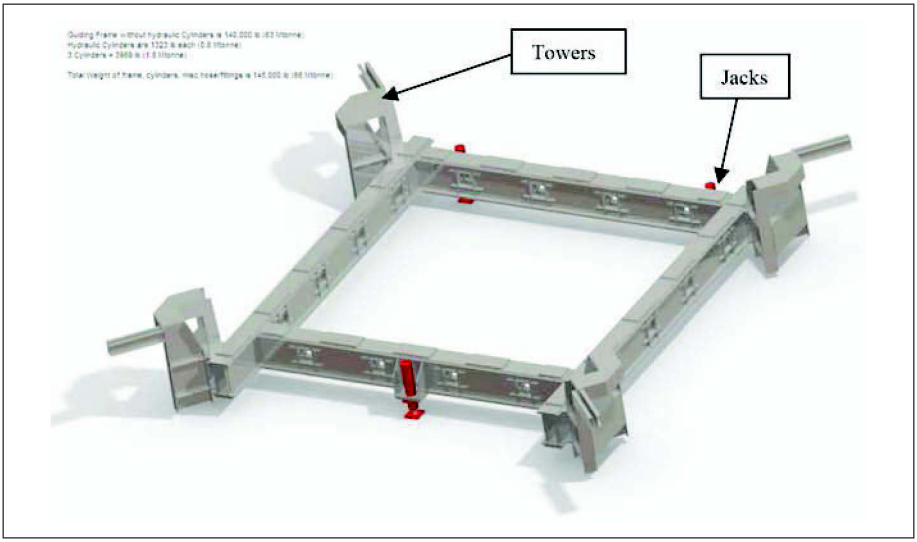

Figure 1. 3D model of guiding frame

To ensure the intake structure would fit between the 4 towers of the guiding frame, the bottom of the intake structure was surveyed during construction and later checked for its dimensions. It was found that the intake structure was within ±13 mm of the design dimensions. The guiding frame towers were then fabricated to match the as-built dimensions of the intake structure (Figure 1).

Final Positioning of the Guiding Frame

Once the excavation of the shaft was complete and the bottom of the excavation was cleaned and ready to receive the structure, the crane barge was prepared to execute the positioning. With engineering assistance from AQUA Engineering, the bow of the crane barge was enlarged from 21 m to 33 m and the Manitowoc 2250 crane was modified from Series 1 (180 ton weight) to Series 3 (about 300 ton) capacity using additional counterweights so it would be able to lift 70 tonnes at a 20 m radius. Additionally, longer cable had to be placed on the crane in order to have enough rope to place the guiding frame on the bottom of the shaft excavation.

The guiding frame was equipped with two subsea levels as well as four connection points for survey buoys. The guiding frame was equipped with all accessories including hydraulic pipes connecting the power pack on the barge to the 3 hydraulic leveling cylinders, gyrocompass connected with transmission cables to the barge, cigar buoys, and a transponder that continuously showed the position during the lowering operation. The frame was equipped with 4 nylon slings rated for 25 tonnes each, terminating to an 80 tonne shackle assembled to a “chair” capable of maintaining the shackle close to vertical so that it could be detached and reconnected with an ROV (provided and operated by Associated Underwater Services) if needed (the system was tested several times before the actual operation; Figure 2).

Before executing the final positioning of the guiding frame, a lightweight test frame was built and positioned to be sure that the bottom of the excavation was relatively flat and wide enough to receive the actual guiding frame. During this test, we mapped the presence of boulders in the bottom of the excavation and, in a few critical areas, we measured the distance between the tops of these rocks and the bottom of the frame.

Figure 2. Guiding frame being lowered, leveling cylinders visible. Frame lowered with cylinders fully extended.

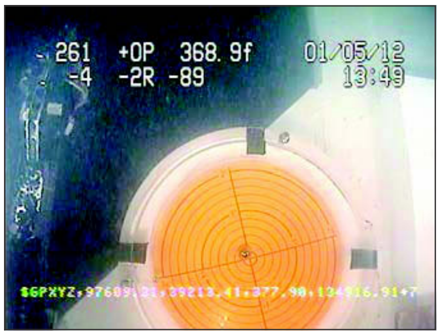

Figure 3. ROV screenshot showing subsea level on guiding frame

We also verified the lateral corridor available between the frame and the walls of the excavation.

A few days after this survey, the guiding frame was then lowered into the excavation. The position was tracked by 2 transponders. Their position was shown relative to the theoretical position on monitors inside the surveyor's cabin on the crane barge. Once the frame was close to the bottom of the excavation, the alignment and position was adjusted based on the information transmitted by the gyrocompass. The frame was laid on the bottom after the alignment of the frame was found to be satisfactory. The frame was leveled using the 3 hydraulic jacks, and then the first phase of survey was performed. This included checking the bubble levels, pitch and roll of the gyrocompass, and location of the cigar buoys (Figure 3).

With the first survey being satisfactory, the load of the frame was released from the crane and the ROV continued its inspections. The ROV checked the contact area between the jack pads and excavated surface, and any contact point between the frame and the other existing materials. All survey information was again checked including gyro, bubble levels, and survey buoys.

A day after the frame positioning, the frame was slightly out of level with one side topped with scattered rock. The rock appeared to have fallen from the edge of the excavation wall and damaged one of the sheaves on the frame. We repositioned the frame using the jacks and verified the pads of the jacks were well settled into the bottom of the excavation. Once all instruments and the ROV inspection showed that the frame was in an acceptable position, we placed 10 m^3 of tremie concrete at each jack using a bottom sealed bucket. Each jack was secured in place to fix the location and avoid any further potential movement. Survey was then executed from land, with the surveyors using

Figure 4. Sealed bucket used to place concrete underwater

a prism positioned on top of each of the cigar buoys. This survey was repeated for several days, with three days of data being shot during optimal weather conditions. Optimal weather conditions were when there was a total absence of wind, lake water was perfectly still, and the current as measured with a floating current meter at various depths showed negligible current readings. This data was used to establish the final coordinates (X, Y and Z) of the Intake. At this point, the gyrocompass was removed and the valves of the hydraulic jacks were closed using the ROV.

After this phase we permanently concreted the guiding frame into place using our tremie pipe/dobber system. This operation was done over the period of several days in order to avoid moving the frame. Approximately 200 m^3 was placed inside the frame using the tremie system. Additional concrete was placed around the outside of the frame using a sealed bucket (See Figure 4) where the space was limited. At the end of each placement, the subsea levels and cigar buoys (See Figure 5) were checked to be certain the frame had not moved.

The buoys were attached to pad eyes located on the four towers of the

Figure 5. Survey buoys attached to guiding frame

guiding frame. In addition to the wire rope, a swivel was used on each buoy and shackles to connect the buoys and cables. All rigging was measured and accounted for during the survey of the guiding frame.

A tower was constructed on the top of the buoys for survey equipment to be mounted. A GPS receiver and a prism for taking shots with a total station were used as two methods to survey the guiding frame. Both methods reported good results and were consistent with each other. The final location of the intake is based on the survey of the guiding frame, which required accurate survey of the guiding frame's elevation orientation, pitch, and roll.

The final coordinates were determined by averaging a collection of many shots. Although the average of the total station shots nearly matched the average of the GPS shots, the GPS shots fell within a tighter range than the shots taken with the total station. Therefore, they were used for the intake's final position.

The survey indicated that the guiding frame was installed at a depth 1 meter deeper than the design. The tolerance of the guiding frame's larger footprint was accounted for in the final survey. The guiding frame allows a possible error of ±25 mm with respect to the intake's actual position.

INTAKE LOWERING

The intake, hanging from strand jacks on the docking barge, was moved into place with tugboats after it was fully assembled. It was transported about 3.2 km from the marine staging area to the final location over a period of about 4 hours. Upon arrival at the lowering location, it was docked with the crane barge which was already on anchor.

Using the ROV, the winches were routed through the guiding frame pulleys with ½" cables on the northeast and southwest corners of the frame. The cables were wound on winches located on the deck of the docking barge then routed down through the guiding frame and back up to an attachment point on the lifting ear of the intake structure. These winches were used to help guide the intake structure into place as it approached the guiding frame during lowering.

The position of the intake structure was monitored with the ROV, a barge mounted GPS system and an Octans 3000 attitude reference unit provided by Ashtead of Houston, Texas. The Octans tracked the pitch, roll, and heading of the structure while the GPS monitored northing and easting coordinates.

Lowering took place over a period of approximately 60 hours due to some delays caused by mechanical problems related to the strand jacks. After the intake structure was docked successfully in the guiding frame, the strands were disconnected from the intake structure. Disconnecting was accomplished by installing eye bolts and a single pin in the Bigge jewels. The main connection pin was pressed out with a 50 ton hydraulic ram (See Figure 6) and the pin was pulled with the crane.

Final position of the intake was verified to be within the tolerance of the design specs. The maximum deviation in northings and eastings was 38 cm difference in the easting of the TBM reception chamber. This final position is fed into the guidance system of the TBM and a small alignment correction will be made as the TBM enters the final length of the tunnel.

TREMIE CONCRETE PLACEMENT OPERATION

After the intake structure was positioned, tremie concrete that was provided by Precision Aggregate Products from an on-site batch plant was placed to secure the structure in its final position. The total volume of approximately 8,300 cubic meters of concrete was placed during March 1–12, 2012. This large quantity was needed to fill the excavation from about elevation 230 m to 244 m. In addition to anchoring the intake structure in place, the tremie concrete was intended to provide cover for the entire

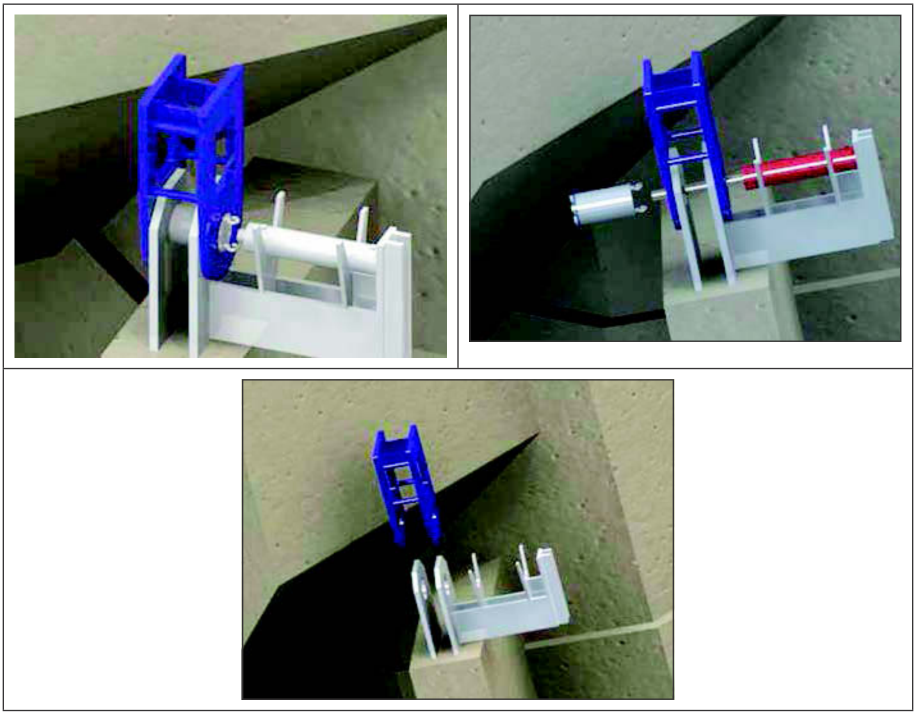

Figure 6. Depiction of the hydraulic cylinder in the working position pressing out the main Bigge Pin and freeing the hardware from the intake structure

length of the TBM shield and two tunnel liner rings (Anagnostou 2010) during the final section of the TBM drive.

The method for placing the tremie concrete is called the “dobber” system and was developed by Rik Pellegrims of Belgium. It consisted of a tremie pipe supported by a “floater” at the water’s surface. The idea was based on making the tremie pipe placing system weightless in water so it could be easily moved around horizontally. The bottom of the tremie pipe had a 3 m diameter plate that allowed the pipe to stay on top of the concrete and glide around the surface of the pouring area.

The tremie pipe used was 25.4 cm diameter and fed by two identical concrete pumping and placing systems that were supplied by Maxon Industries. Each system was capable of delivering approximately 92 m^3 per hour at peak operation. The pumping systems were setup on each side of the crane barge and fed by barges loaded with 8 concrete trucks each. Each truck had a capacity to carry 7.6 m^3 of concrete. The barges and trucks were docked to either the port or starboard side of the crane barge and discharged onto a conveyor which transferred concrete horizontally to an inclined conveyor that deposited the concrete into a re-mix hopper. The hopper controlled the flow into a pump which transferred concrete to the hopper on top of the tremie pipe via a placing boom (Figure 7).

Some problems were experienced during the placement of the tremie concrete, but were quickly resolved. We found that it is important to use a thick walled drop pipe that can withstand the pressures experienced during tremie operations, not only for wear but for stiffness of the column. We also discovered that it is important to always keep concrete as fresh as possible. Any reduction in the flowability of the mix causes serious difficulties for operating the dobber system.

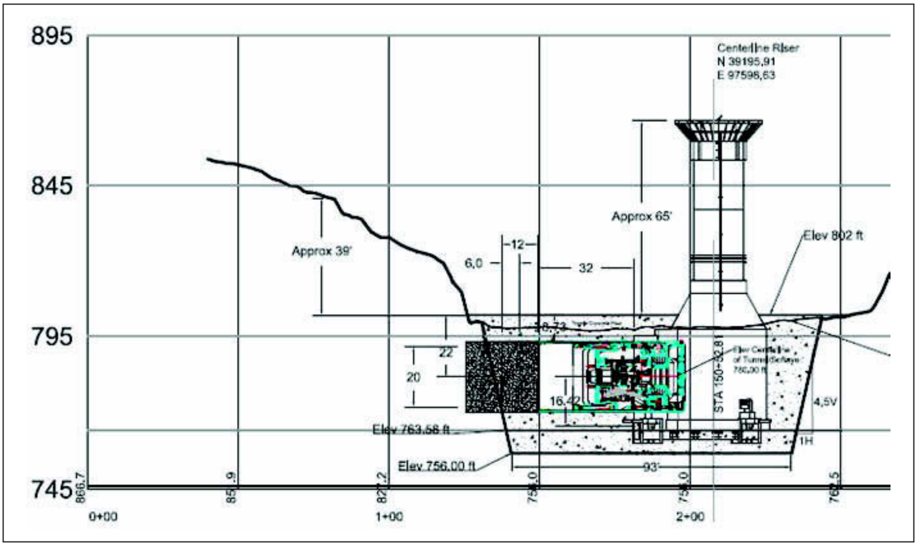

Figure 7. Intake dimensions, intake and guiding frame in final installed position

After a few initial difficulties were overcome, the placement of the tremie concrete went smoothly. Loaded barges were docking with the crane barge on an average of every 80 minutes. Concrete was placed at about 42–46 m^3 per hour when the operation was running without interruption (Figure 8).

The tremie pipe's position at the bottom was tracked by GPS. The ROV also had a landing position 2 m off the bottom of the pipe so the surface of the concrete could be tracked with ROV mounted sonar as manufactured by Mesotech. This allowed the concrete's profile to be monitored graphically on a computer monitor. Final elevations of the concrete were checked with a long tape and confirmed with a multi-beam sonar survey.

Tremie Concrete Mix Design

Because of the many practical and theoretical factors, such as the location of the project, the method of placing the concrete, the depth at which the concrete would be placed, and the engineer's design considerations (See Table 1), the tremie concrete had to be self-consolidating, stable for extended periods of time, slow setting, with low heat of hydration and with minimal washout. To aid with the design of a concrete mix we utilized the assistance of Steve Tatro, who is a consultant with extensive experience in tremie concrete.

Tremie concrete requires self-consolidation to ensure it becomes properly dense without the use of mechanical vibration. Due to the large surface area that was to be covered by the tremie concrete on the Lake Mead Intake #3 project it was determined that the tremie concrete should have the maximum slump flow achievable without causing detrimental effects to the mix. The proper flow of tremie concrete creates a "bulge flow," rather than a "layered flow," where the older concrete is continuously pushed further away from the tremie pipe by the force of the fresh concrete rather than the concrete stacking on itself respectively. Bulge flow decreases the surface area of the concrete that is exposed to the water thereby reducing the amount of washout and laitance.

Washout of the cement paste was another concern, and a minimum amount of washout based on the United States Army Corp of Engineers (USACE) standard test

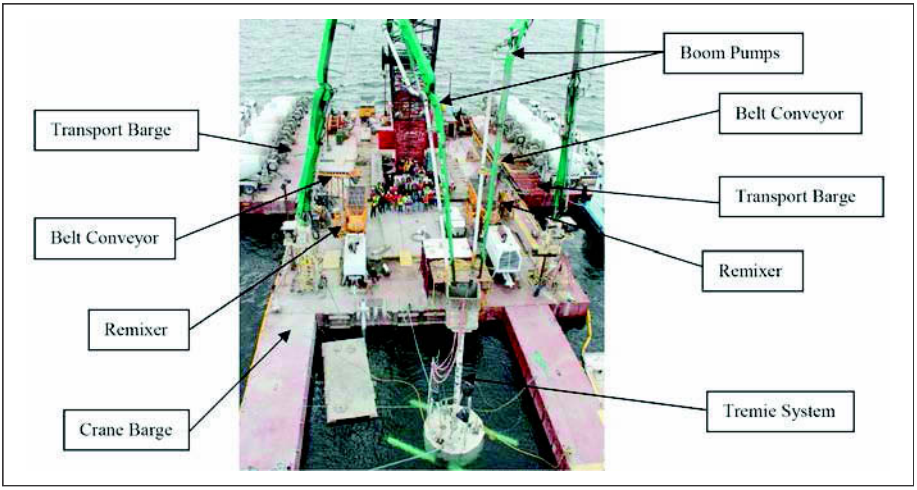

Figure 8. Entire tremie concrete supply system on crane barge

Table 1. Tremie concrete design criteria

Mix Property	Design Criteria
Compressive Strength	3000 psi at 28 days minimum
Slump Flow	22" to 32"
Washout	13% Maximum
Water/Cement Ratio	0.35 to 0.45 by Mass
Cementitious Content	600 to 850 lb/yd^3
Sand To Total Aggregate Ratio	45% to 50%
Maximum Size Aggregate	¾" to 1"
Maximum Curing Temperature	165 Degrees F (With min 20% flyash)

method CRD-C 61-89A was established. To meet this washout requirement with a highly flowable mix, a substantial amount of anti-washout admixture was proposed.

One of the potential concerns with this large mass concrete placement operation was the possibility of the concrete to develop high temperatures during hydration causing thermal cracking, or leading to delayed ettringite formation. To mitigate the thermal cracking and delayed ettringite formation risks, it was specified by the design engineer that 165 degrees F would be the limit with a maximum differential temperature of 35 degrees F for concrete mixes that had a minimum 20% flyash in relation to the total cementitious content of the mix.

Another factor that was to be avoided was the occurrence of cold joints between lifts of concrete. Vertical cracks that allowed water communication through the tremie concrete was the major concern. It was calculated that a single full lift of concrete would take approximately 16 hours to complete before the next lift would start to cover the underlying concrete. To avoid cold jointing the concrete, set time was targeted for a minimum of 24 hours before initial set.

Concrete stability refers to how well the mix maintains its flowability over time. Due to the extended transportation time for the concrete it was necessary to maintain the stability of the mix to a minimum of 2 hours. The concrete was delivered to the tremie pipe via mixer trucks on a barge. The travel time for the 8 trucks was around 45 minutes. Each complete barge required up to 2 hours to discharge.

Table 2. Tremie concrete mix design

Property	Mix PA730FASCC
W/C	0.40
Slump Spread	25" ±2"
Flyash Content	50% at 1.1:1
Sand to Aggregate Ratio	48%
Quantity of Super Plasticizer	1.5–2 oz/cwt
Quantity of Anti-Washout	n/a
Quantity of Viscosity Modifying Admixture	11 oz/cwt
Quantity of Slump Retention Admixture	10 oz/cwt
Quantity of Retarder	5.5–6 oz/cwt
Allowable Addition of Chemical Admixtures as Needed	1–2 oz/cwt of Super Plasticizer 1–2 oz/cwt of Slump Retention Admixture

Through trial batching, we found that a high flyash mix with a carefully proportioned cocktail of chemical admixtures was needed to meet all of the mix design requirements. During the laboratory trial batching phase we encountered several issues which included: the mix having a thixotropic nature when certain admixtures were used together; delayed set times with extremely low ambient temperatures as were expected at the 107 m depth in the lake; poor mix stability at high slump flows.

Following laboratory trials, field testing of the proposed mix design was performed using the actual tremie equipment that would be used during the placing operation. The field tests found issues that lead to changing the physical design of the tremie equipment and also modifications to the laboratory tested tremie concrete mix design.

What we learned was that a mix design with 50% flyash replacement is best to maintain a low thermal gain in a large concrete mass (Table 2). The low heat developed reduced the concern of thermal cracking and increased the stability of high slump flow concrete mixtures. It was also determined that the use of an anti-washout admixture (AWA) or a viscosity modifying admixture (VMA) is necessary when working with a high slump flow mix to stabilize the high fluid mixtures by keeping the aggregate suspended and reducing bleed water. For our purposes, the use of VMA was sufficient to meet the specified minimum washout requirements and the use of AWA in the mix caused gel times that prevented proper flow properties when dropped into the tremie system. One of the more important lessons learned from this project is that preparation and planning is the key to success with deep water tremie concrete placements and that it is necessary to perform a full evaluation of the potential risks by engaging in full scale or large scale testing prior to the tremie concrete placement operation.

COMPLETION

After the tremie concrete placement was finished the demobilization phase began. All anchors were removed from the lake and barges were disassembled. The crane barge was demobilized and the Manitowoc crane unloaded back to land. The flexifloats owned by VTC were stacked on land in the marine area while the rentals were sent back to the owner.

Eventually, when the TBM nears the intake structure, some marine equipment will be mobilized to install the bulkhead. Due to the extended time period before setting the bulkhead, the gaskets for the bulkhead were cataloged and stored in the warehouse at the main site to prevent environmental degradation. A test was performed prior to demobilization to show SNWA that the bulkhead could be installed from the crane barge with the intake installed (Figure 9). The actual bulkhead minus the gasket

Figure 9. Bulkhead test fit

was lowered through the intake riser and seated on the gasket flange inside the intake structure. The bulkhead was then removed and stored on land until it is needed to complete the tunnel connection.

With the exception of the bulkhead installation and anticipation of TBM docking with the intake the marine work for Lake Mead Intake #3 is complete and has been successfully performed.

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TIGHT TOLERANCES: INSTALLATION OF FIVE 60-INCH-DIAMETER, 358-FT DEEP WELL PUMP CASINGS

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Gregg Sherry ▪ Brierley Associates

Robert Steven Viers ▪ Frontier-Kemper Constructors

William Stauber ▪ City of Austin

ABSTRACT

Hydraulic conditions associated with a new deep water intake for the City of Austin's Water Treatment Plant No. 4 required installing five 60 inch diameter steel well casings a depth of 358 feet. Due to requirements of the vertical turbine pumps, these well casings had to be installed to tight tolerances. Using a raise bore method of construction, a qualified contractor combined with an experienced project team installed the well casings as planned, making this difficult installation appear easy. This paper will present a case history of the challenges and solutions found during installation of these well casings on this project.

DESIGN HISTORY

Scheduled decommissioning of the City of Austin's (COA) original water treatment plant and Austin's growth were key factors in the ground breaking construction of the Water Treatment Plant No. 4 (WTP4), the first new water treatment plant constructed by the COA in nearly half a century. In 2002, the COA hired a Carollo Engineers team to perform preliminary site assessment engineering, environmental studies, design, and construction phase services for a new water treatment plant, located in west Austin near Lake Travis. AECOM, a subcontractor for Carollo, was responsible for the raw water portion of the project, with Brierley Associates on the tunneling portion of the project. During design, it was determined that the new WTP4 facility would be planned for an initial treatment capacity of 50 MGD, with plans to expand to ultimate treatment capacity of 300 MGD. Due to the complexity of constructing intakes within large water impoundments like Lake Travis, portions of the raw water system were sized to accommodate the ultimate treatment capacity of 300 MGD, with others sized to mimic the water plant's phased capacity.

Overall, the design of the WTP4 Raw Water System includes an intake at Lake Travis, a tunnel from the intake to the pump station, a new pump station with five vertical turbine pumps, and the transmission main tunnel from the pump station to the water treatment plant site. As shown in Figure 1 Hydraulic Profile, the Raw Water Pump Station (RWPS) lifts water from the suction chamber and pumps it to the water treatment plant. The pump station's five vertical turbine pumps have a firm pumping capacity of 50 MGD. The bowls of each of these pumps are located 40 feet above the floor of the suction chamber, at elevation 540.0 feet, which equates to a pump column length of approximately 334.4 feet from the top of the pump's sole plate to the bottom of the

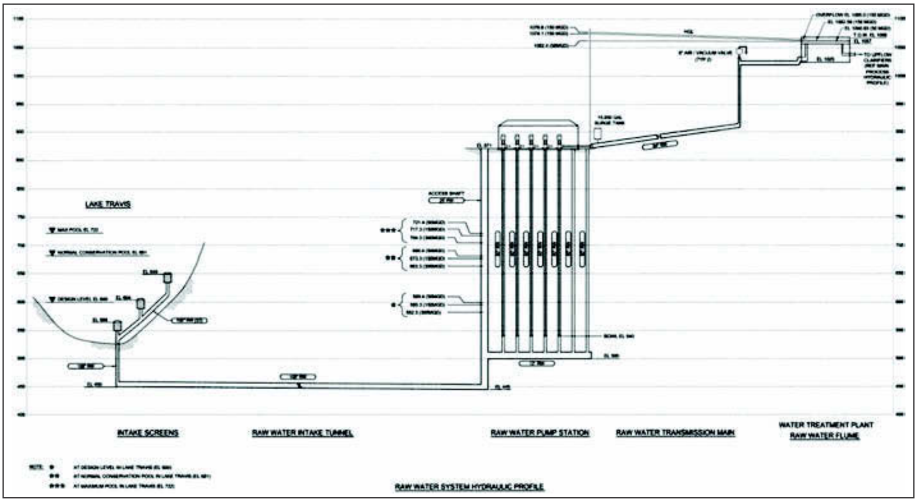

Figure 1. Hydraulic profile

bowl. Because the mechanical components of the pump shaft and pump column are fabricated and machined to insure that the pumps hang vertically, the well casing associated with these pumps also had to be installed to an extremely high level of tolerance.

Key components of work associated with the well casings include the following items:

- Access Shaft: A 425 vertical foot 25.5-ft inside diameter shaft from ground surface at elevation 870.0 feet to elevation 445.0 feet.
- Suction Cavity: A horseshoe shaped tunnel with inside dimensions of 12.5-ft wide by 12.5-ft tall with an overall length of 147.25 ft. The invert of the Suction Cavity is at elevation 500.0 feet, which is 65 feet above the bottom of the access shaft.
- Pump Suction Wells: The pump well shafts are 358 ft deep vertical openings that extend downward from ground surface to the 12.5-ft × 12.5-ft suction cavity. The pump well shafts are steel lined with 60 inch ID pipe that has polyurethane coating on the interior of the pipe and are encased in a minimum 4 inch annulus of grout. The pump well shaft layout consists of a single row of wells that are spaced at 18ft center to center of well. Figure 2 shows a profile view of the access shaft and suction cavity.

GEOTECHNICAL INVESTIGATIONS AND SUBSURFACE CONDITIONS

Geotechnical investigations, performed by Fugro Consultants, Inc, consisted of a total of five bores taken in the location of the pump station: B-11, B-12, B-13, B-14, and B15A. As shown in Figure 3, Site Plan, test boring B-13 was located closest to the access shaft and was drilled to a depth of 520-ft (elevation 400 feet). Test boring B-14, also drilled to a depth of 520-ft (elevation 386 feet), was located close to the end of the suction cavity. The three remaining borings were drilled between 35 and 75 feet to characterize the rock in the locations of the buildings and structures.

Results of the bores indicated that the access shaft, suction cavity, and pump suction wells were all located entirely in the Glen Rose Limestone Formation. The Glen Rose (Kgr) Formation is a relatively competent limestone formation which includes alternating hard and soft beds of limestone, dolomitic limestone, and marl that often

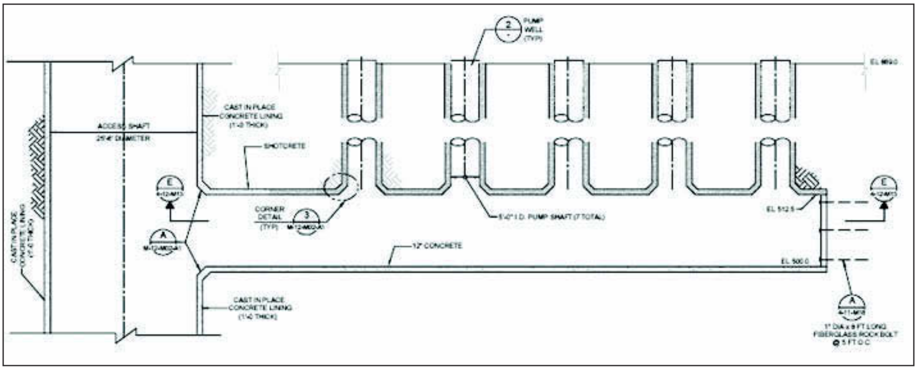

Figure 2. Profile of access shaft and suction cavity

vary in thickness and hardness. Test results identified an RQD ranging from 100 to 42 with an average of 85. It is noted that the Geologic Map of the Austin Area, Texas (Garner and Young 1976), shows no major faults or fractures along the tunnel alignment. Additionally, no evidence of faults were observed during site assessments. Both a Geotechnical Baseline Report and Geotechnical Data Report were included in the bid documents for review.

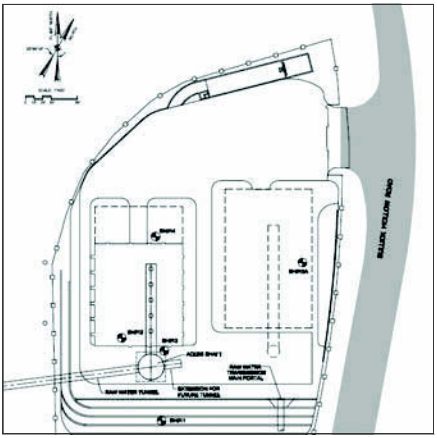

Figure 3. Site plan

CONSTRUCTION

As the project started into the detailed design phase, the COA decided to use the Construction Manager at Risk (CMAR) method of project delivery for construction of the water treatment plant and corresponding raw water intake system. MWH Constructors was selected as the CMAR firm to manage and construct the project. A total of five subcontractors prepared proposals on the raw water intake system. After review, the project was awarded to Austin Hill Country Constructors, a joint venture of Obayashi USA and Manson Construction Co, with Obayashi performing all work on the land, including the tunnels, and Manson completing all marine work. Under Obayashi, Frontier-Kemper Constructors performed the work associated with the raise bores for the vertical pump shafts.

ACCESS SHAFT AND SUCTION CHAMBER

Prior to beginning any of the raise bore operations, the access shaft and suction chamber were constructed (Figure 4). Obayashi started work on the access shaft shortly after the notice to proceed and construction took approximately 6 months to complete excavation from ground level to the invert of the suction chamber. Excavation was performed using an Antraquip AQM 100 shaft sinker specifically designed for vertical rock excavation. As noted earlier, the access shaft was excavated to an outside diameter of 28.5 feet with a final inside diameter of 25.5 feet. The final shaft lining was placed using a 25.5 foot outside diameter circular steel form designed for use in a top-down concrete placement operation. As soon as crews reached the elevation of 500 feet for the suction chamber, excavation transitioned from vertical to horizontal. The Antraquip AQM 100 roadheader was modified to cut the horseshoe shaped suction chamber. The relatively short length of the suction chamber was completed in 10 days. Excavation started on October 16 and was completed on October 26.

Figure 4. Access shaft

After completing the access shaft and suction chamber, the next step was to construct the shafts for the well casings. Due to the tight tolerances required, the contract documents indicated that full length pilot holes would be blind drilled from the surface using directional drilling tools that continuously surveyed during drilling to control the deviation of the hole. The pilot holes could be drilled in any sequence; however, due to the 18-ft center-to-center spacing, shaft excavation sequence was restricted; a shaft could not be excavated until all adjacent shaft casings were installed and grouted. Initially the maximum allowed excavated shaft diameter was 90 inches, but after review the maximum excavated shaft diameter was increased to 96 inches.

RAISE BORED

Raise bore, initially used in the mining industry for ventilation shafts, requires an opening such as tunnel or chamber below the surface. The raise bore process involves drilling an initial small diameter pilot hole to the desired depth of the tunnel or chamber at which point a reamer head is attached and pulled back up to the surface. Brought into the tunnel or chamber in a location different than the pilot hole, the reamer head is specified diameter of the finished hole and the drill cuttings from the reamer fall to the floor of the chamber where they are removed. Due to the ability to accurately control the pilot hole, raise bore is typically one of the most accurate methods to obtain straight controlled holes. In addition, the construction of the suction chamber for the vertical turbine pumps made this project an ideal application for raise bore.

Frontier-Kemper Constructors performed the work on the raise bore using a Robbins 7-SP raise drilling machine which provided both the drive for the pilot hole as well as the lift for the raise bore. The Robbins 7-SP drilling machine, powered by a 250hp DC motor, had the ability to develop a breakout torque of 252,000 Ft-Lbs. with a hydraulic reaming thrust up to 800,000 Lbs. The Robbins drilling machine was installed upon a temporary concrete platform that Obayashi constructed prior to Frontier-Kemper arriving on site as shown in Figures 5 and 6.

The key to accuracy in raise bore is the pilot hole. On the WTP4 project, the pilot hole was drilled using a Micon Rotary Vertical Drilling System (RVDS) developed by the German company Micon. The RVDS, a self steering drilling device, uses modern technology to automatically adjust to the desired target. At the start of the drilling, the vertical target was initialized and programmed into the RVDS. The machine was

Figure 5. Robbins 7-SP drilling machine

Figure 6. Robbins SP7 machine

then assembled and placed into operation after which time it was monitored. Inclination sensors in the RVDS system self control and steer the RVDS down the hole by hydraulic activated ribs. These hydraulic activated ribs, positioned on a non-rotating stabilizer sleeve, continually adjust the tool into the vertical direction. A mud pulse system in the drill mud transmits survey measurements back to the surface where they are digitally displayed on the operators screen during operation. If any significant problems are observed at the monitoring station, the pilot drilling is stopped and the RVDS is removed for observation. Seeing a RVDS in operation makes you aware of the advancement of engineering and modern technology. Even though it seems like magic, the RVDS system self steers itself to a significantly high degree of accuracy. Since Micon first introduced their RVDS in 1994, they have drilled 50,000 meters of vertical holes and Micon reports that they their average accuracy has been less than 0.1%.

The diameter of the pilot hole was 13¾ inch and the Micon RVDS was attached to the Robbins drilling machine with normal raise drill rods. The drilling fluid, used pulse data from the RVDS to the operator monitoring station, also was used to remove the cuttings from the bore hole. The drill fluid was monitored on a regular basis and the viscosity adjusted depending upon the type of rock encountered. The drill mud was circulated in a closed loop system which consisted of a high volume tri-plex pump with three settling tanks. At the completion of the drilling of the pilot holes, the settlings in the baker tanks were removed for disposal.

The 354 foot long pilot hole was drilled on a 24-hour operation. The pilot hole advanced around 5-ft an hour and the first pilot hole was completed in approximately 70 hours. The sequence of drilling pilot holes was well casings 1, 2, 3, 4, and 5.

Once all of the pilot holes were completed, the reamer head was assembled and lowered down the access shaft and horizontally into the suction chamber for connection to the drill rods at the end of the pilot hole as shown in Figures 7 and 8. An 8 foot diameter Sandvik reamer head was used for reaming the full shaft diameter of 96 inches. This cutter head was a flat head design, configured of a standard base fitted with wings to achieve the desired 8-ft diameter excavation fitted with 14 roller cutters. To maintain alignment with the pilot hole and reduce the ability of the cutter head to drift, a series of stabilizers were installed 15–25 feet just above the cutter head. The stabilizer OD was 13.25 inches. As reaming progressed from the suction cavity to ground level, the cuttings fell to the bottom of the shaft into the suction cavity for removal. The amount

Figure 7. Reamer in suction chamber

Figure 8. Starting reaming operations

Figure 9. Reamer at ground level

Figure 10. Completed well shaft

of dust produced from these cuttings was minimal which allowed crews to muck the cutting concurrently with reaming operations. Excavated material from the reaming was removed from the suction chamber with a Caterpillar 277 skid steer loader which loaded a rectangular 8 CY muck box at the access shaft. A Liebherr HS 885HD crawler crane hoisted the muck box up the access shaft stock piling the material at ground level for disposal.

The first well casing reamed was well casing no. 5 which was completed on a 24-hour operation. The reaming advanced around 6 ft an hour and the first well shaft was completed in 59 hours and 30 minutes. Photos of the reamer at ground level and a completed well shaft are shown in Figures 9 and 10.

WELL CASING INSTALLATION

Upon completion of reaming, the steel pipe well casings were installed and grouted. As mentioned earlier, restrictions prohibited reaming the adjacent hole until the well casing was installed and grouted. This resulted in significant planning and coordination between the different operations. Accordingly, installation of the first well casing commenced immediately after the first hole was reamed and the raise bore equipment moved. Installing the 358 feet long well casings in one piece was impractical; therefore, the well casings was fabricated in sections and installed to obtain a complete casing. Initially designed for a full penetration weld, Obayashi proposed a change to flanged joints that was accepted by the project team. Due to trucking restrictions, the well casings were shipped to the project site in 50 foot pipe segments.

Installation of the well shaft casing was completed by top down construction methods. Using a Kobelco CK2500 crawler crane, the 60 inch diameter by 50 foot flanged

Figure 11. Well casing installation

Figure 12. Well casing support frame

Figure 13. Bolting flanges on well casing

Figure 14. Grouting well casing

end pipe segments were lifted from horizontal to vertical and set in a structural steel support frame above the well bore as shown in Figures 11, 12, and 13. The first pipe segment was hung in the structural steel support frame from the pipe flange allowing the crane to unhook from the first suspended pipe segment and rig to the second pipe segment in the erection sequence. Then the second pipe segment was set above the first pipe segment and suspended by the crane until the flanged joint was aligned. A pipe gasket was installed and the flanges were bolted up using high strength bolts. Installation and torquing involved a two step procedure which involved initially tightening the bolts to a snug tight condition using an air impact wrench and then using a calibrated hydraulic torque wrench to tighten to the required torque. The bolts were numbered and tightened in designated sequence, alternating sides to insure proper loading conditions. Once the bolt up of a joint was completed, the assembled pipe segments were lifted off of the structural steel support clamp and this support clamp then hydraulically opened to allow the earlier pipe segments lowered into the shaft. This top down pipe installation process was repeated for all seven pipe segments. In order to account for possible adjustments in elevation, a sacrificial 1-ft extension was added to the last pipe segment. Upon completion of the installation of all seven pipe segments, the well casing was adjusted to final alignment and the top and bottom of casing blocked to keep it secure during grouting operations.

Once the casing was installed and aligned, the top and bottom of casing was blocked and crews constructed the lower grout bulkhead. In addition, additional shoring and falsework was installed to help support the casing and water inside the casing during grouting operations. Grouting of the well shaft casing annulus took place in six individual grout stages (Figure 14). A Putzmeister BSA 2109 pump was used to pump the grout into a hopper set at the top of the tremie pipe. Grout tremie consisted of 10-ft sections of 5 inch diameter slick line with a hopper that was supported by the well casing support frame. The hopper was easily removable to allow the slick line to be removed in 10 foot increments. The first 50 feet of annulus was grouted in three stages to develop a grout plug at the bottom of the casing. Once Stage 3 grout had cured for 24 hours, the entire well casing was filled with water to offset the external grout pressure. The remaining 306 ft of casing was then grouted in three stages. During grouting operations the grout elevation was monitored on a minimum two sides of the well casing to confirm that grout elevations around the casing were within 1 foot of level. Primary measuring devices consisted of measuring ropes with a weight and flat plate on the measuring rope end. Secondary measuring devices consisted of measuring with lasers. It is noted that both the support clamp and suction cavity bracing systems were engineered for this project and remained in place until the well casings were completely grouted.

TOLERANCES AND SURVEY CONTROL

Due to the requirements of the vertical turbine pumps, the centerline of well casing was to be installed within 1 inch of plan center at ground surface (top of casing) and within a 4 inch right circular cone at the suction chamber (bottom of casing). Due to these tolerances required, survey control was a critical part of the installation process. Accordingly, there were a number of survey checks and second checks during installation which included the following:

- Survey of the location of the pilot hole.
- Survey of the exit of the pilot hole in the suction chamber.
- Checking of the trueness of the flanges on well casing piping. During fabrication the flanged edges of the pipe were measured and checked to determine any gross deviations of flange perpendicularity to pipe.
- Checking the well casing joint line during bolt up of each flanged joint. Lasers were fixed to the lead pipe segment with a known offset to the pipe wall. During bolt up of the flange joint, this offset was checked against the previously erected pipe segment.
- Final pipe alignment was set from ground elevation. A bracket was fixed to the top of casing so that the center of bracket was set at center of casing. Using a total station the top of casing center was aligned and secured in the X, Y, and Z planes.
- The bottom of casing was measured and aligned by using a plumb bob hung from the bracket at center of casing. Crews measured and aligned the bottom casing from the offsets from casing wall to center of casing.
- Final well casing survey was completed after grouting.

An addition to the above surveys and checks, there was a requirement to pull a 122 inch long pipeline mandrell through the well casing both prior to grouting and after grouting to insure there was no deflections in the well casing.

LESSONS LEARNED

The following lessons learned on the project.

- The well casings were initially to be welded, but after review, these were switched to flanged joints. This was very beneficial and although there was a cost for the flanges, the flanged joints were easier to install than welded joints. In addition, flanged joints make it easier to maintain alignment during installation.

- Due to the specific tolerances for the well casings, a decision was made to have a Micon representative on site during drilling of all of the pilot holes. This was prudent and beneficial to the project. Although this was a cost, this representative increased the level of insurance on drilling the pilot holes.
- During installation of the well casings, two complete sets of lower falsework were rented for final alignment and grouting operations (Figure 15). In addition, two sets of upper structural support clamps were fabricated with certain reusable elements between falsework setups. Additional schedule savings could have been realized by providing five complete sets of falsework and upper structural support clamps.

Figure 15. Lower falsework in suction chamber

CONCLUSION

Installing 358 foot well casings to tolerances within inches is very challenging; however, the project team led by Obayashi and Frontier-Kemper Constructors planned and scheduled properly to made this difficult installation appear easy.

SHAFT CONSTRUCTION METHODS

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ABSTRACT

The San Francisco Bay Area has historically been a center for tunneling and shaft construction. Shafts are often vital access for construction, operation, and maintenance of tunnels and underground structures. No one-size-fits-all approach exists for shaft design and construction and because of this, the method used for the construction of a shaft must be carefully selected in order for specific project requirements to be met. This paper will examine several common considerations for shaft design and construction including: controlling geologic conditions, constructability concerns, as well as non-technical issues. Common shaft construction methods and the use of a series of Bay Area case studies demonstrate how the design considerations relate to the use of several construction techniques. After discussing strategic design methods, we will examine several case studies of shafts constructed since 2007 and how each of these shafts addresses the various design concerns/considerations.

INTRODUCTION

There are numerous methods that can be used to construct vertical shafts. Each shaft will have a different set of requirements based on its temporary and final use, and specific project requirements. This paper presents the design considerations that can lead the designer or owner to select specific shaft construction methods, along with a general discussion of several methods for supporting vertical shafts. Seven shafts that were constructed in the Bay Area since 2007 will be presented later in this paper. The various techniques employed for these shafts illustrate how there is no one-size-fits-all approach to shaft design even when considering a relatively small geographical region and timeframe. The selection of projects in the Bay Area was chosen in order to limit the scope of this paper.

DESIGN CONSIDERATIONS

When designing a shaft, there are several main considerations that will have a direct effect on the method chosen. Some of these considerations are discussed below.

Most often, design considerations can be grouped into one or more of the following three categories: controlling geologic conditions, constructability concerns, and non-technical issues. The diagram in Figure 1 shows several of these criteria and the category or categories that they fall into. These criteria are discussed later in this section. An owner or contractor’s final decision for support, shape, and size is based on a balance of initial support cost, excavation cost, and the most efficient size to facilitate functionality while minimizing construction cost.

Shape and Size

Structurally speaking, circles are the most efficient shape for a shaft. This is one main reason why most shafts, particularly the deeper shafts tend to be circular. However, there are situations when a circular shaft cannot be used. Most common, is when the

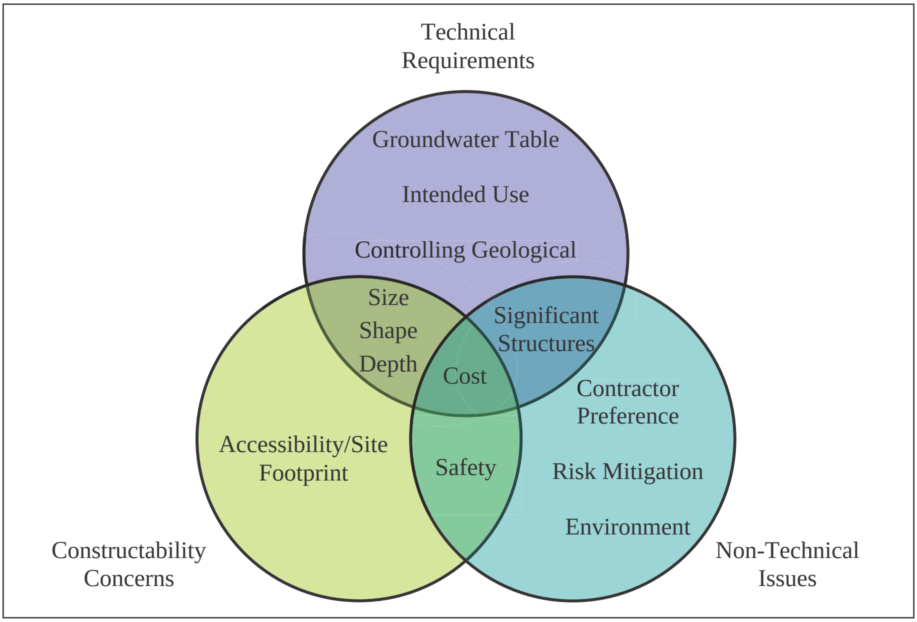

Figure 1. Diagram displaying relationship between design criteria. © S. Von Stockhausen

maximum dimension required by a shaft doesn't fit into a site footprint in both directions, in this case a rectangular or oblong shaft might be used. Another reason for a non-circular shaft would be when the shaft itself will be a permanent structure that requires a shape other than a circle.

Selecting the shape and size of a shaft must be the first question of temporary and permanent function. For example, the temporary function of many shafts is to provide an area for tunnel machines to launch. Most tunnel equipment occupies a rectangular footprint, which means that a rectangular shaped shaft requires the least volume of excavation in order to permit installation of the tunneling equipment. However, the most efficient structural design uses a round shape and often can eliminate the need for internal bracing or walers (depending on the initial support type). The drawback to the round shaft shape is excavation volume inefficiency. To inscribe the rectangular equipment footprint inside a circle requires a diameter that may increase the shaft excavation volume by more than 200%.

Beside consideration for equipment, the shaft size should consider excavation methods, worker safety and ultimately excavation cost. There is a breaking point in shaft size (whether rectangular or round) where the unit excavation cost in a small shaft is so high that total excavation cost may be higher despite a lower total volume of excavation. This outcome is related to excavation methods permitted by the shaft size. Typically, the first 20 feet of any shaft is immune to this result because most commercially available excavation equipment can easily remove spoils within 20 ft of the ground surface. Once the shaft depth has exceeded that dimension where spoils may be easily removed from the surface, unit excavation costs can vary wildly. In extreme cases:

- High Unit Excavation Cost—very small shafts that must be excavated by hand or auger
- Low Unit Excavation Cost—very large shafts where 12 to 18 metric ton excavators can work freely

Depth

The final depth of a shaft will have a significant impact on the construction method selected. Many methods have practical depth limitations—sheet piles, secant piles, slurry walls, etc. In soft ground, the lateral earth pressure can increase with depth and in rock, it is possible to have overstressed conditions at depth. In both of these situations it is likely that the support required at the bottom of the shaft will be more robust than that required at the top which will either require the support type to vary with depth or require that a more conservative support be applied.

Geologic Conditions

The type of material to be supported and the presence of the groundwater table are among the most important factors to consider during the design and construction of a shaft. For example, many methods are either not appropriate for, or require special accommodations for excavation beneath the groundwater table and the stability of excavations and support methods are directly related to the geology. Unsaturated ground conditions, when above the groundwater table, can stand unsupported for a period of time while support is added. These soils are considered to have a long standup time and are beneficial to soldier pile, caisson and segmental lining of shafts. Other conditions, such as excavation either below the water table or in cohesionless material, will not allow unsupported ground during excavation. The presence of swelling clay or squeezing conditions will also affect the support requirement.

Geologic considerations always dictate excavation methods and often dictate initial support selection. A designer and a contractor should ask some practical questions related to geology: Is the strata relatively hard or soft? Does the strata contain material susceptible to raveling, running or flowing during excavation? Where is the water table relative to start and completion elevations? These questions can be found within the analysis provided by a geotechnical document like a Geotechnical Baseline Report (GBR).

Intended Use

Shafts are used for a variety of purposes and the design of a shaft will often rely on the purpose of the shaft. For example, a shaft that is to be used as an intermediate heading will have different requirements than one that is meant for launching a piece of equipment. The question of whether a shaft is meant to be permanent or temporary works will also factor in to the design of support, since temporary structures often do not need to be designed with the same robustness of a permanent structure.

Significant Structures

It is becoming increasingly rare that a project can be constructed without its possible effect on a nearby structure. The term ‘significant structure’ in this circumstance can be used to describe anything from a building, to an underground utility, or an environmental concern such as a nearby body of water. These factors may be deemed important by the owner, designer, or contractor and can become an important factor for the shaft design. In all of these cases the construction of a shaft must not negatively affect the pre-existing structure or condition. A risk assessment may be carried out for different structures in order to determine whether or not they are significant.

Contractor Preference

Oftentimes the design and/or selection of shaft initial support are left up to the contractor. In these instances, once the technical requirements have been met, it is often the contractor’s preference that will be the deciding factor when selecting a support method. The preference may be due to available equipment, familiarity with a technique, or material prices.

Environment

Environment can refer both to environmental requirements, such as limiting the impact of construction activities on local flora and fauna, as well as, limiting the impact of construction on local human residents. In urban or environmentally sensitive areas it is often required that air quality and noise vibration monitoring take place during construction to ensure that construction impacts are within allowable limits. These limits may be used to determine a support method because different construction techniques will have different impacts, e.g., some methods will produce much higher vibration than others.

Safety

The safety of workers and the general public is always of the utmost concern for any construction endeavor. Shafts are often oversized to allow appropriate working room to prevent unsafe situations. Contractors commonly elect to install additional support to increase the safety of the shaft.

Site Accessibility/Footprint

The area available to construct a shaft is just as influential as any other design criteria. It can affect the practicality of using certain methods and for some techniques, extensive equipment and laydown area is required. Site accessibility and available footprint, can also factor into other design criteria such as safety, contractor's preferred equipment choices, and the determination of significant structures.

Cost

Designers and contractors alike are often tasked with selecting an appropriate shaft support method through the balance of a robust and cost effective solution. While it may not be the primary criteria used to make a final selection, when given the choice between two methods that provide comparable support, the less expensive option is generally chosen. However, the desire to build the most cost effective structure cannot override the importance of criteria such as safety and geologic conditions.

TECHNIQUES

There are several common techniques that are used for shaft construction. Examples are discussed in this section. This section also discusses several uncommon techniques to the U.S. tunneling market, which have high costs and are rarely used. The technique section is arranged by methods that are typically used in the U.S. heavy civil construction starting with the least practical depth and transitioning to the most practical depth.

Trench Box and Shoring

Trench boxes, trench shields, and other standard pre-engineering soil/rock support are typically limited to 6 m (20 ft) to 9 m (30 ft) deep and dewatering should occur if the invert of the shaft is below the groundwater table. Used for shallow shafts, slide rail systems are relatively easy and quick to install by utilizing spreader bars, spreader posts and panels tight against the soil or rock. Trench boxes, pit-kits and other pre-engineered support systems are designed to meet and exceed various active soil lateral loading to a limited depth. Slide rail systems, similar to trench boxes, are a series of telescoping boxes, one inside the other. The boxes support the ground on all four sides and can have up to three guide rails to place the telescoping support panels. The use of steel sheeting and sheet piles can provide the direct contact between the primary support system and the soil/rock required.

Sheet Piles

The installation of sheet pile shafts are common with square or rectangular geometries using internal bracing such as walers and struts. Sheet piles can be an effective support method up to approximately 15 m (50 ft) deep in relatively dry conditions. The use of sheet pile shafts is one of the most practical technique for shaft construction. It is considered one of the least expensive support techniques for constructing a shaft, particularly if the sheets are removed for re-use. Sheet piles are typically interlocking and can be advanced into soil using a hammer or vibratory hammer. The soil is excavated once the sheet piles are installed. Internal bracing is required as the excavation progresses. Sheet piles can act as a relatively impermeable wall if installed correctly and typically prevent piping through interlocking joints, though some joint leakage is typically tolerable. The use of sheet piles is one of the few support methods that can be reused, thus increasing the cost effectiveness of each use. There are a number of draw backs to using this support technique such as its application in bedrock, boulders and cobbles. Pre-drilling and even blasting have been used in difficult driving conditions to enable the sheets to be driven to target depth. Smaller sheet pile walls are typically designed to cantilever where large walls are usually anchored. Rectangular cantilevered shafts are limited in depth due to the effective support capacity it can provide.

Soldier Piles and Lagging

Soldier pile shafts are often used for large excavations when a large working area is required to small confined areas with limited access. These shafts are installed sequentially by drilling and sinking steel H-beams around the proposed excavation. Commonly, steel plates or timber lagging are placed between the beams to provide ground support. Steel sheets can be driven quickly if running or flowing sands are present, however timber lagging is placed by hand in most cases. The ground is excavated as the plates or timber lagging are installed. Considerable ground-loss is known to have occurred as the steel plating was not installed sequentially with the advancement of the excavation. This technique is best used for low permeability soils with a considerable standup time where the groundwater horizon is below the excavation. A drawback to using this method is the issue of creating an alignment within the tolerance of the timber lagging or steel sheeting installed.

Caissons

Shaft caissons, or sunken caissons, can be used to advance shafts through soil using a steel or concrete cutting ring at the base and adding one piece of concrete or metal ring to the top as each lift of soil is removed. This method is mostly used in the Midwest, but limited in other regions of the United States. Caisson shafts are not commonly used for shaft excavations over 25 m (80 ft) deep, in areas of complex geological and groundwater conditions, or shafts with a diameter over 4 m (14 ft).

A common difficulty with placing caisson shafts is aligning the caisson vertically, while advancing it into place as a clam bucket removes the soil. Common problems include placing the caisson off vertical and impeding the advancement of the casing if the shaft experiences considerable convergence before the casing is set into place. Wedging or trapping the caisson often incurs expensive techniques to free the shaft support. The design of the steel or concrete shaft may allow for the caisson to be jacked into place if required.

The used of a Vertical Shaft Machine (VSM) was first utilized in the United States in April 2012 by a joint venture of James W. Fowler and Herrenknecht to excavate a 9 m (30 ft) diameter, 44 m (145 ft) deep shaft for the Ballard Siphon Project. The VSM can excavate the soil or rock while a portion of the shaft is underwater, which assists in balancing the groundwater in the in-situ soil or rock. While a similar vertical shaft boring machine (V-mole) has been used in the mining industry in the United States, it is fairly uncommon in the civil tunneling practice in the U.S. This emerging technology is actively used in Europe, Russia and the Middle East and can be considered to have a much higher capital cost in comparison to other methods. Use of the caissons support technique may increase with the development of Vertical Shaft Machine (VSM) technology.

Figure 2. Cutter soil mixing construction at the launching shaft at the Sunol Valley Water Treatment Plant. Photo courtesy of Drill Tech Drilling and Shoring, Inc.

Deep Soil Mixing and Cutter Soil Mixing

Deep soil mixing includes the use of an auger to mix or blend cement with the in-situ soil to create a soilcrete mixture. Cutter soil mixing is a mechanical method that involves mixing or blending the in-situ soil with cement to create soilcrete mixture using a hydromill type of attachment. The soilcrete mixture is designed to increase the in-situ engineering soil parameters such as the uniaxial compressive strength, shear strength and reduce the permeability of the soil. Soil mixing is commonly used as a supplementary support method for improving the ground conditions, particularly where there are nearby structures. The use of the deep soil mixing method is limited to about 27 m (90 ft) depth due to equipment constraints and cutter soil mixing has the ability to reaching over 61 m (200 ft) depth. The San Francisco Transbay Transit Center achieved successful result with cutter soil mixing up to 240 ft depth.

A soil-cement mixture is developed based upon soil borings where an optimum ratio is developed to provide an adequate shear and compressive strength before the actual soil mixing begins. A wide variety of mixing tools and configurations can be used to meet a variety of complex geology settings. Cobbles, boulders, and bedrock mixing are poor mixing materials for the cement mixing and should be avoided for this technique. Typically, field trials are conducted to ensure the mix design meets the compressive and shear strength specifications and the entire soil mixing column can be met. Figure 2 shows typical equipment used for cutter soil mixing.

Secant Piles

Secant piles are a common method of shaft support and are closely associated with solder piles and drilled shafts. The method involves the drilling of circular holes/shafts, typically ranging from 0.6 (2 ft) to 1.2 m (4 ft) in diameter, and filling them with a cement mixture, often concrete. Secant pile shafts have a maximum practical depth of 37 m (120 ft). Improving equipment and accuracy, the depth may increase in the near future.

Figure 3. Secant pile shaft at the New Irvington Tunnel Project. Photo © Jacobs Associates 2003.

The holes are drilled in the circular pattern around the shaft in two stages. The first stage involves drilling and placing concrete in alternating holes (primary piles). The second stage consists of drilling a secondary pile through adjacent primary piles, thus creating an overlapping pattern. The secondary holes/piles are drilled after the cement is set, but is still green. In ground conditions where the soil is loose, casing can be installed into the hole before the concrete is placed and high strength concrete can be used. The piles can be unreinforced or reinforced with either steel rebar or steel beams. Often the casing is pulled for the hole when concrete is placed to form a pile in the ground. Alternating piles are drilled, so that when interlocking piles are completed, the casing does not travel out of vertical due to the resistance of concrete vs. soft soil. This method is good for shafts that will be constructed below the groundwater table, variable ground conditions where both hard and soft ground conditions may be encountered, and in situations where nearby structures are deemed sensitive to shaft construction. Figure 3 shows a secant pile shaft under construction at the New Irvington Tunnel Project, Vargas Shaft.

It is possible to construct shafts in many shapes using this method, but secant piles are often used to construct circular shafts to take advantage of the circumferential stress that develops when secant piles have been installed correctly.

It is important to drill each bore hole accurately and ensure the hole is completely filled and overlapping, creating an impermeable barrier and adequate support for the ground. With improved accuracy of the equipment in North America the typical maximum effective depth a secant pile shaft can be constructed is 37 m (120 ft) due to the deviation of each drilled pile and possible raveling of soil into the drilled pile, however the Transbay Transit Center Secant Pile Shaft test program achieved 73 m (240 ft) depth with excellent results. It is costly to fix such a problem, which seems relatively common for shaft sinking in civil applications. A common approach to fix a partially cemented shaft or windows is to drill another series of secant piles around the existing shaft, which increases the overall cost of the shaft.

Diaphragm Slurry Walls

Diaphragm walls are best suited for civil applications for shaft stability of large open work sites where a structure is required below 25 m (80 ft) depth, and have been successfully completed over 80 m (260 ft) in depth, and used for shaft excavations up to 100 m (330 ft) deep.

This highly specialized technique using a guide-wall to align the clamshell excavation. Panels typically vary from 0.5 to 1.5 m (1.6 to 5 ft) in width; however the thickness has reached 1.8 m in width for Lee Shaft in the United Kingdom. During panel excavation, stability is maintained by the use of bentonite powder mixed with water (slurry). Bentonite is a common drilling additive that is used to stabilize the excavation by decreasing the fluid loss into the soil or rock and increasing the density of the material in the excavation, thus creating a higher lateral pressure to counter the soil lateral earth pressure trying to push the soil into the excavation. The most common approach is to use a crane-mounted clamshell to remove soil. A hydrofraise is used to excavate the deeper portions of the panels. The installation sequencing occurs by excavating every other panel in the first phase (primary) while using slurry to prevent the soil from raveling into the excavation. A structural steel or steel rebar structure can be placed in each panel to provide additional stiffness to the support system. The panels are poured with concrete after reinforcing cages are placed in the specified panels. A concrete tremie slab is poured in each panel by displacing the slurry with concrete to create an initial or final shaft support. Stop Ends are placed on one side of each primary panel and removed after the concrete is poured or excavated as part of the secondary panel and are used to create improved quality joints between the primary and secondary panels. The use of diaphragm walls are conducive for most soil conditions including most cohesionless soils, however it may be difficult to maintain panel stability in loose material/high groundwater, groundwater velocity and weak bedrock. Diaphragm slurry wall shafts are most effective in urban environments where ground control is critical, dewatering is not practical and typical shoring is difficult. High costs for specialized equipment, requirements for pre-trenching to avoid utilities, obstruction removal in soil with boulders and complete closure to create a watertight are a number of difficulties experienced with this technique.

Drilled Shafts

The drilled shaft support technique is constructed by drilling a cylindrical hole of the required depth and subsequently filling it with casing such as precast concrete, steel, or corrugated metal pipe. The use in cohesive soils or bedrock creates an arching effect, which retains the media. The series of drilled shafts can be grouted to achieve a greater stiffness of the soil or rock mass to achieve a higher arching affect. This technique is the fastest and most cost efficient installation method commonly used for sinking shafts, however the diameter is limited in most cases to about 6 m (20 ft) using a single blind shaft drilling system and approximately 3.5 m (12 ft) in diameter using an auger guided drill rig. However, this diameter is increasing as effective equipment is produced for the U.S. market. The shafts maximum effective depth is approximately 80 m (260 ft). This method is best in dry cohesive material with a number of drawbacks including high costs, due to the specialized equipment, and is limited by the diameter of each drilled shaft and is not applicable for a range of geologic conditions.

Precast and Cast-In-Place Segments

Precast segments are typically made out of high strength concrete. Rubber gaskets may be applied to the precast segments to create a water tight barrier. There are three common methods for installing the precast segments while constructing a shaft. The jacked caisson method allows for high advance rates and does not have to be dewatered during excavation. The use of jacks (gallows) on the surface allows for advance into soft soil, such as sand, peat, alluvial material, and soft clay. Excavation occurs after the liner is jacked into place. This is similar to the caisson shaft. The segments can also be installed using the chimney method (bottom up) and the underpinning method, which requires the shaft to be dewatered and have competent soil conditions or bedrock and a long stand-up time during installation. Often one segment width is exposed at a time to reduce the risk of instability or failure. The structural concrete can be reinforced with steel rebar, synthetic or steel fibers. The diameter of the shaft requires a minimum diameter of 2.5 m (8 ft) and there is no extent to the effectiveness of the precast segment method. Annulus grouting is required to reduce settlement and provide structural stability. This method is best utilized for mostly dry conditions and its effectiveness is reduced when the soil is wet or saturated. Additionally, vertical shaft machines can be utilized for this shaft construction technique and may have advantages with cohesive soils or weak bedrock with long stand up times.

Liner Plates

Liner plate installation includes the placement of small panels of steel panels designed to interlock. Liner plates can be installed using the underpinning method or caisson method by jacking the liner plates into place, which is not typical. However, liner plates are considered to provide more flexibility during installation due to the workability of the steel and the wide range and availability of various plates. Steel sets/ring beams placed within the excavation can provide additional support for heavy loading conditions or large diameter shafts. The most obvious advantages of this structure are that maximum support per foot of tunnel that is obtained with minimum weight of steel. Liner plates, properly grouted or backfilled as tunneling progresses, form a dependable structure in either cohesive or non-cohesive ground. Liner plate structures generally require no additional support in tunnels up to 4.5 m (14 feet) in diameter (DSI, 2012). However, liner plates typically require the absence of the ground water table and minimal groundwater inflow during installation. Additionally, liner plates are limited by soil conditions in running or flowing sand or gravels. Duration of construction and environmental conditions may necessitate galvanized plates or additional metal thickness to mitigate excessive corrosion.

Ground Freezing

Ground freezing for shaft excavation is commonly used when excavating beneath the groundwater table in unstable soils and at a soil and rock interface. Ground freezing is the process by which soil pore water is frozen in-situ to create frozen soil material and impart strength and create impermeability in the soil mass (Schmall, 2012). Therefore, the in-situ presence of ground water or human induced groundwater presence is required for this technique to work properly. A typical installation technique involves inserting freeze pipes in drilled holes, which are socketed into the bedrock, around the shaft and circulating chilled brine or liquid nitrogen and is frozen based on shaft diameter and freeze thickness. The freeze pipes are drilled in a designed pattern to create complete closure around the shaft, thus creating an impermeable media around the shaft to the freeze depth. The freeze condition is a temporary method for stabilizing the complex ground conditions until a permanent shaft lining can be installed. Excavation occurs sequentially with support added before the next lift is excavated. Freeze shafts can be excavated without providing initial support depending upon the geological conditions. Initial or temporary support is typically a pre-fabricated liner or steel set/ring beams with lagging. Once the shaft has been excavated to depth and all liner pieces placed, the annulus between the excavation and the outside of the liner is grouted to maintain a watertight seal. The invert of the shaft may be frozen to prevent seepage and destabilization. Historically, the use of freeze shafts in the United States has been successful up to 84 m (275 ft) (Schmall, 2012).

This method is especially beneficial in areas with saturated, granular soil with low cohesion, and in areas where it is not realistic or desirable to pump groundwater. Ideal use of ground freezing can create an improved environment for difficult and variable ground conditions at contacts between soil and bedrock. “In fact, for deep mines, no better method has yet been established for sinking production shafts through deep, water-bearing ground” (Schmall, 2007). Additional steps need to be taken when using the ground freezing technique if there is a constant flow of water through the planned

Figure 4. Drilling for rockbolts at the Calaveras Dam intake shaft. Photo courtesy of Drill Tech drilling and Shoring, Inc.

excavation. Commonly liquid nitrogen is used at a lower freezing temperature. A planned freeze temperature of –10 to –20° C with the use brine solution is common for freezing the in-situ soil. Sometimes ground water velocity may be too high to properly create a complete freeze. The ground water velocity or "specific discharge (m/day) is a function of hydraulic gradient multiplied by the hydraulic conductivity. For ground freezing the traditional rule of thumb when water is flowing in a single direction through the proposed freeze area is that if the discharge is greater than 1m/day, the freeze wall may not close properly. This is a general rule of thumb and is not applicable for all cases.

Rock Bolts/Dowels and Shotcrete

Rock bolts and shotcrete lining is common for shafts that will be excavated in rock that is either above the water table, or, when groundwater inflows can be managed during excavation. During construction, lifts of a pre-determined height are excavated followed by the installation of rock bolts and then placement of shotcrete. The shotcrete design may have synthetic or steel fiber or more commonly welded wire fabric is placed. This sequence is repeated until the final shaft depth has been reached. Shotcrete, welded wire fabric, lattice girders and ring beams can provide additional support for larger diameter excavations and where high lateral loads are expected. Lattice Girders and beams are common to help maintain circularity and the shaft profile in soft ground conditions. Figure 4 shows a crew at the Calaveras Dam intake shaft drilling for rockbolts.

CASE STUDIES

The following shaft case studies are presented with a brief project background and construction details. The construction details include information on design criteria used-during design and/or construction that affected the choice of shaft support method.

Lenihan Dam Outlet Modification Project, Los Gatos, CA

The Lenihan Dam Outlet Modification Project, built for the Santa Clara Valley Water District, in Los Gatos, CA included an approximately 610 m (2,000 ft) long tunnel, intake and outlet structure, and intake shaft. The intake shaft served as the tie-in between the intake structure and the pipeline located in the tunnel and was also the ending point for the tunnel. Geology for the tunnel and shaft included Franciscan Mélange and

Table 1. Lenihan Dam outlet modification project

Owner	Santa Clara Valley Water District
Shaft contractor	Drill Tech Drilling and Shoring, Inc.
Shaft purpose	Intake shaft, temporary works
Size	4.8 m (16 ft) diameter 12.3 m (40.5 ft) excavated depth
Selected support method	Secant Piles
Primary design criteria	High groundwater table Risk mitigation (frac-out into reservoir)

serpentinite (Lenihan Dam GBR, 2007). Table 1 shows a summary of the shaft at the Lenihan Dam Outlet Modification Project. A discussion of the construction follows.

Of primary concern for the shaft was its proximity to the Lexington Reservoir, less than 3.5 m (10 ft) from the banks (the reservoir level was drawn down for this portion of the work). The original design was for the shaft excavation to be supported by ring beams and liner plate with sequential excavation and support within curtain grouting to maintain a dry excavation. The ring beam and liner plate shaft was designed for a 4.5 m (15 ft) inside diameter. Excavation would have occurred sequentially after support was installed leaving up to 1.5 m (5 ft) of open ground at a time.

The contractor proposed a no-cost change order to substitute a secant pile shaft in place of the ring beam and liner plate shaft originally included in the design. The main advantage of secant piles was that they eliminated the need for curtain grouting, thereby eliminating the risk of grout frac-out into the reservoir. Also, the installation of secant piles meant that there would not be any open ground during the excavation of the shaft. The owner and designer approved the change and the secant pile shaft was constructed. The installed shaft was constructed of 27 overlapping 36 inch secant piles to a depth of 17.5 m (57 ft) with 23 piles drilled to a depth of 8 m (27 ft) to allow for access. Only 11 of the secant piles for the access structures were reinforced with I beams with all of the shaft secant piles being unreinforced. Elevation and plan view of the shaft can be seen in Figure 5 and Figure 6, respectively.

Access to the shaft was provided with secured ladders from the surface and the upper excavation. Excavation was first carried out with a mini-excavator from the upper excavation for as long as it could reach and completed by hand excavation with rock splitters, as needed, for the final depth. Aside from some light seepage groundwater was not encountered during shaft excavation.

Ultimately, it was not a technical requirement that determined the shaft method that was selected but one that reduced the project's risk of encountering a hazard.

New Irvington Tunnel, Vargas Shaft

The New Irvington Tunnel is part of the San Francisco Public Utilities Commission's (SFPUC) Hetch Hetchy Water System. The System was built in the early 20th century and many critical components of this system have reached the end of their useful life and are vulnerable to seismic activity. The repair, replacement, and seismic upgrades taking place are part of the SFPUC's $4.6 billion Water System Improvement Program (WSIP) and are crucial to the Bay Area's economic viability and the public health and safety. The approximately 5.6 km (3.5 mile) New Irvington Tunnel will house a new water pipeline that will transport drinking water into the San Francisco Bay Area.

The Vargas shaft, located in Fremont, CA, acts as an intermediate heading for tunnel excavation. Shaft geology included fill and alluvium overlaying bedrock consisting of very weak siltstone and weak to moderately strong sandstone (New Irvington

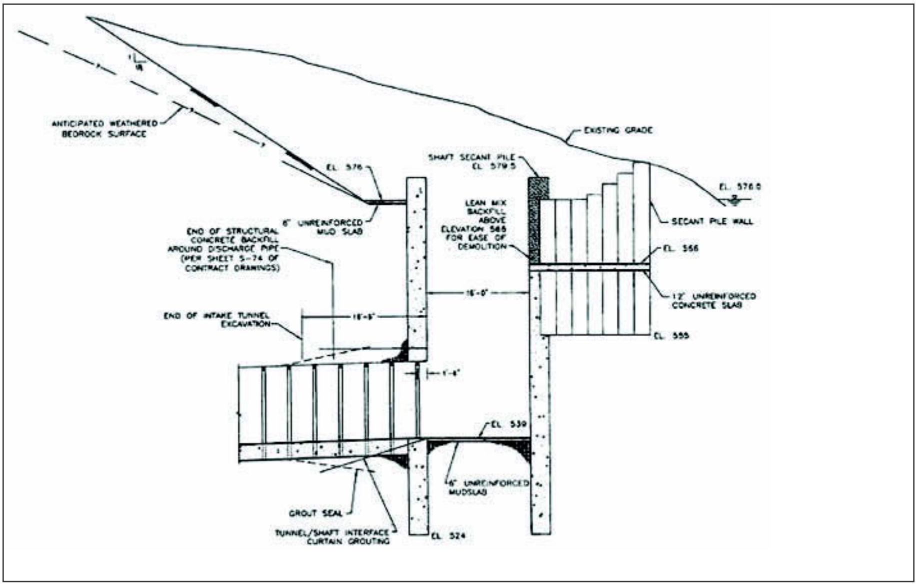

Figure 5. Elevation view of Lenihan Dam intake shaft

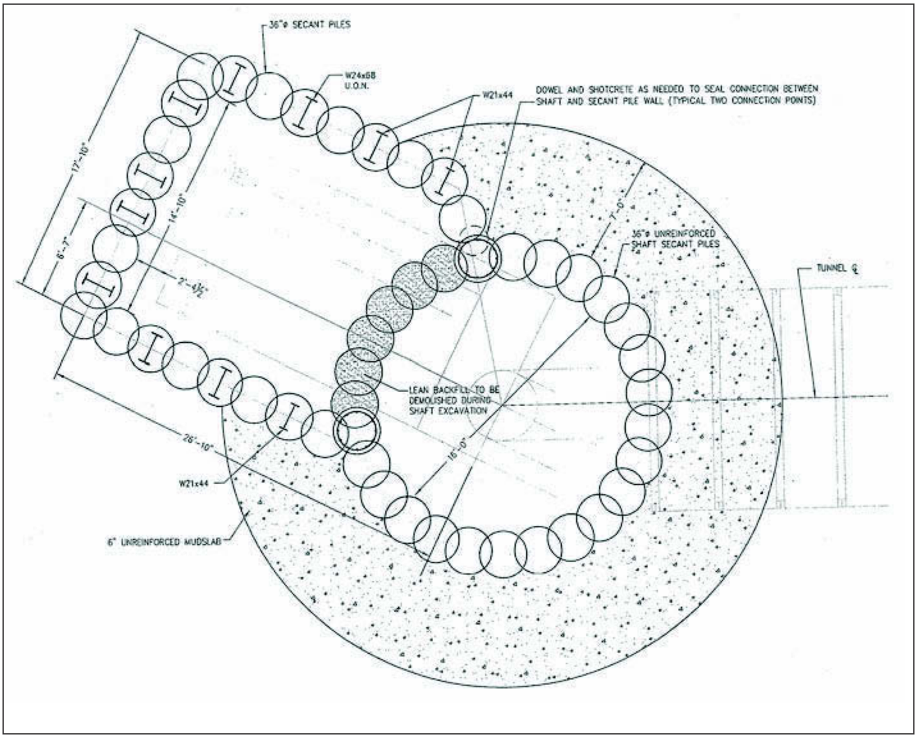

Figure 6. Plan view of Lenihan Dam intake shaft

Table 2. New Irvington Tunnel

Owner	San Francisco Public Utilities Commission
Shaft contractor	Malcolm Drilling Company, Inc.
Shaft purpose	Intermediate heading, temporary works
Size	12.5 m (41 ft) diameter 35 m (115 ft) deep
Selected support method	Secant Piles
Primary design criteria	Groundwater table Geologic conditions

Tunnel GBR, 2009). Table 2 presents a summary of the Vargas Shaft. A discussion of the construction follows.

The original design for the shaft was to be a combination of secant piles and rock bolts and shotcrete. During initial drilling it was determined that it would be challenging to construct the shaft as designed based upon soil conditions encountered and it was agreed to continue installation of secant piles for the entire depth of the shaft (Lindquist and Jameson, 2011). The 35 m (115 ft) deep, 12.5 m (41 ft) internal diameter shaft was constructed with 76, 1 m (3.3 ft) unreinforced secant piles. Elevation and plan views of the shaft can be seen in Figure 7 and Figure 8, respectively. Because of the potential for vertical drifting of the secant piles, verticality of the piles was checked prior to concrete placement. Excavation of the shaft core was conducted with an excavator and a bucket. Upon completion of the tunnel excavation and pipeline installation the shaft will be backfilled.

Calaveras Dam Replacement Project, Intake Shaft

The Calaveras Dam Replacement Project is another component of the SFPUC's $4.6 billion WSIP. In this project, a new earth and rock fill dam will be built to replace the original dam built in 1925. This required the construction of a new intake tower which is to be built above a new intake shaft.

Geology at the shaft included greywacke, siliceous schist, and mélange shale (GDR Calaveras Dam Replacement Project, 2008). The groundwater table was encountered during exploratory drilling at a depth of 12.5 m (41 ft) below the surface but was not encountered during construction. Table 3 shows a summary of the Calaveras Dam Replacement Project intake shaft. A discussion of the construction follows.

The intake shaft was required to be 6.7 m (22 ft) in diameter after initial support and provide a minimum finished diameter of 6.1 m (20 ft) and was excavated to a depth of 43 m (141 ft). The final shaft lining is to be reinforced concrete with initial support being provided by rockbolts and shotcrete. The plan and elevation view of the shaft can be seen in Figure 9 and Figure 10, respectively.

The upper portion of the shaft was primarily excavated with mechanical excavation and the lower portion of the shaft requiring drill and blast methods. Each lift was approximately 1.5 m (5 ft) high and proceeded with the following sequence: (1) drill out round, shoot, (2) excavate half to two-thirds of the lift in order to more easily drill for rock bolts, (3) drill for and grout in rock bolts, (4) complete excavation of spoil, (5) apply shotcrete. And for mechanical excavation: (1) excavate half to two-thirds of the lift in order to more easily drill for rock bolts, (2) drill for and grout in rock bolts, (3) complete excavation of spoil, (4) apply shotcrete. Excavation of spoil material completed with mini-excavator. Shaft access was provided with a man-cage.

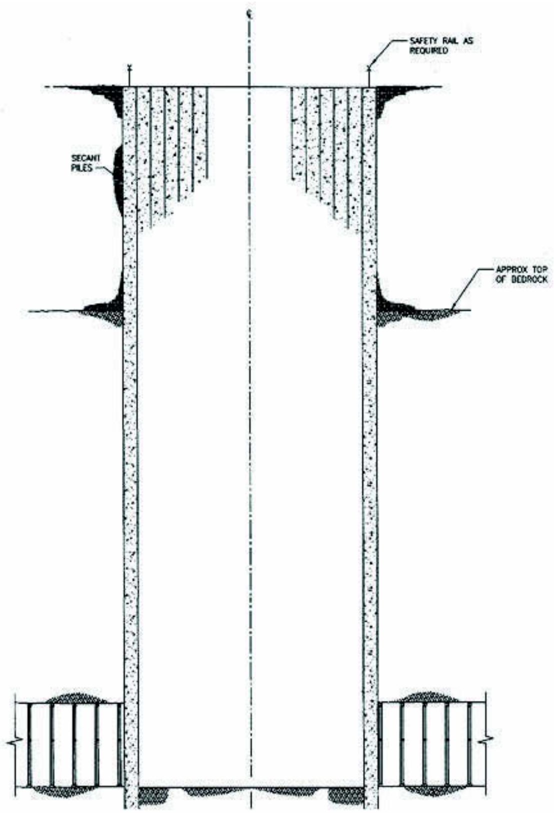

Figure 7. Elevation view of Vargas shaft

Bay Tunnel, Launching and Receiving Shafts

The Bay Tunnel Project is a $215.3 million portion of the SFPUC's $4.6 billion WSIP Hetch Hetchy Water System Improvement. The purpose of the project is to supply water to the San Francisco Bay Area using a pipeline encased in a tunnel under the bay to replace an aging pipe network which spans the San Francisco Bay from Newark, CA to East Palo Alto, CA. The project includes two shafts used to carry water in a pipe line under the San Francisco Bay. Table 4 and Table 5 show a summary of the Bay Tunnel launching and receiving shafts, respectively. A summary of construction follows each table.

The two shafts are located near the San Francisco Bay in an environmentally sensitive area and within miles of the San Andreas and Hayward faults in the loose Bay sediment. These faults have been determined to have a maximum credible earthquake of magnitude 7.9 along the San Andres Fault located nearest to the western launching shaft (Ravenswood shaft) and a 7.1 magnitude earthquake located near the eastern receiving shaft (Newark shaft). The shafts are excavated down to the San Antonio Formation Complex, which is a continuous formation within the 8 km (5 mile) long tunnel alignment under the Bay, containing mostly sandy and silt clay.

The Ravenswood shaft is designed with an inside diameter of 18 m (58 ft) and has a finish invert depth of 38 m (124 ft) with slurry wall panels extending to 43 m (141 ft)

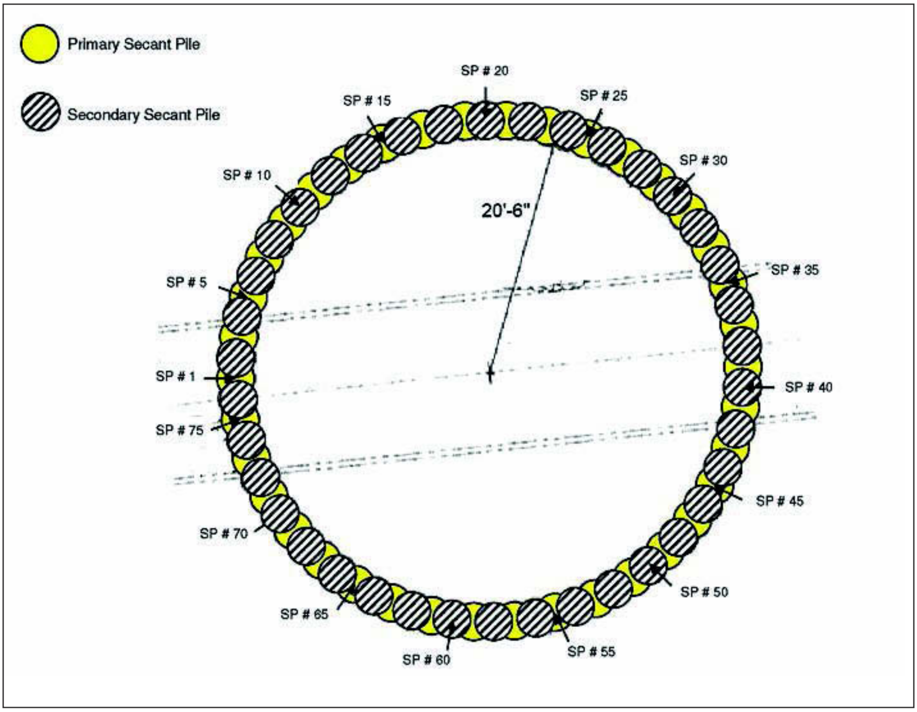

Figure 8. Plan view of Vargas shaft

Table 3. Calaveras Dam replacement project

Owner	San Francisco Public Utilities Commission
Shaft Contractor	Drill Tech Drilling and Shoring, Inc.
Shaft Purpose	Intake shaft, permanent works
Size	6.7 m (22 ft) diameter 43 m (141 ft) deep
Selected Support Method	Rockbolts and shotcrete
Primary Design Criteria	Geologic conditions

below the surface. The shaft excavation began August 2010 and was completed May 2011 using a diaphragm slurry wall shaft which was constructed using 3 foot wide panels. At a depth of 11 m (35 ft), the shaft was flooded with potable water. The remaining depth was excavated with the shaft nearly full of water to resist exterior groundwater pressure (Labonte, 2012). The contractor placed two 18 metric ton (20 ton) rebar cages with a steel support frame within two slurry panels used as additional support for the TBM breakout from the shaft. A 3.7 m (12 ft) thick tremie concrete invert was placed with two rebar mats tied into the vertical slurry wall cages to resist the uplift force since the majority of the shaft is located below the ground water table. A photograph during shaft construction is presented in Figure 11. The Ravenswood shaft is located approximately 152 m (500 ft) from the San Francisco Bay, the ground water table at 1 m (3.5 ft) below the surface. The top of the shaft was excavated through recent alluvium deposits and fill at the shaft collar of 7 ft elevation above mean seal level. The alluvium deposit overlays very soft to very stiff Young Bay Mud and the San Antonio Formation underlies

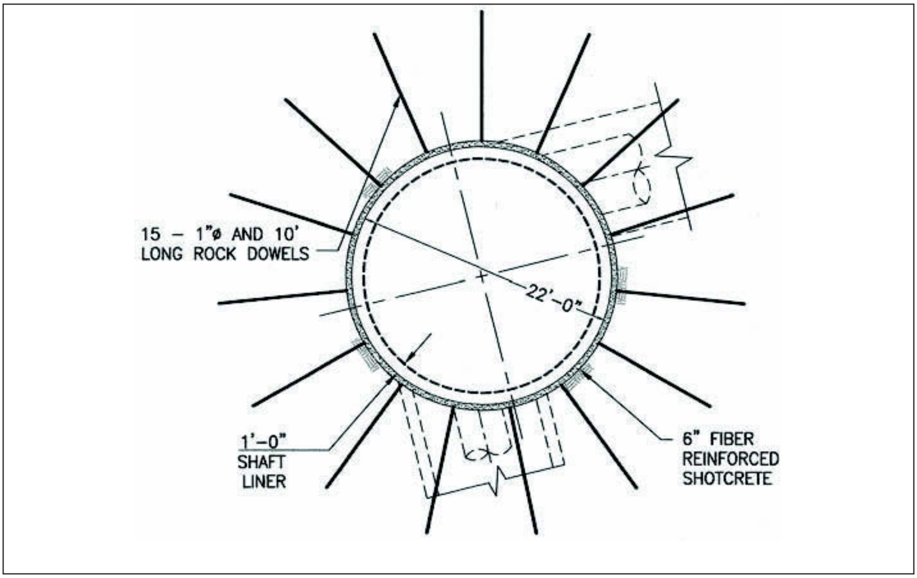

Figure 9. Plan view of Calaveras Dam intake shaft

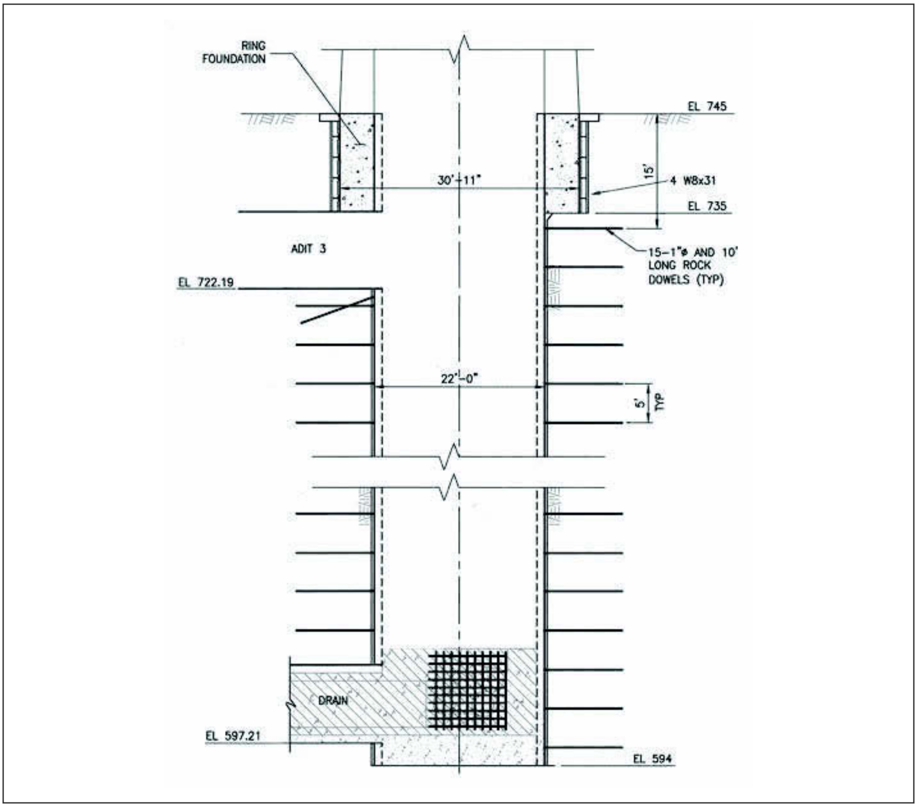

Figure 10. Elevation view of Calaveras Dam intake shaft

Table 4. Bay Tunnel–Ravenswood (launching shaft)

Owner	San Francisco Public Utilities Commission
Shaft contractor	Michels/Jay Dee/Coluccio Joint Venture
Shaft purpose	Launching shaft, temporary works, permanent connection
Size	18m (58 ft) diameter 38 m (124 ft) deep
Selected support method	Diaphragm Slurry Wall Shaft
Primary design criteria	Groundwater and Geologic Conditions

Table 5. Bay tunnel (receiving shaft)

Owner	San Francisco Public Utilities Commission
Shaft contractor	Michaels /Jay Dee/Coluccio Joint Venture
Shaft purpose	Receiving shaft, temporary works, permanent connection
Size	11.3 m (28 ft) diameter 26 m (86 ft) deep
Selected support method	Freezing Shaft
Primary design criteria	Groundwater, environmental and geological conditions

the Young Bay mud for the remainder of the shaft. The San Antonio formation varies from medium dense/dense sand to sandy clay to soft/stiff Old Bay Mud (Wong et al., 2011).

The Newark shaft was constructed using a ground freezing technique to create a watertight seal. The shaft has a 11.3 m (28 ft) in diameter and 26 m (86 ft) deep (Labonte et al., 2012) and is supported using ring beams and timber lagging as initial support. The shaft is constructed through similar soil conditions as the Ravenswood shaft, however the Newark shaft does not encounter the Young Bay Mud. The contractor used freezing pipes to form a 8.5 m (28 ft) diameter circle at the Newark shaft location. The ring includes 50 freeze pipes, 39 m (128 ft) long around the shaft diameter

Figure 11. Bay Tunnel–Ravenswood shaft using diver to inspect and connect steel reinforcement. Photo courtesy of the Global Diving and Salvage.

creating a frozen matrix allowing the contractor to excavate through soft soil and high water inflow that would have occurred (Figure 12). Near the middle of the freeze shaft a circle of 10 longer freezing pipes reaching 49 m (160 ft) were installed (SFPUC Fact Sheet). A seal consisting of a 4.5 m (15 ft) diameter and 3 m (10 ft) long steel pipe and a rubber blowup seal to prevent the ground material to enter the shaft is used as the TBM enters the shaft.

Figure 12. Bay Tunnel–Newark shaft freezing with ring beams and wooden lagging support. Photo courtesy of the SFPUC.

Sunol Valley Water Treatment Plant, Launching and Receiving Shafts

The Sunol Valley Water Treatment Plant (SVWTP) Expansion is another component of the SFPUC's $4.6 billion WSIP. The SVWTP Expansion included a 121 m (400 ft) microtunneling drive with a fixed profile to accommodate the required hydraulic profile requiring both a launching and receiving shaft. Geology at both shafts consisted of sand with gravel and cobbles over sandstone with a high water table. Table 6 and Table 7 show a summary of the SVWTP launching and receiving shafts, respectively. A discussion of the construction follows.

In order to select a jacking (launching) shaft shape and size the shaft subcontractor consulted with the Micro-Tunnel subcontractor to first determine space requirements for the jacking equipment. In this particular case, the Microtunnel contractor required a large, open span 10 m (32 ft) to install the microtunnel equipment. A round shape was selected because of its ability to deliver a large open span with a minimum initial support cost. Additionally, the added working area provided by the "excess" diameter provided enough of a benefit to warrant the round shape. For the contractor, some consideration is always given to what equipment is owned and readily available when making decisions about methodology. The shaft subcontractor elected to use cutter soil mixing (CSM) panels for the launching shaft. Rock bolts and shotcrete were also used in the shaft below the CSM refusal elevation. The excavated internal diameter of the jacking shaft was10.6 m (35 ft) and the excavated depth was 14 m (47 ft) and CSM panels were installed to a depth of 15.8 m (52 feet). Figure 13 and Figure 14 show the elevation and plan view, respectively, of the launching shaft.

In order to select a receiving shaft shape and size the shaft contractor consulted the microtunnel contractor to establish space requirements to permit retrieval of the micro-tunnel machine. The microtunnel contractor required a span of 4.5 m (15 ft) to recover the equipment from the receiving shaft. The contractor decided that a 15 foot diameter round shaft could be excavated using drill and auger equipment, and used its preference for using certain equipment as a primary criteria for choosing a support method. The contractor elected to install a secant pile wall shaft for initial support in part

Table 6. Sunol Valley Water treatment plant (launching shaft)

Owner	San Francisco Public Utilities Commission
Shaft contractor	Drill Tech Drilling and Shoring, Inc.
Shaft purpose	Launching shaft, temporary works
Size	10.4 m (34 ft) diameter 14.3 m (47 ft) deep
Selected support method	Cutter Soil Mixing
Primary design criteria	Geologic conditions

Table 7. Sunol Valley Water Treatment Plant (Receiving Shaft)

Owner	San Francisco Public Utilities Commission
Shaft contractor	Drill Tech Drilling and Shoring, Inc.
Shaft purpose	Receiving shaft, temporary works
Size	4.6 m (15 ft) diameter 23.8 m (78 ft) deep
Selected support method	Secant Piles
Primary design criteria	Groundwater table Microtunnel equipment size Site footprint Contractor preference

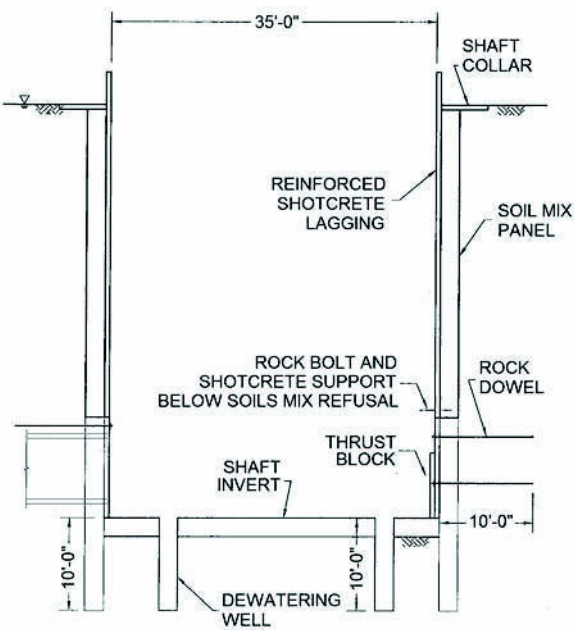

Figure 13. Elevation view of SWTP launching shaft

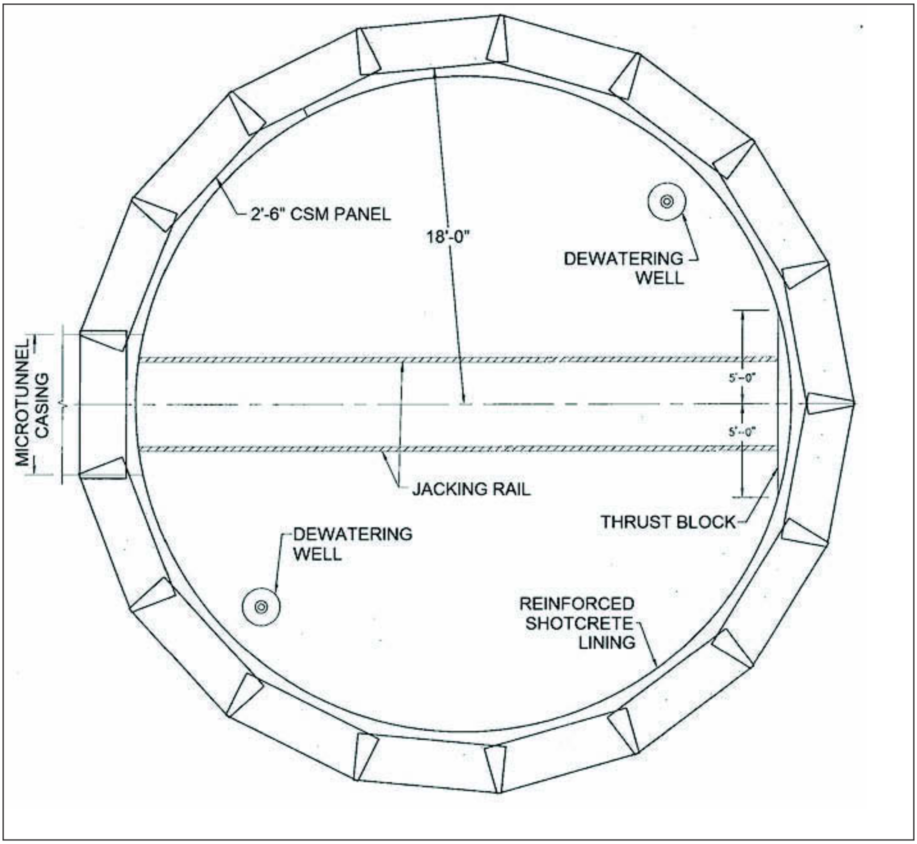

Figure 14. Plan view of SVWTP launching shaft

because the required staging area was smaller which was a benefit for the given site footprint. The shaft 71, 91 cm (28, 36 inch) diameter piles, drilled to a planned depth of 26.3 m (86.25 ft). After the installation of initial support the contractor used a 4.5 m (15 ft) diameter auger to excavate the shaft. Figure 15 and Figure 16 show the plan and elevation view, respectively, of the receiving shaft. Table 8 presents a summary of the project information and construction data gathered for this paper.

CONCLUSION

This paper provided a summary of shaft techniques and illustrates that while different shaft construction methods are intended for specific conditions, often times multiple methods are appropriate for use on a particular project. Shaft design considerations such as shape, size, depth, geologic conditions, safety, cost, and contractor preference play a considerable role in determining the shaft construction technique that is best suited for each situation. Designers, owners, and contractors must also consider other criteria to make an educated final decision. These criteria can include the non-technical considerations and constructability concerns discussed in this paper. Because shaft construction methods proposed by designers are not always implemented by contractors, as shown by the Lenihan Dam and New Irvington Tunnel projects, designs and specifications should be flexible enough to accommodate changes for to site conditions, alternative means and methods, and the use of several shaft techniques proposed by

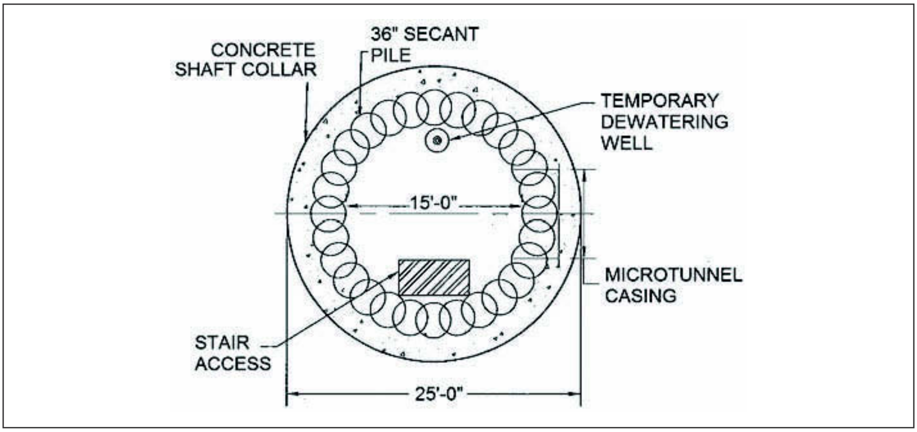

Figure 15. Plan view of SVWTP receiving shaft

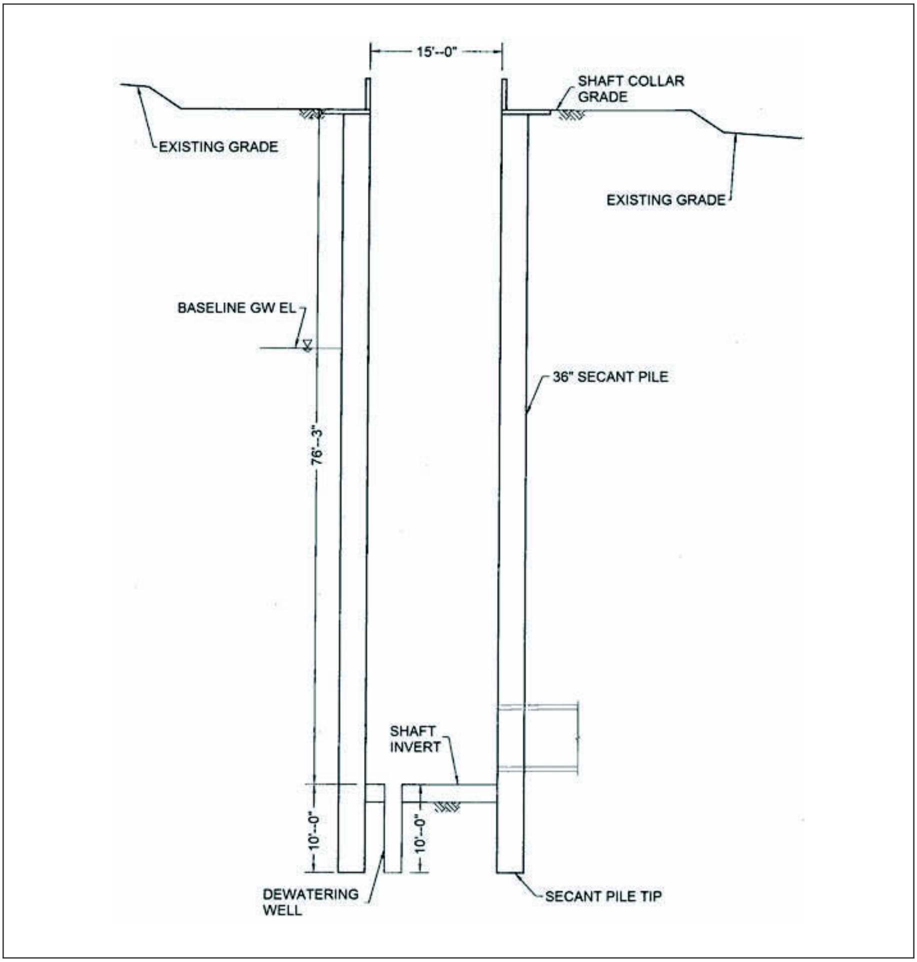

Figure 16. Elevation view of SVWTP receiving shaft

Table 8. Summary of shaft construction data

Shaft Name	Function	Total Depth Supported/ Invert	Construction Method	Geology (Description)	Shaft Diameter Outside/Inside	Average Excavation Rate (CM/shift) (CY/shift)	Average Support Installation Rate	Below Groundwater Table
Lenihan Dam Outlet Modification Project	Reservoir Intake	16.9/12.3 m 55.5/40.5 ft	Unreinforced Secant Piles for round portion, reinforced for square portion	Weathered Bedrock with a ucs range of 0.15 to 138 MPa (20 to 20,000 psi)	6.7/4.9 m 22/16 ft	24.5 CM/shift 32.0 CY/shift (10 hour shift)	34.7 m/day 114 ft /day	Yes, 1 meter (3 ft) below shaft collar
New Irvington Tunnel, Vargas Shaft	Access/ Production Shaft	35.7/35.0 m 117.0/115.0 ft	Unreinforced Secant Piles	Soil/Bedrock: Sand, and Silt /Weathered Siltstone and Sandstone	14.5/12.5 m 47.6/41 ft	109.9 CM/shift 143.7 CY/shift (12 hour shift)	73.2 m of piles drilled/shift 240 ft of piles drilled/ shift	Yes, 2.3 m (14 ft) below shaft collar
Calaveras Dam Replacement Project, Intake Shaft	Access for construction and reservoir intake	43.0/42.1 m 141.0/138.0 ft	Rock Dowels and Shotcrete	Bedrock: Greywacke, schist, and shale	6.7/6.1 m 22/20 ft	13.4 CM/shift 17.5 CY/shift (10 hour shift)	Installed During Cycle Time	No, not encountered during construction
Bay Tunnel, Ravenswood Shaft	Launching/ Production/ Shaft	43.0/37.8 m 141.0/124.0 ft	Reinforced Diaphragm Slurry Wall	Soil: Alluvium, Bay Mud, Sand, and Clay	19.5/17.7 m 64/58 ft	Unavailable at time of publishing	Unavailable at time of publishing	Yes, 1 m (3 ft) below shaft collar
Bay Tunnel, Newark Shaft	Receiving/ Drop Shaft	48.8/26.4 m 160.0/86.0 ft	Ground Freezing and ring beams with timber lagging	Soil: Alluvium, Bay Mud, Sand, and Clay	10.4/8.5 m 34/28 ft	Unavailable at time of publishing	Unavailable at time of publishing	Yes, 1 m (3 ft) below shaft collar
Sunol Valley Water Treatment Plant, Launching Shaft	Production/ Jacking Shaft	15.8/14.3 m 52.0/47.0 ft	Cutter Soil Mixing and Shotcrete	Soil/Bedrock: Sand with Gravel and Cobbles over Sandstone	11.0/10.4 m 36/34 ft	71.2 CM/shift 93.1 CY/shift (12 hour shift)	14.5 m of panel/ shift 47.6 ft of panel/ shift	Yes, at shaft collar
Sunol Valley Water Treatment Plant, Receiving Shaft	Receiving Shaft	26.3/23.8 m 86.25/ 78.0 ft	Secant Piles	Soil/Bedrock: Sand with Gravel and Cobbles over Sandstone	5.5/4.6 m 18/15 ft	18.6 CM/shift 24.3 CY/shift (10 hour shift)	35.0 m of piles drilled/shift 115 ft of piles drilled/shift	Yes, 6 m (20 ft) below ground surface

contractors, in order to minimize project cost and risk. It is important to incorporate local expertise and shaft construction techniques during the design process in order to select an appropriate method for a shaft's design.

By limiting the case studies to the San Francisco Bay Area since 2007, the selection of projects taking place in a relatively small geographical region and in a relatively short period of time, it clearly shows that shaft technology used still varies greatly. Using the key considerations for shaft design and construction can help owners, engineers, designers, and contractors evaluate the best shaft construction technique for minimizing risk and cost while optimizing the safety and accessibility for future shaft construction projects.

ACKNOWLEDGMENTS

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BLUE PLAINS TUNNEL: DESIGN AND CONSTRUCTION OF LARGE-DIAMETER DIAPHRAGM WALL SHAFTS

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ABSTRACT

The Blue Plains Tunnel (BPT) is a design-build project for the District of Columbia Water and Sewer Authority (DC Water). BPT is one of the three main tunnels being implemented for the collection of combined sewer overflows to be treated by DC Water's Blue Plains Advanced Waste Water Treatment Plant, the world's largest advanced waste water treatment plant.

Upon completion of the in-depth analysis and design, two combined shafts in a figure-8 configuration will be constructed through soft ground. The figure-8 will consist of a 76-foot diameter screening shaft and 132-foot diameter dewatering shaft approximately 170 to 190-foot deep. The shafts will be constructed through fill, alluvium, and very stiff to hard silt and clays, by means of slurry diaphragm wall for the support of excavation. The slurry walls will be excavated to approximately 194-feet. The screening shaft will provide the ability to coordinate all the tunneling operations including the launch of the earth pressure balance tunnel boring machine.

INTRODUCTION

The BPT project is part of a large scheme to reduce the discharge of combined sewer overflows into the local water ways in the District of Columbia and surrounding areas which is referred to as the Long Term Control Plan (LTCP). The LTCP is DC Water's plan for controlling the combined sewer overflows, which when complete will include a new 12.7 mile long tunnel system and comply with the Clean Water Act.

The system is comprised of four large diameter tunnels through the soft ground of the District at a depth of approximately 100-foot. Upon the LTCP approval by the Environmental Protection Agency, the United States, the District of Columbia, and DC Water entered into a Consent Decree to be implement and completed by 2025. The entire program is estimated at approximately $2.4 billion. The program will reduce the combined sewer overflows by 96% which will benefit the environment, local waterways, and the District of Columbia and surrounding area residents.

PROJECT DESCRIPTION

The BPT is one of the major tunnels of the LTCP. The BPT project consists of five shafts, near surface structures, and tunnel. The project consists of two large diameter shafts at the DC Water Waste Water Treatment Plant that will be utilized to launch the soft ground Tunnel Boring Machine (TBM). The two shafts are the BPT screening shaft (BPT-SS) at 76-foot diameter and the BPT dewatering shaft (BPT-DS) at 132-foot diameter. The TBM will drive a 23-foot internal diameter tunnel approximately 24,000-foot long. The project also consists of an overflow drop shaft at the Joint Base Anacostia Bolling site owned by the Navy, a combination drop/junction shaft at Poplar

Point which will be the receiving shaft for the future Anacostia River Tunnel, and the termination of the BPT tunnel at the Main Pumping Station Drop Shaft near 2nd Street and Tingey St. SE (Figure 1).

Traylor Skanska Jay Dee JV (TSJD) was awarded the BPT Division A Contract on May 5th, 2011 and the project is to be completed by November 2015. TSJD partnered with Halcrow Inc. for the design of the project. TSJD joined with Bencor for the construction of the slurry wall system. The BPT will be constructed by an Earth Pressure Balance (EPB) TBM. The tunnel will be lined with bolted, gasketed precast concrete segments to meet the required 23-foot diameter requirement (Figure 2).

Figure 1. Blue Plains tunnel alignment

GROUND CONDITIONS

Fourteen borings were drilled in the vicinity of the proposed DS and SS shafts during the owner's ground investigation program ranging from 53 ft to 392 ft below ground surface using sonic, mud rotary and direct rotary drilling methods.

Four main geologic formations have been defined and described below:

- Fill
- Alluvium
- Potomac Group–Patapsco/Arundel Formation–KP (P/A)
- Potomac Group–Patuxent Formation–KP (PTX)

Fill

The BPT site was pre-excavated by a previous contract to clear the previous structures and any potential obstructions. Subsequently fill materials were placed covering the surficial site. Prior to pre-clearing of the site the typical description of the fill deposits is fine to coarse silty sand with gravel, and sandy lean clay (sometimes organic) containing fragments of construction debris, including wood, concrete and metal. The thickness of fill is somewhat variable over the area of the shaft and ranges between 25 ft to 36 ft.

Alluvium

Younger alluvial soils are present at the shaft location. The Alluvium contains the full range of soil types from clay to gravel, with cobbles and boulders possible most likely to occur on the contact with underlying Potomac Group soils. The Alluvium is generally weaker and less dense than the Potomac Group soils, because the Alluvium has mostly consolidated under only its own weight or a modest thickness of fill. The total alluvium

Figure 2.

thickness ranges between 17 feet at the BPT-DS and 25 feet at the BPT-SS cell.

Potomac Group (P/A and PTX)

Underlying the alluvium, the majority of the shaft is located in dense and hard soils of the Potomac Group. These seem to be the same general P strata as encountered in the Washington Metro tunnel construction. These soils range from stiff to hard silt/clay to sand and gravel, which have been over-consolidated under the weight of hundreds of feet of overburden which have since been removed by erosion. Two different Potomac formations have been identified:

- The upper Patapsco/Arundel (P/A) formation is primarily silt/clay materials with thin interlayers of granular sand/gravel soils.
- The lower Patuxent (PTX) formation is mostly sand/gravel with interlayers of silt/clay. The Potomac clays, mostly G1 with some G2, are indicated to extend below the shafts to about Elev. –208, and are underlain by a thick sequence of mostly G3 and G4 Potomac sands (Figure 3).

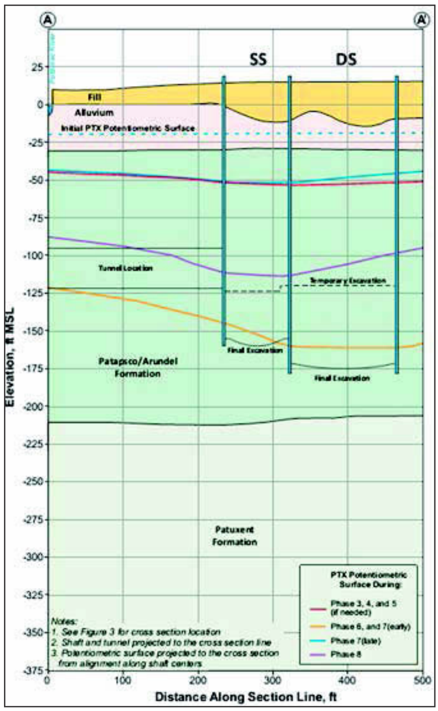

Figure 3. Idealized subsurface profile at the BPT DS/SS Site

The thickness of the PTX formation under the site has not been determined as wells or borings installed during investigations at the site penetrated only the upper 175 ft of this formation.

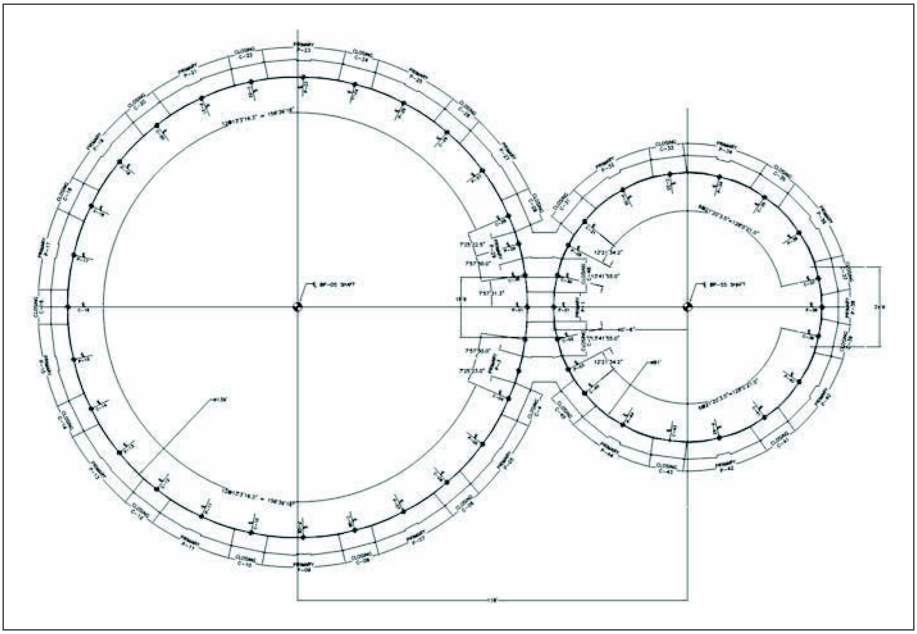

Figure 4. Shaft and diaphragm wall panel layout (plan view)

Groundwater

Piezometric levels were monitored within the vicinity of the site in observation wells in completed borings and from vibrating wire piezometers installed in the completed borings. The piezometric level in the Fills and Alluvium (Upper Aquifer) ranges between 20 ft to 30 ft below ground surface while the piezometric surface of the PTX (Lower Aquifer) at the shaft locations is at about 35 ft below ground surface, at an elevation of about –20 ft MSL.

DESIGN CONSIDERATIONS

General

Two primary methods of excavation support for the deep shafts have been considered during the tender stage. While ground freezing was deemed a viable option for the excavation support, diaphragm walls were selected during the final design. The design for the de-watering and screening shaft eliminated the need for a separate interconnector tunnel between the shafts. This was achieved by eliminating the separation of the shafts, placing them back-to-back with a common section of wall between them.

The design of the BPT-DS and BPT-SS shafts using a dual-cell slurry wall shaft configuration requires a 5-ft-thick diaphragm wall to enable the excavation of the shaft without the need for installation of the CIP liner as excavation proceeds (top down final lining construction). The minimum wall thickness at the joint shaftwall is 8 ft.

The larger cell is 132 ft in diameter in accordance with the specified requirements; however, the small cell has an oversized internal diameter of 76ft which is based on the space requirement for launching the TBM (See Figure 4). The excavation depths range between 170 ft and 190 ft below ground surface (See Figure 5).

The ground loadings used to design the diaphragm wall was based on the temporary conditions (including the 100 year flood level of EL +11) up until the final CIP liner is installed and the permanent ground loads and 500 year flood level can be shared.

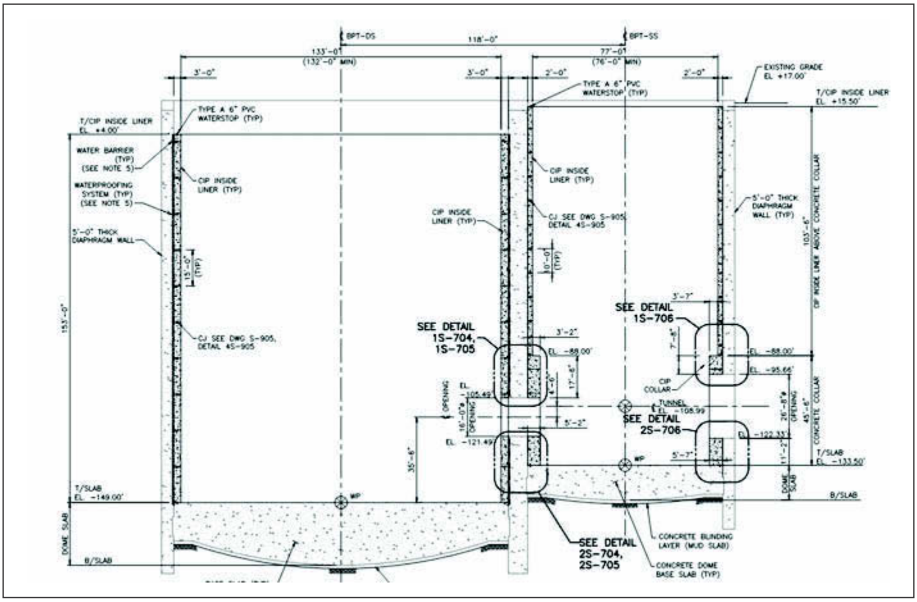

Figure 5. Shaft section

Durability Study

Since the diaphragm wall is part of the permanent structure, its durability needs to meet the required design life of 100 years. A detailed durability assessment was performed and the following parameters determined to be sufficient to achieve the design life:

- Minimum cementitious material content of 575 lb/yd^3, with a w/cm ratio of 0.4 and high slag/fly ash replacement.
- Minimum cover requirement of 2 inches.

Earth Pressure Study

The shaft is located in heavily over-consolidated soils of the Potomac Formation that exhibit high in-situ lateral stresses (ko =1.2 to 1.4, DC Water, 2011). The shaft wall was designed to withstand earth pressures during the temporary construction stages and the operational condition. Detailed numerical calculations were performed in order to refine the earth pressure coefficients for soils in the Potomac group for the temporary earth retention system at the BPT DS/SS shaft, in particular the use of a modified “short-term” ko values of 0.86 to the design of the diaphragm walls.

The accurate assessment of the earth pressure distribution is essential to the design of large and deep shafts in soils such as the proposed Blue Plains DS/SS shaft. Construction of diaphragm wall panels causes considerable stress changes in over-consolidated soil deposits and induces ground movement reducing the at-rest earth pressure (see Figure 6). Recommendations in the literature, in particular industry guidelines developed in the UK (Gaba, 2003), suggest that realistic bending moments for diaphragm walls can be obtained assuming a lateral earth pressure coefficient of unity (K0 = 1.0) prior to excavation even for over consolidated clays with coefficient of earth pressure at rest K0 well above 1.0. Further reductions, which would result in earth pressure coefficients below unity, are justified by detailed analysis, like the finite element calculations performed for this project.

For the diaphragm wall shaft, two different construction processes are of particular importance. The trench excavation procedure and simultaneous slurry support, which stabilize the trench, and the concreting procedure, which takes place once the reinforcing cage has been installed in the slurry-filled trench. In order to estimate reliably earth pressure redistribution, numerical calculations have been made using the finite element method (Figure 7). The diaphragm wall panels are instrumented with inclinometers, total stress cells (see Figure 8), rebar strain gauges and concrete strain gauges to monitor the performance of the structure.

The construction processes of the slurry wall panel are simulated in the finite element calculation with the following multiple steps:

- The in-situ stress state is calculated based on an at-rest coefficient at the upper boundary of the values given in the GBR.
- Stepwise excavation of the slurry wall panel and simultaneous filling with bentonite. The bentonite/polymer slurry is simulated by means of an artificial 'water' pressure that increases linearly with depth.
- The entire excavated trench is filled with wet concrete. The wet concrete is simulated by change in the artificial water pressure. A stepped concrete pressure model is used to reflect the consolidation and hydration process.
- The hardening of the concrete is simulated by removing the artificial pressures, reactivating the excavated clusters and assigning the concrete material set to these clusters. Assuming that no cracking occurs during the construction process, the diaphragm wall is described by a linear-elastic constitutive model.
- Consolidation analysis is used to calculate the time-dependent generation or dissipation of excess pore pressures.

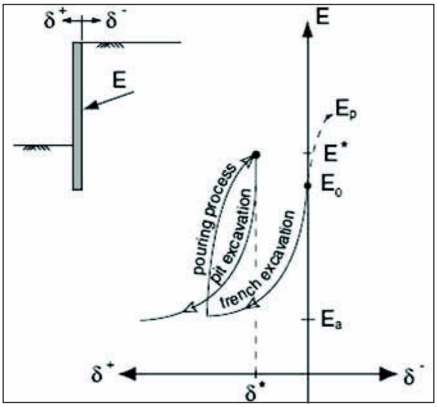

Figure 6. Generalized earth pressure distributions at different stages (Triantafyllidis, 2004)

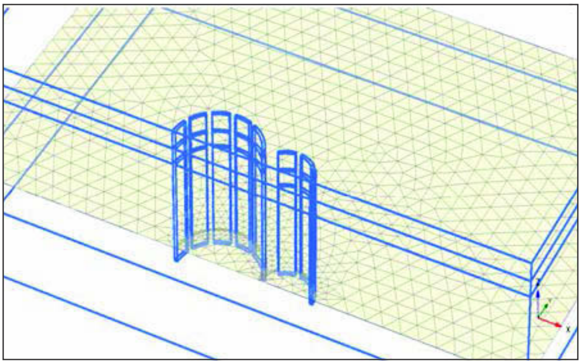

Figure 7. 3D Numerical model (Plaxis 3D)

- Simulation of shaft excavation by excavation the SS shaft portion first to temporary level, followed by the DS shaft excavation to temporary level. As part of the study the time dependency of excess pore pressures and their dissipation after wall installation processes is analyzed.

Diaphragm Wall Design

The diaphragm panel layout consists of approximately 25 foot long primary panels and around 9 foot closing panels, with a minimum width of 60". The primary panels are comprised of two complete bites and one shorter middle bite. The panel layout is shown in Figure 4. Larger primary panels have been selected to reduce the amount of joints, but also to reduce the earth pressures acting on the panel due to a larger relaxation of the ground in comparison to a sequence of small primary and secondary panels. The panels in the wall that joins both shafts required multiple bites.

The thrust, horizontal and vertical bending moments in the panels were determined by analyzing the shaft planned geometry using the PLAXIS 3D software. The calculated internal thrust and moments were used to design the section using moment-thrust interaction diagram for the reinforced panels in accordance with ACI 318-08. The results from the finite element calculations were also compared against the "inscribed circle method." The specified slurry diaphragm wall panel's out-of-vertical tolerance results in a minimum panel contact of 54" (1.37 m) between adjacent panels. The bearing design between panels is based on a 54" thick (1.37 m) contact between panels.

ACI limit under section 10.3.6.2 specifies an axial load capacity reduction factor of 0.8 for compression members with tie reinforcement which accounts for minimum eccentricities that may exist in columns but not considered in column design. In the case of BPT-DS and SS shaft diaphragm wall design, these eccentricities due to out-of-vertical tolerance (10% of thickness) were considered in the design. The design is based on a polygon shaped wall with resulting internal compressive, hoop, axial, moment and shear forces and eccentric moments eccentricities due to construction tolerances. By designing for the eccentric loads, the 0.8 factor in ACI equation 10-2 was ignored as it is for the same purpose when the engineer does not design for eccentric compressive loads. Therefore this ACI limit on thrust and moment interaction diagrams will not apply. Nominal concentric load capacity of shell structure panels has been considered.

Design criteria used minimum temperature and shrinkage reinforcement ratio of 0.0018 for horizontal steel per ACI section 7.12.2.1 which is greater than 0.0015 required by ACI 14.3.2 for "vertical reinforcement" for walls calling compressive loads in the vertical direction. The figure-eight shaft arrangement incorporated additional reinforcement where the two shafts are connected so that the interconnecting tunnel can be formed without need for further internal propping or support.

Figure 8. Total stress cells mounted on rebar for diaphragm wall panel

Table 1. Diaphragm wall properties

Concrete Strength	Reinforcing Steel	Concrete Cover
fc’ = 7,000 psi (56 days)	fy = 60 ksi	6"

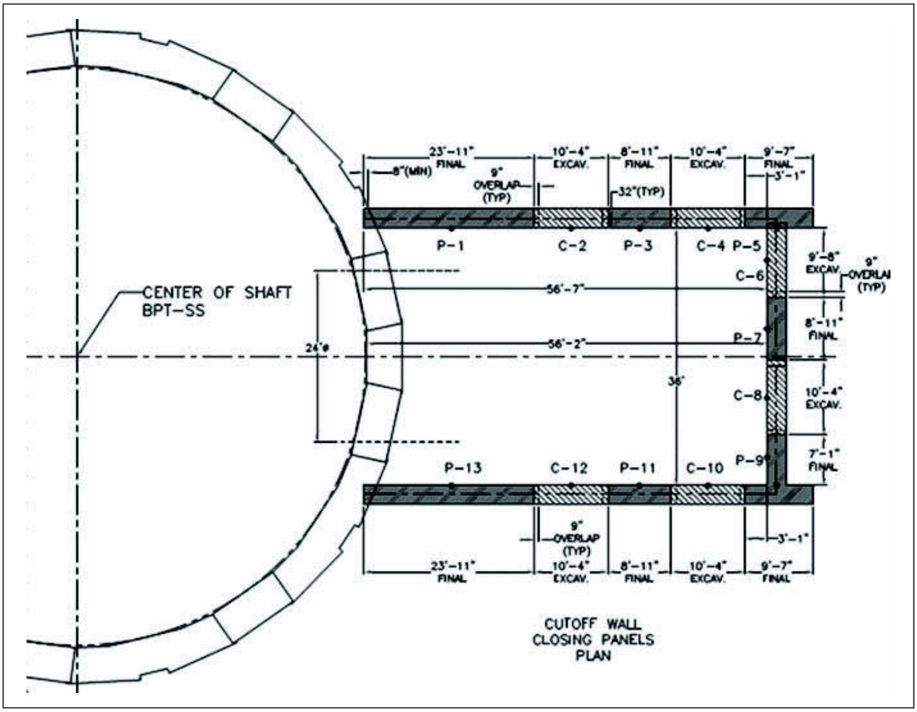

Figure 9. TBM launch box

For the diaphragm wall the main compressive loads are in the hoop, or horizontal direction, hence the vertical steel reinforcement ratio is provided per ACI section 14.3.3 (0.0025). The joints across primary and closing panels remain unreinforced for a total distance of 18". The main design parameters are provided in Table 1.

TBM Breakout Area

The breakout of the TBM from the Blue Plains Screening Shaft is a critical activity involving significant inherent risk, arising from the possibility of ground and groundwater ingress occurring into the shaft as the TBM breaks through the shaft diaphragm wall and enters the ground. To mitigate this risk a cutoff wall, consisting of unreinforced slurry wall panels was constructed, prior to the construction of the reinforced panels for the shaft support (Figure 9).

The TBM will be launched in the closed mode with the first penetration through the “soft eye” of the diaphragm wall, which contains fiberglass reinforcing.

CHALLENGES

The slurry wall’s size was a real challenge. The design required that the panels be constructed as 60" wide, requiring a minimum continuous wall of 54". This means a

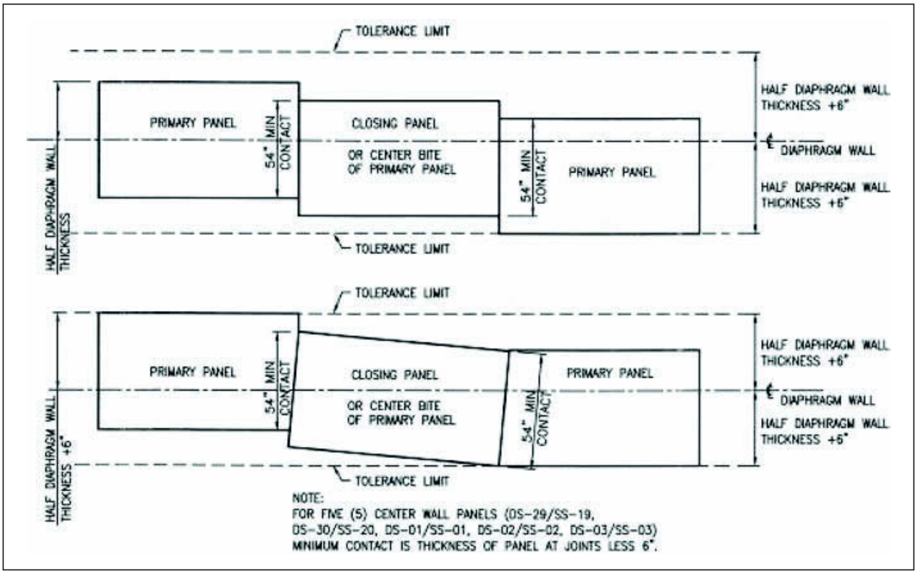

Figure 10. Panel installation tolerances

tolerance of 0.12% is required if each panel deviates in opposite directions. Also, the utilization of 2 hydromills and 2 clam buckets was required to perform the job.

Design Criteria and Configuration

The slurry walls for the BPT-DS have the following design criteria (Figure 10):

- Internal Diameter (ID)—Constructed 139.00 feet
- Internal Diameter (ID_{min})—Minimum 138.00 feet
- Outside Diameter (OD)—Constructed 149.00 feet
- Outside Diameter (OD_{min})—Minimum 148.00 feet
- Length 452.00 feet
- Width 60 inches
- Elevation of working platform +16.00 feet
- Elevation Top of Slurry Diaphragm Walls El. +14.00 feet
- Elevation Bottom of Slurry Diaphragm Walls El. –178.50 feet
- Depth of Excavation 194.50 feet
- Estimated Area Approx. 88,000 SF
- Number of Primary Panels 15
- Number of Secondary panels 15
- Concrete Compressive Strength @ 56 days 7,000 psi

The slurry walls for the BPT-SS have the following design criteria:

- Internal Diameter (ID)—Constructed 81.00 feet
- Internal Diameter (ID_{min})—Minimum 80.00 feet
- Outside Diameter (OD)—Constructed 91.00 feet

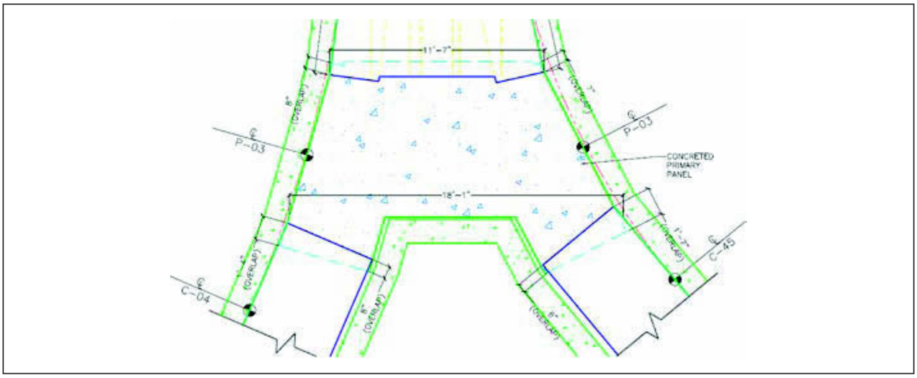

Figure 11. “Y panel”—shared primary panel by both shafts

- Outside Diameter (OD_{min})—Minimum 90.00 feet
- Length 270.00 feet
- Width 60 inches
- Elevation of working platform +16.00 feet
- Elevation Top of Slurry Diaphragm Walls El. +14.00 feet
- Elevation Bottom of Slurry Diaphragm Walls El. –160.00 feet
- Depth of Excavation 176.00 feet
- Estimated Area Approx. 47,551 SF
- Number of Primary Panels 7
- Number of Secondary panels 8
- Concrete Compressive Strength @ 56 days 7,000 psi

Panels for the connecting walls of both shafts (middle wall) are of a minimum thickness of 96 inches (8' or 2.44 meters) and maximum thickness of 217 inches (18' or 5.5 meters). See Figure 11. To our knowledge this was never done before, and several parties initially raised doubts about the feasibility of actually constructing what was designed. Bencor was confident that it could be done.

Excavation of Slurry Walls

As we know, the geotechnical formation that the slurry walls were installed into was mainly in clay soils, which caused another big challenge of the slurry management. Centrifuges were added to the slurry plant and desilters to deal with the large amount of fines.

A few new slurry systems were experimented with that are new to the slurry walls industry. Bencor utilized a slurry blend consisting of bentonite and polymers; also utilized was a gypsum based mud slurry to inhibit the swelling and hydration of the clays. Each slurry system was tried, and it was a big challenge as the amount of fines associated with such large volumes was not an easy job to control.

The existence of clay in such amounts also presented a challenge to the excavation with the hydromill. Different types of cutting wheels were utilized, and cleaning brushes were mounted on the wheels, but still the hydromill wheels tended to become like a tire due to the fat clays being very plastic and sticky.

Figure 12. Bauer cutter

Figure 13. Clam shell

Slurry Wall Reinforcement

Approximately 1,250 tons of rebar was installed at the BPT-DS & SS shafts. Cages were constructed on the ground in horizontal position and uplifted by means of 2 lifting cranes. Some single cages were approximately 50 tons; critical lift plans were utilized for every single rebar cage.

Glass Fiber Reinforced Polymer (GFRP) was installed at the tunnel penetrations utilizing ASLAN 100 GFRP.

Slab Dowels

The DS shaft had 2,620 slab dowels installed and an 8 foot shear key all around the bottom for the base slab; at the SS shaft only a 8 foot shear key was installed. The shear key and the slab dowels were installed on the rebar cages at the correct elevations, as referenced from the guide walls.

Tremie Concrete

The volume of concrete for each primary panel was approximately 1,000 CY done as a single pour; this required a lot of coordination and a lot of team work effort by all parties. Approximately 30,000 CY of tremie concrete was poured at the BPT-DS & SS shafts.

SHAFT CONSTRUCTION METHODS

Mobilization

Major equipment consisted of the following:

- Two (2) Liebherr 885 Crane and Bauer BC 40 Cutter (Figure 12)
- Two (2) Clamshell and Chisel (Figure 13)
- One (1) Liebherr HS 895 or equivalent
- One (1) Liebherr LR 853 or equivalent

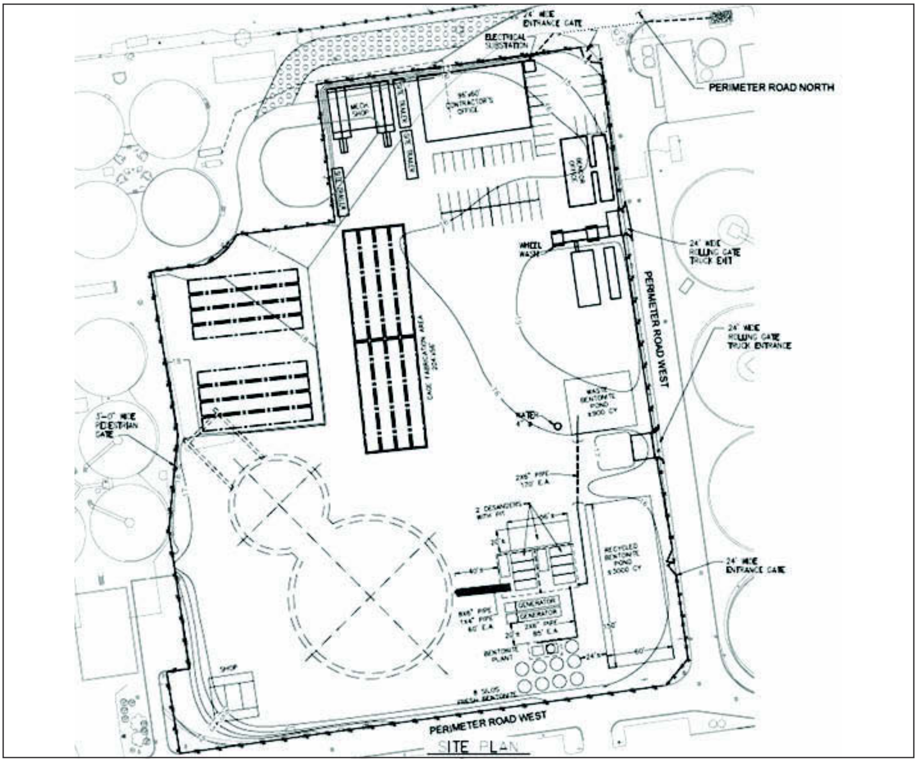

Figure 14. Site layout

- Two (2) Sotres 450 Desander Unit
- One (1) MAT Bentonite Mixing Plant

Site Utilization

The site was utilized as shown on Figure 14 and mainly consisted of the following areas:

- Office trailers in the northeast corner
- Slurry Plant in the southeast corner
- Rebar cages fabrication on the west side of the site just north of the shafts
- Repair shop in both the southwest corner
- Slurry ponds in the southeast corner
- Glory hole to store excavated materials north of the slurry pond

Construction of Guide Walls

Guide walls were constructed to be utilized for the excavation equipment placement and accuracy. The guide walls consisted of 2 parallel reinforced concrete beams (1'-2' wide, 4' deep) constructed in segments to form the shape of the shafts. The tolerance of the inside guide wall is –0 and +0.25" to ensure the 6" tolerance of the slurry wall panel.

Guide walls were constructed at Elevation +16. It was crucial to have a very high level of quality control on the guide walls as all measurements were taken from the guide walls, and due to the very tight tolerances on the slurry walls.

Excavation of Slurry Walls

Four major pieces of equipment were utilized to excavate the slurry walls: 2 clam buckets and 2 hydromills sized to the designed dimensions of the slurry walls.

Excavation was performed first by the clam bucket to remove as much clays as possible from the panels, then the hydromill started excavation with smaller size wheels (32") to minimize the amount of fines to deal with, then the second hydromill with the 60" wheels completed the excavation, it was found that this was the most efficient way to deal with the soils and to reduce the amount of suspended fines in the slurry systems used.

Different types of wheels were used during the excavation. The primary panels utilized clay wheels, and rock wheels were used during excavation of the closing panels to cut through the previously poured concrete. This concrete often exceeded a compressive strength of 10,000 psi.

Two desanding plants and two centrifuges were utilized to clean the slurry systems used.

Slurry Systems Utilized

Blended Slurry (Bentonite and Synthetic Ploymer)

The process started by using a slurry blend consisting of bentonite and a synthetic polymer (Polyblend). The polymer acts as a flocculent and the bentonite forms the bentonite cake and deals with stability of the soils.

This system worked good when utilizing the clam bucket, as the alluvium materials and clays were removed during excavation, silt and fine sand precipitates at the bottom of the panel but requires more time. The second day the panel is cleaned by the bucket where most of the materials in suspension will have settled.

The good thing about this system that it requires no desanding; this system was utilized during the installation of the cutoff wall and at the beginning during installation the primary panels.

Gypsum-Based Slurry

The gypsum based slurry fluid (high calcium) is a clay inhabitant by means of exchange of calcium ions, the free calcium in the system inhibits the anionic receptor sites on the clay particles and prevents the clay from hydrating and creating a water envelope, therefore inhibiting the undesired characteristics of the clay (sticking, swelling, high density, etc.)

The gypsum based slurry was utilized on the majority of the slurry walls panels at the BPT-DS & SS shafts. This system works fine, but requires a lot of maintenance and to constantly keep a close eye on the characteristics of the drilling fluid. It also required the addition of PHPA polymer to deal with the upper layers of the soils.

Reinforcement

Reinforcement of the slurry walls consisted of rebar tied together to form rebar cages. The cages were constructed in a horizontal position on the ground and were uplifted by means of lifting cranes. The cage was constructed in two sections, an upper and a lower and each cage weighed up to 100,000 lbs.

Figure 15. Rebar cage constructed

Figure 16. Blue Plains shaft BPT-SS 76' diameter—slurry walls exposed

Each cage was considered a critical pick as it was lifted by two cranes for tripping from a horizontal position on the ground to a vertical position (Figure 15). Once in the air, the main crane traveled from the cage building area to the slurry wall and installed the section into the excavation and locked it off to the guide walls by means of channels. The top half of the cage is then picked, again by two cranes to trip it, then again the one crane transports the cage to the panel and both cages are spliced together and Crosby clips are installed on all picking bars.

Since the middle wall's width varied from 8 feet to 18 feet, it was necessary to build a platform so that the iron workers could reach and build the cage splices.

Tremie Concrete

The design required that the slurry wall concrete to achieve a minimum compressive strength of 7,000 psi and the concrete breaks reached a maximum of 12,000 psi, therefore excavation of the closing panels was very difficult and required a lot of maintenance to equipment due to the very high strength of the concrete (Figure 16).

Panel volumes were sometimes above 1,000 CY. Some panels were poured by 5 tremie pipes due to the geometry of the panel and the large volume of the concrete pour.

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CHALLENGES OF LARGE-DIAMETER BLIND DRILLING AND LINING IN FRACTURED GROUND

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Wendy Stiller ▪ Frontier-Kemper Constructors, Inc.

Neal Wedding ▪ Frontier-Kemper Constructors, Inc.

ABSTRACT

The underground operation at Barrick's Cortez Mine, located near Elko, NV, required the construction of a 12 foot diameter smooth-lined ventilation shaft. Barrick preferred to mechanically excavate from the ground surface without putting people in the excavation and considered blind drilling by Frontier-Kemper Constructors the best option. Design and construction challenges arose throughout both the excavation and lining stages. A large portion of the rock to be drilled was highly fractured. Important risks that were addressed included water loss into the formation, zones of squeezing ground, damage to drilling tools caused by blocky ground and fallout, and potential hazards during cast-in-place concrete lining. Detailed explanations of these challenges and the methods used to overcome them will be described.

PROJECT BACKGROUND

Barrick Cortez, Inc. contracted Frontier-Kemper Constructors, Inc. (FKCI) to blind drill and concrete line a 12' finished diameter by approximately 1,500 foot deep airshaft at their operation 18 miles south of Crescent Valley, NV. The three options considered for the construction of this shaft included blind drilling, raise boring, and conventional drilling and blasting. Upon weighing the safety, cost, and timeline requirements of its underground operation, blind drilling was chosen as the best solution for sinking the shaft.

One of the primary factors that pushed the decision toward blind drilling was the reduced potential for safety incidents over conventional shaft sinking. Since the machine sits on the ground surface and no person enters the excavation at any point during the process, the exposure risk to ground falls and other dangers inherent to shaft sinking were eliminated. Unlike raise drilling, blind drilling is performed with no underground access available, making it independent of underground operations. Underground development toward the shaft location proceeded during shaft construction and did not impact the schedule. Additionally, blind drilling shafts in this size range generally costs less than conventional sinking.

DRILLING

To begin the construction process, Barrick completed the necessary site work and pre-grouted the area around the shaft. After pre-grouting, FKCI constructed a 44 foot deep concrete collar and placed the foundation for the blind bore machine (BBM).

The basic components of our BBM include a draw works and mast capable of lifting 350 metric tons, a hydraulic rotary table, powered sliding platforms to support the rotary table, high-pressure air compressor, drill pipe, and bottom hole assembly (Figure 1). Construction of the rig was performed under a partnering arrangement with Aker Wirth.

Frontier-Kemper designed and manufactured most of the surface equipment. Aker Wirth provided the rotary table, drill pipe, and down-hole tools, including the cutter head, stabilizers, and weights.

Drilling began on December 14, 2011 and concluded on June 17, 2012. The main challenges encountered during drilling were related to the highly fractured zones in the rock formations (Figure 2). These challenges included water loss into the formation, difficulty in drilling through squeezing ground, and damage to equipment from fallout.

Figure 1. Blind bore machine

Loss of Circulation

The blind drilling process uses reverse air circulation to remove cuttings during excavation. This requires that the shaft be filled with enough water to maintain the pressure differential between the annulus around the drill pipe and the inside of the drill pipe, which serves as the muck removal path. Compressed air is injected into the inside of the drill string at depth to create a lower pressure environment and generate the fluid velocity required for muck transport. The intake trough on the bottom of the cutter head connects the higher-pressure water-filled shaft excavation and the lower-pressure space inside of the drill string. This pressure differential forces water to flow up the drill string and out the discharge pipe on surface, carrying drill cuttings with it. The muck stream discharges into a settling pond or mechanical separator so that the fluid portion can be sent back to keep the shaft filled (Figure 3).

When blind drilling through a permeable formation, water loss into the formation can lead to loss of circulation and is, consequently, a major concern. Based on the evidence of severely fractured zones seen in the rock cores, water loss was expected to be an issue. Before drilling began, Barrick pre-grouted the area around the shaft in an attempt to fill voids and seal the most fractured zones. Additionally, lost circulation material (LCM) was added to the water from the secondary pond on its way into the shaft annulus. The lost circulation materials for this project were supplied by Baroid Industrial Drilling Products and administered by Jentech Drilling Supply. The drilling method, along with environmental restrictions, made the selection of appropriate and effective LCM a challenge. Environmental regulations prohibited the use of soluble organic LCM (e.g. sawdust, nut shells), leaving us with only the more expensive insoluble or inorganic LCM materials (e.g. cellophane flake, mineral fiber). Most of the loss control materials approved for use on the project are very light and prone to floating in water. When used in direct circulation drilling operations, this is acceptable, as the drilling fluid is pumped to the excavated face. This is not the case in reverse circulation drilling, and the LCM must be heavy enough to sink through the drilling fluid to find the leaks.

Neither pre-grouting nor the addition of LCM absolutely solved the loss of water problem. While drilling through the top 500 feet, the most fractured zone on the project, water was lost at rates up to 100,000 gallons/day. Overall, for the full shaft depth, water loss averaged around 30,000 gallons/day.

Figure 2. Samples from fractured zones

With nearly all drilling projects, the expense of additives that slow or stop water loss must be weighed against the cost of simply replacing the water lost to the formation. For slow leaks, adding water would be the least expensive solution. To help mitigate the risk of encountering a large open fracture and losing water faster than we could make it up, we maintained a level of LCM in the fluid as recommended by our supplier. Although fluid loss was identified as one of the biggest risks to the project early in the planning stage, the prevention measures taken allowed us to maintain the shaft water level well enough to successfully complete the project and maintain circulation.

Figure 3. Blind drilling site

Figure 4. 14' Cutter head

Difficult Ground

The fractured condition of the rock formation had the potential to cause complications beyond a loss of drilling fluid. In preparation, FKCI scheduled times to remove the drilling tools for inspection and cutter replacement at three intervals. During drilling, multiple zones of squeezing ground and fall out were encountered.

FKCI used a 14 foot diameter, semi-flat-faced cutter head with randomly spaced, or strawberry button, cutters (Figure 4). Cutter wear was very uneven, as is always the case on large diameter drilling heads. The outside cutters traveled about 6 times farther than the inside cutters with each revolution. In an effort to prolong cutter life and equalize wear, we relocated the cutters each time we tripped the tools out.

Inspection of the cutter head after the first trip at 551 feet revealed no observable serious issues. After performing routine maintenance, drilling continued from 551 feet to 954 feet without incident. At a depth of 954 feet, FKCI again tripped out, inspected, and performed routine maintenance on the cutter head.

At 1,337 feet, before the third scheduled trip, the drillers observed a larger loss of circulation than had been normal. When tripping out to determine the problem, severe squeeze zones were encountered. In order to pass the cutter head through these zones, the cutter head was rotated in an effort to ream back up. These swelling zones

Figure 5. Fallout on stabilizer wings

followed sections of fallout, with boulders weighing as much as 3 tons falling onto the bottom hole assembly (BHA) and landing on the stabilizer wings (Figure 5).

Inspection of the cutter head clearly showed the cause of the loss of circulation—a missing cutter. Presumably, rough drilling through the zones of broken rock sheared the retaining bolts and dislodged the cutter from its saddle. The combination of rough drilling in the rock and the obstacle of the freed cutter resulted in extensive structural damage to the underside of the cutter head. Eventually, the cutter lodged in the intake tube and clogged the drill string. While tripping the drill string out for investigation, the cutter fell to the bottom of the shaft.

Drilling operations shut down from April 21st to May 25th in order to perform all of required repairs on the cutter head, including replacing broken saddles, reinforcing all saddles, rebuilding the intake trough, and repairing cracks in the body of the cutter head. During the last four days of this period, drillers used a single action clam shell bucket to fish out the rogue cutter, removing a large quantity of fallout from the shaft bottom in the process (Figure 6). Upon completion of repairs, drilling continued. The soft material in the dike zones required re-reaming to pass through. Subsequently, drillers tripped back up through the zones weekly, reaming to check for tights. The total reamed depth was 1,475.5 feet.

Deviation

During large diameter blind boring, verticality is maintained by holding back some of the drill string weight and taking advantage of the plumb bob effect. Enough weight still must be maintained on the cutters to fracture the rock at an acceptable rate. Balancing these parameters requires the attention of an experienced driller and is made more difficult when cutting hard previously-fractured rock. The fractured zones in the formations

Figure 6. Clamshell used to remove broken cutter

required more weight on the bit to keep drilling steady, which led to some deviation. Although most of the fallout was a result of drilling through the two separate dike zones that crossed the path of shaft construction, the deviation may also have contributed to some fallout.

In order to measure shaft verticality during drilling, FKCI took a nondirectional survey every 40 feet for the first 1,300 feet, measuring the position of the center of the drill string. Beyond 1,300 feet, two sonar surveys were completed: one at 1,325 feet and one at total depth. Sonar surveys, performed by the COLOG division of Layne Christensen, gave us an accurate picture of the entire cross section of the shaft, not just the centerline, and were ideally suited for use in our drilling fluid. The sonar unit deployed easily reached the shaft ribs and most of the fallout areas, but was not powerful enough to reach the deepest fallout zones (Figure 7). The shaft was drilled without a pilot hole for guidance and deviated a maximum of 5.5 feet from vertical.

Before tripping out for the final time, FKCI recirculated twice the total volume of water in the shaft through the drill string to ensure the removal of all cuttings. The shaft was drilled approximately 15 feet deeper than the planned liner depth to allow room for large boulders that may have fallen to the bottom to settle below the shaft lining. Over 8,100 cubic yards of cuttings were removed from the shaft during drilling. FKCI drilled for a total of 143 days with an average penetration rate per drilling day of 10.01 ft/day.

LINING

After drilling concluded at a total depth of 1,475.5 feet, FKCI disassembled the BBM so the lining process could begin. The original contract called for the installation of a cast-in-place concrete liner. However, to avoid exposing people to the hazards of working inside an excavation, Barrick asked us to provide a steel lining installed from the surface instead.

Liner Design and Installation

The 12 foot diameter ring-stiffened steel lining was designed to resist hydrostatic pressure between the depths of 1,100 feet and the bottom of the lining at 1,460 feet. The liner was fabricated as a steel plate cylinder with channels welded to its exterior surface, forming steel tube stiffeners. It was delivered to the site in complete rings 10 feet high. Upon arrival, access holes were cut in the flanges of the stiffeners, and a low expansion grout was pumped in to fill the void. This helped to ensure that the stiffeners were not crushed by the water pressure from inside and outside the liner during installation. During installation, four full-depth vertical pipes with slots were added around the lining perimeter to serve as guides for the grout pipes during backfilling.

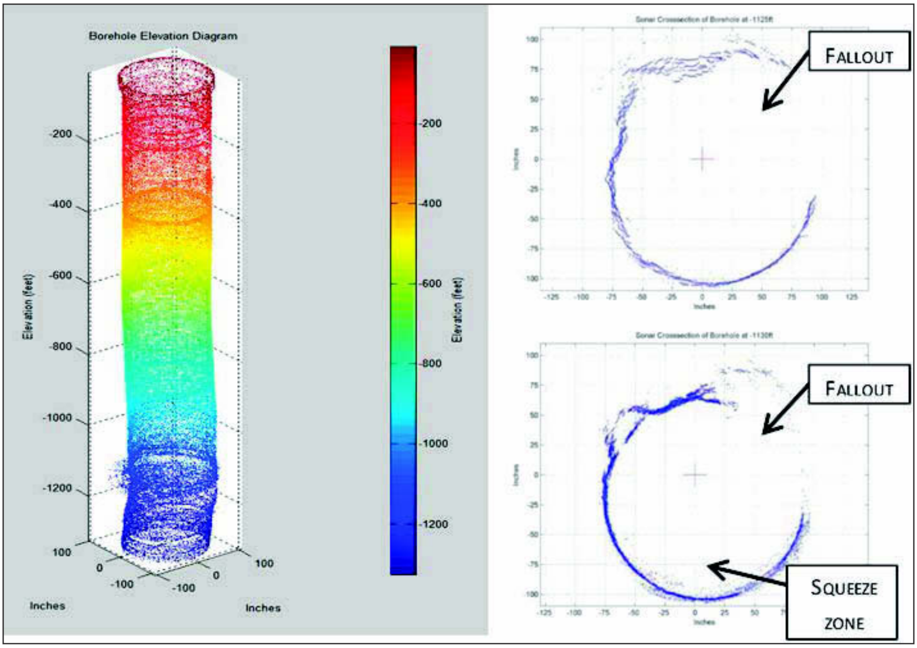

Figure 7. Sonar survey of shaft

Figure 8. Liner installation system

In order to install the lining, Frontier-Kemper designed, fabricated, and installed a hydraulic gantry system that rested on the shaft collar (Figure 8). In addition to the eight 200-ton hydraulic cylinders used to raise and lower the lining sections, the gantry included work platforms for access during fit up and welding and pneumatically-actuated chairing doors.

During installation operations, the previously-placed liner section chaired on the bottom platform with the lower elevator closed, supporting the weight of the submerged liner. The next section was then rigged into place and chaired on the top lift platform, which was supported by large hydraulic cylinders. Welders working from the top platform completed the full-penetration weld joint. All welds were ultrasonically tested for defects by a certified inspector.

After welding the new liner section in place, the upper elevator was raised, supporting the lining weight, while the chairing doors on the lower platform were released. This allowed room for personnel to weld in guide pipes for the grout lines. The entire section was then lowered and chaired on the bottom elevator. This process was repeated until the lining reached the bottom elevation.

Although the hydraulic gantry had enough design capacity to carry the weight of the entire lining, we opted to use the buoyant force available in the shaft to help. A bulkhead was bolted to the bottom liner to seal the interior from the fluid that remained in the excavation, allowing the liner to float. By adding controlled amounts of water to the inside during installation, the steel lining was sunk while minimizing the jacking force required for support. Sudden water loss into the formation was one of the biggest risks identified during planning of this operation. If the water level in the excavation fell more quickly than it could be replaced, more load would have been transferred to the hydraulic gantry. This could have resulted in the weight of the liner plus the weight of the water inside resting on the steel platforms, a recipe for catastrophic failure. The lining alone weighed 2 million pounds, while the hydraulic gantry was designed to support 2.4 million pounds. The weight of water inside the lining at full installation depth was nearly 5 times the weight of the lining. To mitigate this risk, the bulkhead was designed to detach from the lining if the force inside exceeded the buoyant force from outside by 400 tons. This would limit the shock transferred to the gantry lift while providing enough margin against accidental bulkhead failure. In the field, water was lost to the formation at a rate of about two inches in ten minutes (24 gpm). This was easily made up for by supplying water to the annulus. Localized deflections in the bulkhead also contributed to minor leaks, but at a slow and controllable rate.

Grouting

After landing the casing at a depth of 1,463 feet, FKCI back-grouted the annular space between the steel liner and the rock. The four slotted grout guide pipes welded to the outside of the liner acted as standoffs to keep the casing away from the shaft wall, ensuring the availability of adequate clearance for the grout pipes to be inserted down the entire shaft depth. Low strength, high yield grout was placed through three of the four guide pipes at a time in lifts up to 90 feet (Figure 9). The three tremie pipes were then rotated at every subsequent pour through the four guides.

As grout was injected between the casing and shaft wall, water from the annulus overflowed into the settling pond through a built-in overflow pipe in the collar. Grout was placed at a rate of about 90 cubic yards per day. Upon comparing theoretical fill volumes to actual volumes, it was clear the some grout seeped into the surrounding formation and some filled fallout zones.

After grouting, the inside of the shaft liner was dewatered. At total depth, there were 1.7 million gallons of water in the excavation. Dewatering the shaft began late in November, 2012, and was completed mid-January 2013. Pumping took longer than expected, mostly due to complications from sediment that collected in the bottom of the liner. Below a depth of 1,300 feet, the fluid was too dense to pump and was removed with a steel bailer built for the purpose (Figure 10).

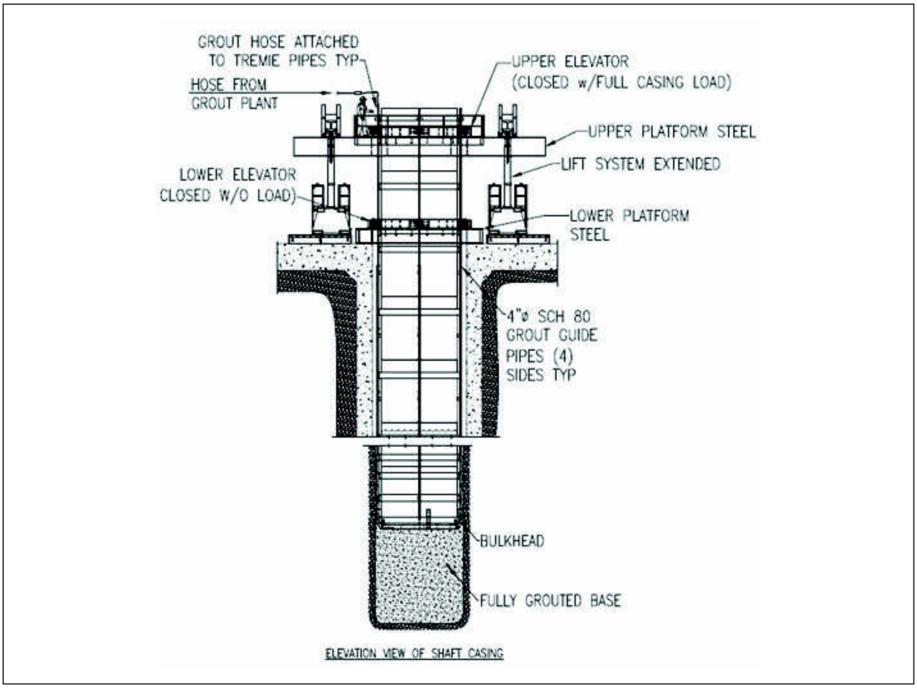

Figure 9. Grouting schematic

CONCLUSION

Despite difficult ground conditions and unexpected equipment damage, the team drilled through squeezing ground, handled large fallouts, maintained adequate circulation for blind drilling, and designed a system to line the shaft meeting all safety requirements. This was a successful pilot project both for Barrick Cortez and Frontier-Kemper, as no other shafts had been blind drilled through similar ground.

Figure 10 Steel bailer

PART 22

Tunnel Finishing and Liner Installation

Chairs

Seamus Tynan
McNally Construction, Inc.

Stuart Lipofsky
JF Shea Co., Inc.

CONSTRUCTION OF THE CONCRETE SLAB TRACK AT THE GOTTHARD BASE TUNNEL IN SWITZERLAND

Franz Pacher ▪ Alpine BeMo Tunnelling GmbH

ABSTRACT

The Gotthard Base Tunnel is the longest railway tunnel under construction in the world. Alpine together with Balfour Beatty Rail GmbH is currently installing the concrete slab track and the access tracks to the existing line. The presentation includes an overview of the progress of work and a summary of the contractual and technical design of the railway works. The presentation's focus is the installation process for the slab track, the associated development of special machines such as the concreting train, transportation shuttle, sleepers and rail mounting systems as well as the selection and testing of the concrete recipes used.

PROJECT DESCRIPTION

General

The 57-km Gotthard Base Tunnel, currently the longest railway tunnel under construction in the world, is the first base tunnel across the Alps and is being constructed at an altitude of 550 m above sea level. The tunnel is designed as a twin-tube tunnel with two emergency stop stations that divide the tunnel into three approximately equally long sections with cross-passages up to every 350 m to connect the two tunnel tubes. Emergency stop stations at Faido and Sedrun divide the tunnel into three approximately equally long sections. The so-called multifunctional stations (MFS) each contain two track crossovers and cavern systems for railway equipment. Moreover, the railway equipment (switch cabinets etc.) is housed in the track crossovers and caverns. Alpine-Bau, within the TAT consortium, is involved in the construction of contract sections Bodio and Faido (together approximately 30km of the Gotthard Base Tunnel) and is also a member of the railway systems general contractor consortium Transtec Gotthard (TTG). Alpine-Bau holds a 25% share in each of the joint ventures.

Client and Contractor

The client for the railway equipment contract is AlpTransit Gotthard AG (ATG). ATG is a 100%-owned subsidiary of the Swiss Federal Railways (SFR; in German: SBB = Schweizerische Bundesbahnen) and was founded with the purpose of building the New Railway Link through the Alps (NRLA; in German: NEAT = Neue Eisenbahn Alpen Transversale). The financing is provided by the country of Switzerland. The superior oversight and approval authority is the Federal Office of Transport (FOT; in German: BAV = Bundesamt für Verkehr), which provides the SFR or ATG, respectively, with the necessary resources from the budget. The involved group for the Railway equipment is joint venture Transtec Gotthard (Alpiq InTec AG, Alpine-Bau GmbH, Balfour Beatty Rail GmbH, Alcatel Lucent Schweiz AG and Thales Rail Signalling Solutions AG). The contract sum is 1.57 Bio CHF.

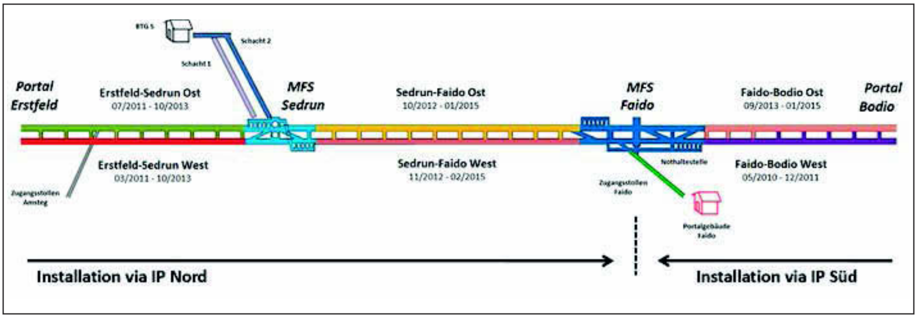

Figure 1. Installation sections railway equipment (Source: Transtec Gotthard, 2012)

New Railway Link Through the Alps (NRLA)

The Gotthard Base Tunnel, the core of the New Railway Link through the Alps, has a total length of 57km, which makes it the longest railway tunnel under construction in the world. It is located in central Switzerland and is being constructed as a base tunnel at a height of about 550m above mean sea level.

The NLRA is designed to create a continuous flat-rail connection from Basel to Milan, which will accordingly cut travel times and connect Switzerland to the European high-speed railway network. In addition, this new connection should shift as much freight as possible from roads to rails. The selected layout of the line makes a design speed of 240km/h possible. The Gotthard Base Tunnel is currently projected to open in 2016.

GOTTHARD BASE TUNNEL—RAILWAY EQUIPMENT

Construction of the concrete slab track in the Gotthard Base Tunnel is part of the overall technical design of the railway works for this tunnel. The CHF 1.57 Billion contract for turnkey construction of the railway equipment for the Gotthard Base Tunnel including the planning works for the construction permit and for works execution was signed on April 29, 2008 between AlpTransit Gotthard AG (as the contractor) and the joint venture Transtec Gotthard (Alpiq InTec AG, Alpine-Bau GmbH, Balfour Beatty Rail GmbH, Alcatel Lucent Schweiz AG and Thales Rail Signalling Solutions AG).

For installation of the railway equipment the tunnel is divided into six sections: the two South sections Bodio-Faido East and West, the two North sections Erstfeld-Sedrun East and West and the middle section Sedrun-Faido East and West. For installation of the railway equipment and completion of general construction works these sections are being handed over one by one as tunnel with secondary lining. Depending on the progress made by the construction contractors, the general construction program will start in section Bodio-Faido West. This section will also be used by SBB (Swiss Federal Railways) for early test and trial operations. Thereafter, the entire railway equipment works site will move from south to north and start work on the section Erstfeld-Sedrun East, followed by the Erstfeld-Sedrun West section and the two middle sections. The last section will be Bodio-Faido East, which will be commenced from the south. The entire railway systems will be delivered to the client and the operator (SBB) section by section, finishing on May 31, 2016. SBB trains will start scheduled operations through the Gotthard Base Tunnel in December 2016 (Figure 1).

The items of railway infrastructure can be grouped into the following main categories:

- superstructure: ballastless slab track in the tunnel for maximum stability, points and points-mechanisms

- traction current systems: catenary, switching stations at 15 kV/16.7 Hz level
- electrical systems: lighting, 50 Hz power supply, cable systems and cabinets
- telecommunication systems: mobile radio, data transmission and switching, power information system
- safety and automation systems: trackrelease signalling systems, driver‘s cab signalling and signals, control centres, points monitoring systems. The safety installations are an important component of the railway infrastructure. The control centre sets and monitors the points and gives the trains permission to proceed via trackside signals or displays in the driver‘s cab. The Gotthard Base Tunnel will be equipped with the new standardized European Train Control System (ETCS) Level 2 which will also be introduced on other European railway networks at the same time.

The works are divided into two parts: the temporary installation 40% of the contract and only necessary to do the permanent installations for the whole tunnel. The temporary installations include the logistics and transportations, the site installation, organization of safety and logistic installations. The installations in the tunnel for ventilation and cooling to reach not more than 28° Celsius working place temperature, power supply, lighting and the communication with landline and wireless.

The permanent installations started with the track work. (1) This included 114 km slab track in the tunnel, 14 km open track outside of the tunnel to connect the tunnel with the existing rail system, 40 km ballast track and 30 switches and crossings. (2) The second part is the Power Supply 50 Hz and the cable installations for the tunnel which includes 5800 km of cables and 2000 control cabinets in the cross sections and the caverns of the MFS Faido and Sedrun. Overall there operates the control systems of the 50 Hz installations. (3) The third part is the Traction Power 16.7 Hz with 114 km catenary in the tunnel and 40 km outside including the connection to the existing rail systems. There also must be done the earthing of the whole system, the switch gears for the security of the different parts and the monitoring and control system for 16.7 Hz. (4) Telecommunication and signaling completes the tunnel system to make it possible running on ETCS Level 2. It includes the Railway control systems, the tunnel radio network for communication between the train and the control center as well as to use your mobile phone and internet in the time drive through the tunnel. Most of these parts are programmed and created in labors to design the IT-systems for safety and signaling.

GOTTHARD BASE TUNNEL—CONCRETE SLAB TRACK

In keeping with the preliminary agreements Transtec Gotthard as the umbrella consortium contracted the works for the permanent infrastructure (concrete slab track, 16.7Hz & 50Hz power supply, telecommunications, radio and tunnel control system as well as the signalling and switching systems to four subcontractor joint ventures. The umbrella consortium itself is installing the temporary infrastructure in the tunnel, managing the overall coordination for planning and works execution, building and operating the installation sites North and South and is responsible for all logistics inside and outside the tunnel (Figure 2).

The joint venture Alpine-Bau GmbH-Balfour Beatty Rail GmbH (AFTTG) was awarded the CHF 349 million contract for the concrete slab track. This contract includes the construction of 114 km of concrete slab track using the LVT system (low-vibration-track system) and 40 km of ballasted track to connect both ends of the tunnel to the existing north-south railway line (Figure 3).

After the contracts were signed, the planning works were commenced and were completed in March 2012. Work at the Installation Site South started in June 2009. In June 2010 the joint venture AFTTG (ARGE Fahrbahn Transtec Gotthard) began

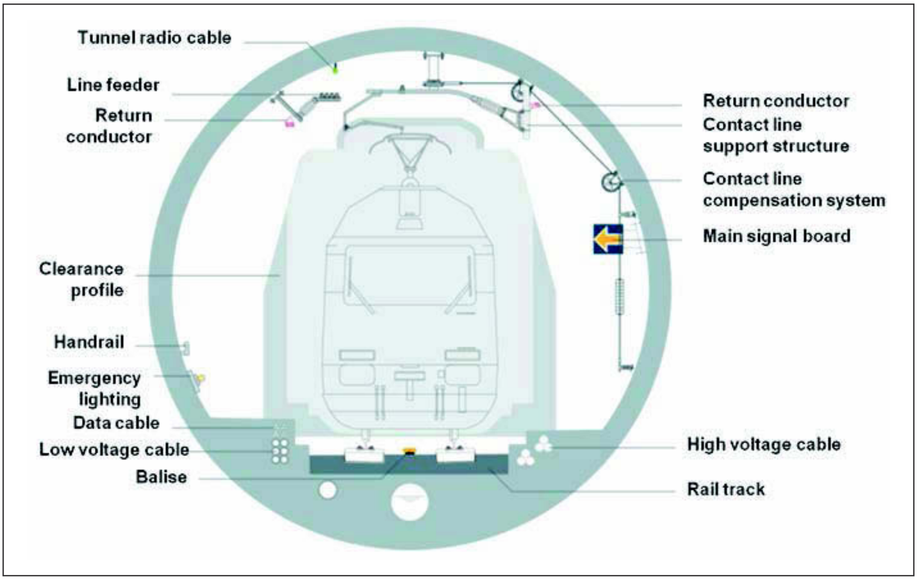

Figure 2. Tunnel cross-section (Source: Transtec Gotthard, 2012)

Figure 3. Tunnel cross-section (Source: ABH, 2010)

construction of the ballast substructure South, that was also used to connect the Installation Site Biasca and the tunnel portal at Bodio. The ballast substructures are being constructed by the Swiss track construction company Scheuchzer AG, a subcontractor of AFTTG.

Due to the tunnel's design, all track construction work can be accessed only from the north or south portals. This means the already constructed concrete slab track is used to reach the particular track construction site.

Tunnel sections behind the area to be poured have to be used for other activities and therefore special machines had to be developed and manufactured for laying and aligning of the 120-m-long rails, placing the LVT sleepers and for pouring concrete for the slab track in the tunnel's track bed.

At the heart of the concreting operation is the concreting train (Figure 4). The concreting train is a self-sufficient concrete mixing plant set up on 22 railway flat cars. The concreting train consists of 12 aggregate cars, two cement cars, water tank cars, flat cars equipped with transformer stations for electric power supply and a diesel engine for emergency power supply in the case of a power failure in the tunnel, the flat car with the actual batching plant and various additional cars for rescue containers, control plants, tanks for the concrete additives and for spare parts. The capacity of the concreting train is designed to permit concreting in the tunnel during two eight-hour shifts. Thus, the shifts are in constant operation with two shifts worked in the tunnel and the third shift in the concreting hall (Figure 5), where the train is cleaned, serviced and stocked with supplies for the next day's operations. Other special equipment includes the self-propelled concrete transport shuttle and the equipment for pouring concrete in the track bed.

Construction operations proceed as follows: The concreting train is filled and serviced and then driven into the tunnel by logistics locomotives; it stops at the last sufficiently cured portion of the slab track and is connected to the tunnel's electric power

Figure 4. Concreting train (Source: ABH, 2010)

Figure 5. Concreting hall (Source: ABH, 2010)

Figure 6. Self-propelled concrete transport shuttle (Source: ABH, 2010)

Figure 7. Special machines for attaching LVT sleepers to the rails (Source: ABH, 2010)

supply. In the track bed 2,600 m of rails have been set out and aligned together with the sleepers. The concreting operation always starts at the most distant point and works toward the concreting train. The train's batching plant mixes the first batch to produce 6 cubic meter of concrete. The concrete is transferred via a conveyor belt and a concrete pump to the self-propelled concrete transport shuttle (Figure 6). This transport shuttle drives on the tunnel's shoulders to the pouring station, where the concreting team is waiting. The transport shuttle docks onto the pouring station and transfers the concrete to the distribution hoppers. Then the shuttle returns to the concreting train to get the next batch. In this way concrete batching, transport and pouring are well coordinated and 264 m of slab track can be concreted over the two concreting shifts. This corresponds to the maximum loading capacity of the concreting train with aggregates, cement, water and additives.

Before starting the concreting operation 2,600 m of rails were laid and aligned. To attach the LVT sleepers to the rails two special machines (Figure 7) were also developed. The 120-m-long rails are laid out and welded. The rails are then moved onto the sleepers and fastened. Next, the rails and sleepers are lifted to the proper position and

Figure 8. Completed tunnel Bodio-Faido West (Source: ABH, 2012)

first roughly, then precisely, aligned on a support system with a tolerance of ⅒ mm. Now the slab track is ready for concreting. This process takes 12 work days.

The first installation section Bodio-Faido West was completed in the scheduled time using this special method (Figure 8). Installation from the north in the section Erstfeld-Sedrun East was commenced on February 14, 2012.

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CUSTOMIZED CONCRETE FORM DESIGN FOR SOUTH COBB TUNNEL PROJECT

Ran Chen ▪ J.F. Shea Construction Co., Inc.

ABSTRACT

The South Cobb Tunnel, located in Austell, Georgia, is a 5.5 mile (8.8 km) long, 27 ft (8.23 m) excavated diameter tunnel, with a 24 ft (7.32 m) finished diameter cast-in-place concrete lining. Three tunnels are connected to the bottom of the 40 ft (12.2 m) diameter South Cobb shaft: the 24 ft (7.32 m) diameter TBM tunnel, the 24 ft (7.32 m) diameter tail tunnel, and the 10 ft (3.1 m) pump station tunnel. Customized concrete forms were designed and fabricated on-site to satisfy concrete lining requirements at these junctions. Due to their large dimensions, the tunnel-shaft junction forms proved challenging to design as well as fabricate. AutoCAD 3D is a very effective tool used to characterize the geometry of the complicated shapes required by the tunnel-shaft junction.

INTRODUCTION

The South Cobb Tunnel is located in southwest Cobb County, Georgia. It is designed to meet wastewater flows for the next 100 years. The primary feature of the South Cobb Tunnel Project consists of excavating 29,000 lf (8840 m) of 27 ft (8.2 m) diameter tunnel, in which approximately 75% of tunnel was lined with an 18 inch (45.7 cm) thick concrete liner with the remaining portion of tunnel receiving only a flat invert concrete slab. There are two construction shafts (Sweetwater Shaft and South Cobb Shaft) which provided ingress and egress of construction operations. Both shafts were excavated to 45 ft (13.7 m) diameter, 300 vf (91 m) and 200 vf (61 m) respectively, and then concrete lined to a 40 ft (12.2 m) diameter finish. Several smaller tunnels were also excavated with drill and shoot methods, with tunnel lengths ranging from 68 lf (20.7 m) to 3,400 lf (1036 m), to connect the intake chambers and drop shafts to the main tunnel. The project also includes a deep pump station with a capacity of 130 mgd (492 million liters per day) at the South Cobb Water Reclamation Facility (WRF) to convey wastewater flows from the tunnel to the plant for treatment. The pump station shaft was excavated to 118 ft (36 m) diameter and to a depth of 200 vf (61 m). One connection tunnel between the South Cobb Shaft and the Pump Station Shaft was also excavated with drill and shoot methods and concrete lined to a 10 ft (3.05 m) diameter finish. More information on this project can be found in documents (Lipofsky and Forero, 2011).

CHALLENGE OF CONCRETE FORMWORK AT TUNNEL-SHAFT JUNCTION

Two tunnel-shaft junctions exist on this project requiring concrete lining. The shaft concrete is 40 ft (12.2 m) diameter and the TBM tunnel is 24 ft (7.3 m) diameter. The typical way to build this junction is to set the shaft form on the shaft invert and set the tunnel forms against the shaft form. However, there will always be gaps at both sides between the shaft form and tunnel form, as shown in Figure 1. On larger shafts, this gap becomes greater when the shaft and tunnel diameters increase and become closer

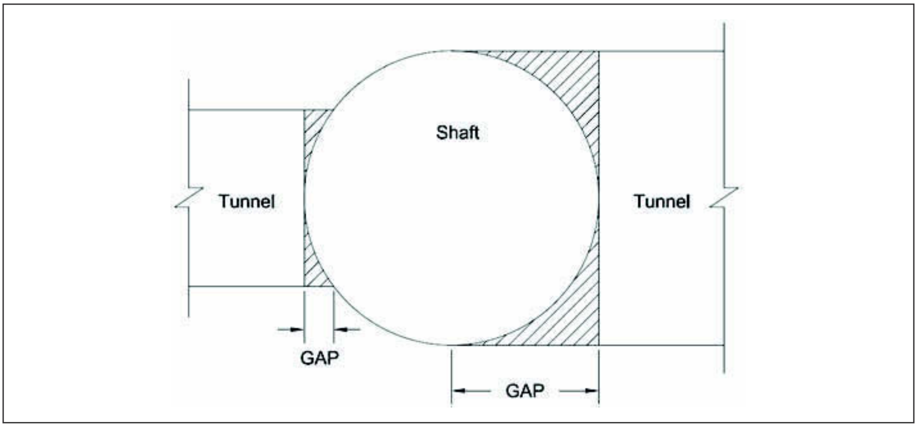

Figure 1. Sketch showing the gap between shaft form and regular tunnel form

in dimension, as is the case of the South Cobb Tunnel Project, where the maximum gap between the tunnel and shaft form is 4 ft (1.2 m) at tunnel spring line.

The most common way for a contractor to procure concrete forms for the unique tunnel-shaft junction geometry is to solicit bids form concrete form suppliers. There are many commercial concrete form suppliers servicing this industry, however, most of them produce multiple usage steel tunnel forms for circular or horse-shoe shaped concrete linings. Certainly they are able to fabricate these junction forms, but for a single use form the costs can be exorbitant due to the complicated geometry and engineering design required. There are some firms providing customized forms, but few of them service the tunnel industry, and the firms that do service the industry more or less monopolize the market. When customized forms are rented, contractors typically will pay much more than the original budget due to some uncontrollable delay. However, if contractors build these single use forms on site themselves, they will be more competitive and have more control over their project. In this paper, the method of designing and fabricating these junction forms will be presented, and the AutoCAD 3D technology used to characterize the geometry of these forms will be introduced as well.

DESIGN CONCEPTS

Since most contractors own steel tunnel forms and shaft forms, the most cost-effective and most logical way to build the junction concrete form is to fill the gap between the shaft and tunnel forms, as shown in Figure 1. In order to handle the form easily and limit its weight and buoyancy, the dimension along the tunnel alignment should be kept as short as possible. The preliminary design concept was to fasten the custom built junction form to the steel tunnel form to act as a circular cantilever. Since the form is cantilevered, no connection for strength purpose between the junction form and the shaft form is necessary. Although the shaft form will probably take some load from the junction form most likely around tunnel spring line if the “cantilever” is not very rigid, the junction form will still be considered as a circular cantilever and work with the steel tunnel form as a unit.

LOAD AND PRESSURE

All concrete formwork must “support all vertical and lateral loads that may be applied until such time as these load can be carried by the concrete structure itself” (M.K. Hurd, 2004). The majority of pressure and loadings on the forms are due to the fresh concrete

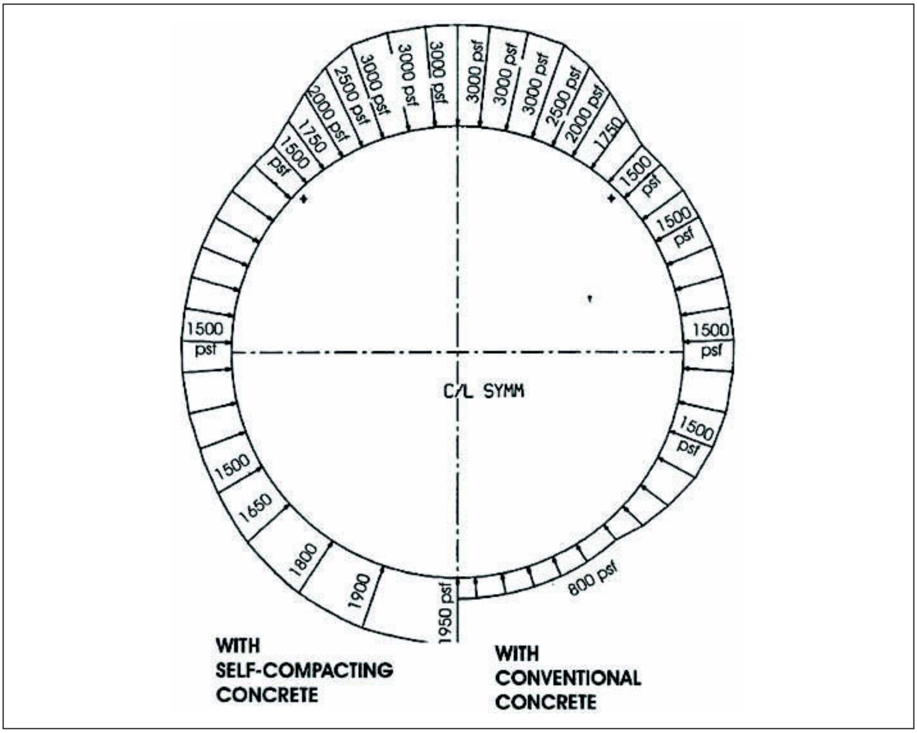

Figure 2. Form design pressures for a 12 ft (3.66 m) diameter full round telescoping tunnel form (from M.K. Hurd, 2004)

pressure, but loads attributed to self-weight, the weight of reinforcing steel and live loads imposed during the concrete placement must be considered as well. Special conditions such as unsymmetrical placement of concrete, impact from machine delivered concrete, uplift, and concentrated loads have to be considered in form design. Some safe assumptions which will hold good for conditions generally encountered must be made because all loads on the forms cannot be predetermined precisely.

In accordance with ACI Committee 347 recommendations, formwork for tunnel linings placed by concrete pump is subjected to high pressures which should be determined from firsthand information of tunnel lining operations. Pressures of 3000 psf (143.6 kPa) or higher can be induced at the crown of a tunnel form. Flotation effects must also be considered when inverts are placed together with side walls. Committee 347 recommends that the pressure assumed for design be at least 1500 psf (71.8 kPa) acting normal to the forms, plus the dead weight of concrete in the arch section. Figure 2 shows maximum pressure developed during concreting a full-round, 12 ft (3.66 m) diameter tunnel. M.K. Hurd (2004) gives out a simplified cross sectional pressure distribution. The designer assumes that the wall section extends up to the 1:30 and 10:30 clock positions, and that the arch spans between these two points. Lateral pressure acting on the wall form may be calculated from the general equation for wall forms, but no less than 1500 psf (71.8 kPa) should be used for design. Pressure acting on the arch form may be assumed to reach a maximum at the crown, and to be a minimum at the wall-arch transition points.

Considering this project case, since the shaft-tunnel junction is located at the edge of the shaft, pump valves at the tunnel arch were not used and concrete was delivered

Figure 3. Concrete delivery from outside of tunnel form instead of crown pump valve

Table 1. Base values for lateral (normal) pressure on forms (from M.K. Hurd, 2004)

Rate of placement, R, ft per hr	[150 + 43,400/T +2800 R/T] applies where placement height is more than 14 ft. Where placement height is 14 ft or less [150 + 9000 R/T] shown in the boxed areas may be applied for R less than 7 ft per hr.											
	90F		80F		70F		60F		50F		40F	
1	663	250	728	263	810	279	920	300	1074	330	1305	375
2	694	350	763	375	850	407	967	450	1130	510	1375	600
3	726	450	798	488	890	536	1013	600	1186	690	1445	825
4	757	550	833	600	930	664	1060	750	1242	870	1515	1050
5	788	650	868	713	970	793	1107	900	1298	1050	1585	1275
6	819	750	903	825	1010	921	1153	1050	1354	1230	1655	1500

from outside of the tunnel forms, much the same as for above-grade concrete work, as shown in Figure 3. According to the concrete volume estimation and the available pump capacity, the rate of placement was around 3 ft/hr, but form design conservatively assumed 4 ft/hr. The temperature of concrete during placing was chosen to be 60°F (15°C). From Table 1, the base value for normal pressure on the concrete form was 750 psf (35.9 kPa). The maximum pressure was considered to be 800 psf (38.3 kPa) conservatively in design. As proposed, the junction will be placed in two lifts separated at tunnel spring line. Figure 4 shows the profile of shaft-tunnel junction. The pressure diagram of each lift is shown in Figure 5. As shown, the maximum design pressure around the junction form is 800 psf (38.3 kPa).

After the pressure on the form is determined, typically the form will be designed step by step in the following manner:

a. Sheathing thickness and stud spacing
b. Stud size and spacing of supports
c. Wale size and spacing of supports

One of two items in each step above will be predetermined and the other calculated to correspond with it through checking bending, deflection and shearing. The design sequence outlined above can be reversed if the wale supports are chosen first. As previously proposed, the junction will be bolted on the steel tunnel form and act as circular cantilever. On this connection sheathing thickness, stud size and spacing, and

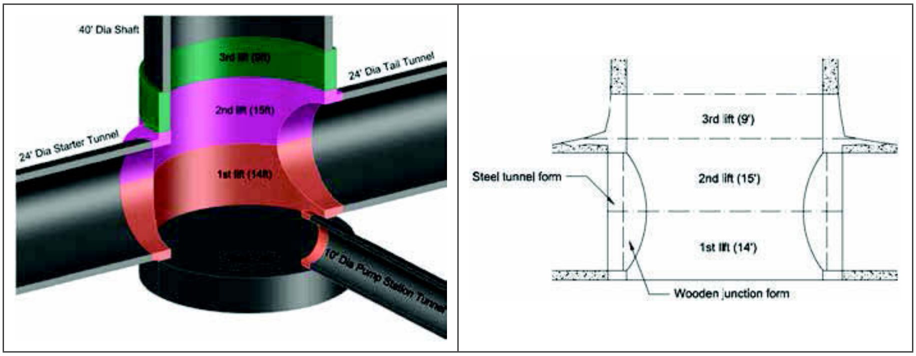

Figure 4. Tunnel-shaft junction lift drawing

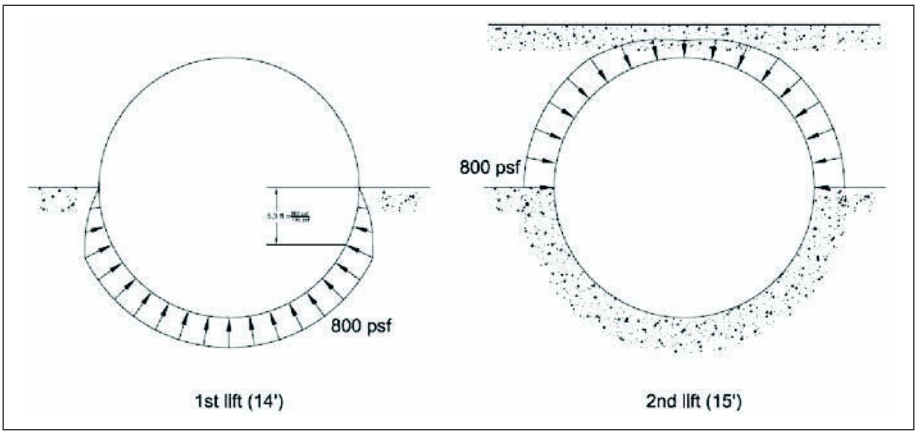

Figure 5. Pressure diagrams of each lift at tunnel-shaft junction

stud support (wood flange) need to be considered. Instead of designing following the above steps, all components are first assumed from previous form design experience and then checked for bending, deflection and shearing. The preliminary design is the following:

a. Two layers of ¾-in form plywood as sheathing. Both are ¼-in deep scored on the inner face along the grain (used in the weak direction) in order to be bended to a 12 ft (3.66 m) radius smoothly. Both layers are screwed to the studs in a staggered pattern.

b. 2×10 lumbers as studs. Lumbers are 8-in (20.3 cm) on center. Each lumber is cut to a specific length and angle to fit the geometry.

c. Staggered 2×12 lumber trimmed to radius as stud support (wood rib/flange). The end of each stud is screwed to the wood ribs using 5-in screws. The ribs will be bolted to the steel form.

A uniform concrete pressure 800 psf (38.3 kPa) was assumed in this form design as explained above. Considering a 12-in (30.5 cm) wide strip of plywood, a continuous beam over two equal spans model is used for checking bending, deflection and shear in the plywood. Considering an 8-in wide strip of plywood with one stud under it, a cantilever (fixed at one end, because the stud end is screwed to the rib and the rib is bolted to the steel form) beam model is used to check bending, deflection and shear of studs.

The screw connection between studs and rib is checked and bolt connection between wood rib and steel form is checked as well.

GEOMETRY DESIGN USING AUTOCAD 3D AND FORM FABRICATION

AutoCAD is a software application for computer-aided design (CAD) and drafting. The software supports both 2D and 3D formats. AutoCAD 2D drawings are typically used in the current design and construction industries. However, 2D drawings require that the same object is drawn multiple times for plan, elevation and section views. With the latest version of AutoCAD, it is found that it is faster and simpler to draw a solid model, in comparison to creating multiple 2D views. When revisions are made, these multiple views must be updated requiring even more work. However, only one time revisions are needed in a 3D model. Also, 3D models can be rendered and rotated to view from any angle. Additional reasons why 3D is superior to 2D are as follows:

a. A 3D model is a real, life sized entity created via the software and can give people a realistic image of a project to be designed
b. A 3D model is much easier to visually comprehend while reviewing with other members of a team
c. A 3D model avoids problems created by conflicting views of 2D drawings
d. A 3D model can give users any geometric property, such as dimensions, area, volume and center of gravity quickly and accurately
e. 3D design gives users a competitive edge in bidding a job

In this project, a 3D model of the tunnel-shaft junction form was built in the following steps:

a. Create two layers of plywood fitting in a 24 ft (7.3 m) tunnel in quarters
b. Create one 2×8 timber inside of plywood and array the object 90 degree with 8" on center to model studs
c. Create a 40 ft (12.2 m) diameter shaft, match the center of shaft with the center of tunnel and subtract the solid shaft from the plywood and studs
d. Slice the object to length proposed (here, 6-in at the top)to get a quarter of the model
e. Mirror the model into the other three quadrants to get a whole model
f. Verify dimensions with theoretical calculations

The wood ribs were also made in quarters, each of which was built by 2×12 timbers trimmed to radius and screwed together in a staggered pattern, as shown in Figure 6. In order to strip the junction form with ease, four 1.5-in thick keyways were put between joints. The complete form 3D model is shown in Figure 7. After the geometry was verified, all sizes for plywood and studs were dimensioned from the 3D model. Precise material takeoffs were obtained to minimize waste. Studs were cut with one square end and one inclined end to fit the shaft curve. A total of eight quarters were built one by one in a carpentry shop with

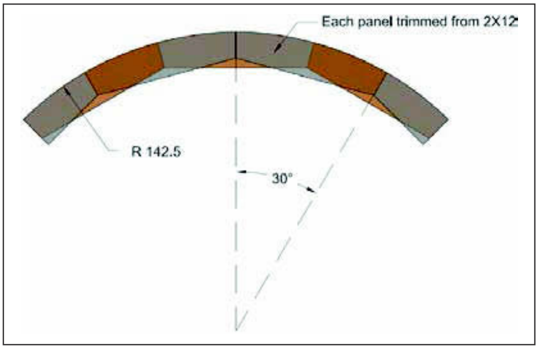

Figure 6. The layout of wood rib in a quarter section

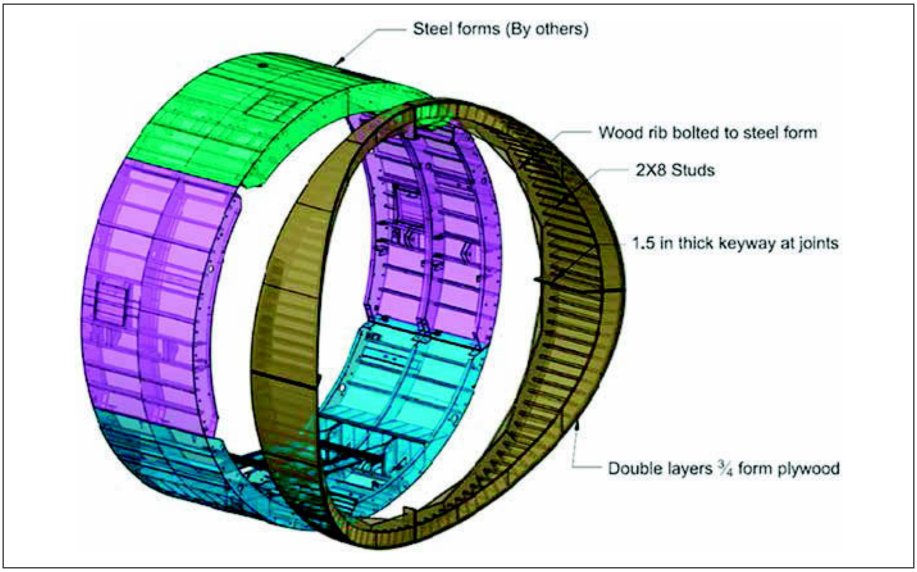

Figure 7. 3D model of shaft-tunnel junction form

Figure 8. Geometry validation using 40-ft-diameter shaft form

the available area of 15ft × 30ft (4.6 m × 9.1 m). The 40 ft (12.2 m) diameter shaft form was used to validate the geometry of the junction form when two quarters are finished (Figure 8). Since all components are pre-cut to a designated size, it only took a two man crew (a master carpenter and a general laborer) eight weeks to finish two sets of rings. The total cost including material and labor is about half the cost of a custom rented form.

ERECTION AND CONCRETE PLACEMENT

These forms were used two times, at the South Cobb shaft in May 2012 and at the Sweetwater shaft in November 2012. The form was lifted and erected in place as a whole unit (Figure 9a). In the case that there were conflicts with electric cables and

Figure 9. Junction form erection

discharge lines, the form was set in halves (Figure 9b). The lifting devices were located through the center of gravity of each half section, which was determined in the AutoCAD 3D model. In order to set the junction form flush with the steel form, four steel angles were welded on the outside skin of the steel form at the invert and extended out several inches. The wood form was set on these angles temporarily while the two forms were bolted together.

The reinforcement was tied around the tunnel-shaft junction forms and shaft wall, followed by the shaft form erection. Around 300 cubic yards of concrete were placed for each of the 14 vf and 15 vf lifts at both shafts. No obvious displacement of the junction forms occurred during placement, which was verified by survey as-builts.

CONCLUSION

Concrete form design for tunnel-shaft or tunnel-tunnel junctions is typically a challenge, especially when both the tunnel and shaft have large dimensions, not only because of the complicated engineering concerns, but also because of budget constraints and complex geometries. In this project, the concrete pressure and loads were selected based on the placement configuration. The junction forms were designed to act as cantilevers and fabricated to the geometry determined by AutoCAD 3D, which was employed to characterize this complex geometry. This allowed fabrication at a high efficiency. The junction forms satisfied the engineering requirements for concrete placement and, with the use of AutoCAD 3D, all design, fabrication and erection were completed by the contractor "in-house," in the most economical way possible.

ACKNOWLEDGMENTS

The author acknowledges Curt Reidy, a master carpenter, for offering his abundant knowledge on concrete form construction along with his willingness to accept new ways of building forms with the use of AutoCAD 3D. Without his help, this paper would not be possible.

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FREEFORM CONCRETE

Andy Thompson ▪ Hatch Mott MacDonald

Ron Federico ▪ Superior Gunite

ABSTRACT

Typically permanent concrete linings for underground structures have been installed using cast-in-place methods. Although a proven technique it does have its downsides especially where non-uniform shapes are required. Building and installing complex forms is time consuming and can lead to logistical restraints in the tunnels. The use of free or non-formed concrete has recently been adopted on several underground projects in New York for non-uniform and junction concrete operations. This paper describes the use of this technique, the challenges to using it and the paradigm shift in thinking that is needed to ensure successful implementation.

INTRODUCTION

After excavation has been completed the creation of the final, permanent lining has to be undertaken to turn the newly created space into the finished facility required by the Client. The most common method of performing this is through the use of cast in place concrete lining with or without a waterproofing layer.

Although cast in place methods can be used for virtually every combination of shapes and space there are drawbacks to its use especially when non-uniform cross sections and junctions etc. are required. As Clients strive to manage the scarce capital to be expended to manage existing and build new facilities designers and constructors are increasingly being challenged to minimize the excavation and lining quantities. This brings new challenges to the use of cast in place concrete due to the complex nature of the shapes being designed.

This paper describes the use of an alternative method of concrete placement using pneumatically applied concrete (PAC). The paper focuses on the use of this method on the East Side Access Project in New York and outlines the testing process, quality control, safety challenges and the rationale that drove the adoption of the method. In addition the limitations of the PAC method will be discussed.

ESA PROJECT OVERVIEW

The Metropolitan Transportation Authority Capital Construction (MTACC) East Side Access (ESA) Project will bring the Long Island Railroad (LIRR) directly from Long Island into a new station located 40m (120ft) beneath the existing Grand Central Terminal (GCT), in the heart of Manhattan. When completed, it is predicted that the new link will handle 160,000 passengers per day helping to reduce overcrowding in Penn Station, the West Side of Manhattan, and the Subway lines that currently transport people from Penn Station to the East Side of Manhattan. ESA is the first expansion of commuter railroad facilities in New York in the 100 years since Penn Station and GCT were constructed and it will provide a showpiece gateway to one of the great cities in world. ESA is the largest project undertaken by the MTACC and at a cost of over $8 billion, one of the largest infrastructure projects currently underway in the United States.

PROJECT EVOLUTION

In developing the alignment of the railroad a number of switches and crossovers were included to maximize the operational flexibility of the finished link as can be seen in Figure 1.

In addition to the station caverns which are a uniform shape and cross section throughout their length there are eight Wye caverns that will house switches and two crossover caverns that will house crossovers. When the design was undertaken it was assumed that six of the Wye caverns would be used for re-launching of the TBM's and as such would require a start chamber to be incorporated into the excavation to enable the gripper TBM's to be re-launched. The remaining two Wye's at GCT 1 and 2 at the south end of the caverns are three level structures and can only be excavated once the TBM excavation is complete. The crossover caverns were designed as uniform cross section as it was assumed that a standard section would be more constructible using cast in place formed concrete. During construction various changes to the methods of construction occurred. This included the use of a cast in place plug to facilitate the re-launch of the TBM's in five of the six GCT3, 4 and 5 Wyes as well as the use of road header excavation in other locations. As a result of the change in excavation methodology the necessity to enlarge these structures was investigated and in the majority of cases it was determined that the final lining could be incorporated into the TBM or road header mined excavation without the need to undertake enlargement using drill and blast.

This provided a number of benefits to the project: the quantity of drill and blast excavation was reduced which not only assisted in schedule but also reduced the impact on the overlying Grand Central Terminal and the coordination required with Metro North Railroad operations: blast fracturing and extent of over break was reduced compared to a drill and blast excavation thereby creating savings in primary support as well as backfill preparation for the waterproofing: the ability to mobilize a percentage of the primary support to be part of the final lining thereby reducing the final lining thickness: reduction in cross section area and future ventilation requirements.

However the use of continuously changing cross sections brought its own challenges and it was recognized that the ability of cast in place concrete for this work would be somewhat limited. The use of non formed or freeform concrete was investigated and following extensive testing the Pneumatically Applied Concrete method was

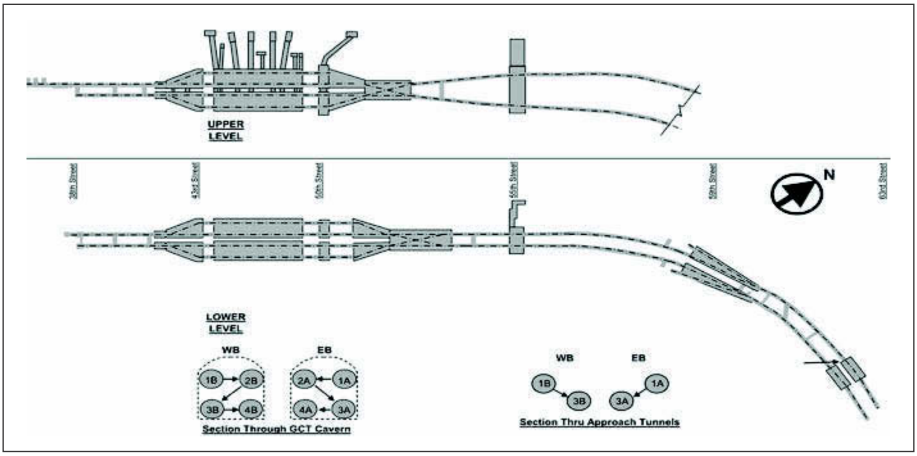

Figure 1. Manhattan tunnels and caverns

adopted for the creation of certain final linings as this method appeared to offer schedule benefits and be of comparable if not better quality.

PNEUMATICALLY APPLIED CONCRETE

Pneumatically Applied Concrete (PAC) is the application of structural concrete utilizing compressed air as the means for achieving consolidation, compaction, and a uniform distribution of concrete constituents. The end product is a one half inch minus Portland Cement Concrete (PCC) capable of achieving conventional and high strengths, while maintaining or exceeding required end properties by design. Commonly referred to as Shotcrete or Gunite, the process we will primarily address is the wet mix process, wherein materials are delivered in a wet, pre-mixed state ready to place. Materials are pumped wet to the nozzle where air is added at high pressure to achieve the required spray pattern and velocity for the concrete application.

Why is it different than shotcrete final lining? PAC makes a distinction between the application of Shotcrete for initial rock support, smoothing layers, sand walls, and other interstitial fill methods used to attain a stable, smooth, surface for water-proofing applications, and subsequent final concrete linings that are typically formed and poured. In most tunneling operations, “Shotcrete” methods are done robotically, in real time with the heading of the excavations, and in support of the safe and stable advancement. PAC addressed here is for the purposes of utilization in Structural Final Lining applications for the East Side Access Project.

PAC PROCESS

- **Method**—Shotcrete/Gunite are methods of casting concrete in place pneumatically.
- **Application**—Shotcrete may be applied by the “Dry Mix” or “Wet Mix” processes.
- **Wet Mix**—Shotcrete is typically applied by the wet mix process where ready mix concrete is pumped to the nozzle and air is added to create the velocity and spray pattern needed to encase reinforcement properly and completely on new walls, pilasters, and beams, as well as other similar structural concrete applications.
- **Dry Mix**—Gunite is typically applied by the dry mix process where sand and cement are mixed dry and conveyed by air to the nozzle and water is added to hydrate the materials in a very dry state to repair structural concrete surfaces of buildings, bridges, dams, and tunnels.
- **Design Mixes**—Concrete Mix designs for Structural Wet Mix Shotcrete processes are created for use in conventional ready mix supply of wet materials as well as onsite delivery and mixing or batching of dry materials.

PAC excels in tunnel applications where conventional forming methods are difficult logistically as well as costly to construct. Where conventional methods use large, heavy, and in most cases steel forms that have limited flexibility in final position, PAC finds its most effective uses. The benefits that the use of PAC brings include no need to engineer, fabricate, install and remove a form system in a restricted underground space which means the forms are also not going to block the tunnel during concrete placement operations. Scaffolding is needed but typically there is a need for scaffolding for the lathers and in any case scaffolding is lighter and easier to transport and install than a form system. PAC can be used with or without waterproofing, be it sheet membrane or spray applied although enhanced QC will be required for sheet membrane systems especially in overhead applications to ensure the membrane is tight against the substrate.

In the many different structures that attach to a TBM heading, PAC has proven effective for caverns, wyes, cross passages, vent shafts, air plenums, ancillaries, incline well ways, TBM crossovers and intersections “fish mouths,” that render uniform linear applications venerable to customization requirements. PAC may be “free formed” using various wire and steel rail methods to achieve literally any final shape and limit required. An extreme example is the “fish mouth” intersection of two tunnel headings or a tunnel heading and a cross passage or crossover. In all cases, these features prove difficult in their requirements for any method of concrete placement. PAC affords a monolithic placement while allowing the Designer and Contractor to achieve the needed variations in conforming to the dynamic conditions of the project which would not otherwise be achievable with a fixed forming system.

Conversely, traveling form systems in uniform linear applications such as TBM final concrete linings, where steel fiber reinforced concrete can be placed in long runs, conventional methods should prevail. PAC has its advantages and disadvantages rooted in the simple principal that while PAC placement is limited, fixed form systems are not, and where fixed form systems are limited, PAC is not. PAC complements tunnel concrete endeavors of all types and finds its limited, effective use as an invaluable method of final application.

ACCEPTANCE PROCESS ON ESA

PAC specifications that have and are continuing to develop require invariably that preconstruction as well as production testing practices be adhered to and supported by ACI, CIB, and many other associated publications on all PAC considerations. PAC requires skilled craftsmen and operators to produce the level of safety and quality required.

Preconstruction mock up panels that are of the size and shape to mimic the areas of the placement that are most heavily congested with reinforcement and/or large encasements of structural steel members. This can include embeds, pilasters, beams, and other elements that require in-situ placement. All of this must be carefully considered in all PAC applications. Mock up panels can be done offsite or onsite as conditions may dictate, but in all cases, must address as closely as possible the design and conditions dictated by the work.

Upon submission, review and planning, mock ups are constructed and tested using destructive test methods that allow close visual examination of the work. Reinforcing must be completely encased, and all Shotcrete material must exhibit sound consolidation and uniformity. This can be accomplished by sawing or coring full section samples as required to insure that the desired level of quality has need obtained by the application team consisting of Foreman, Nozzleman, Gunman, Operators, and Finishers.

All team members must work in sync to produce consistency, and quality over a large scale project. PAC teams must have many thousands of hours of qualifications to be able to achieve the level of skill needed to maintain the demands of a large construction.

In situ mock-ups may be considered when time and space are limited and logistical difficulties make elaborate mock ups unreasonable. A section of a project may be chosen as a test section; in which added sacrificial rebar can be placed randomly and cored in place for visual examination and quality verification. Care must be taken in this practice as not to compromise any of the work or adjacent work and features.

In addition to the surface mock ups on ESA an in situ underground test was performed to test the logistics of the proposed method, test in situ application methods and understand the limitations of the use of PAC and establish any lessons learned that could be incorporated into the procedures prior to commencement of production placement. As such an in-situ test was performed at a cross passage located in the non-public areas of the project in the tail tracks. This location was also chosen as it would represent one of the longest pump operations required for concrete placement.

To test the placement process additional sacrificial rebar was added into the rebar cages in the cross passage and locations were also identified where coring could be undertaken to check for compaction, shadowing and delamination without compromising the performance of the final lining. After placement and curing a number of cores were taken and the results were excellent with 100% encapsulation of the rebar and no shadowing evident on any of the rebar. The surface finish was also acceptable and complied with the construction tolerances for finished structures. One minor problem was identified with delamination of the two inch thick finishing coat which was put down to accelerated hydration of the previous layer due to the heat generated in the tunnels as a result of the test location and other concrete operations in the vicinity. To resolve this issue a minor adjustment to the process that required the air temperature to be controlled and the surface saturated prior to finishing coat application was implemented and since then no similar defects have been encountered. Before all PAC applications work plans are reviewed and a readiness review held to ensure that all lessons learned are being followed up and there is an ongoing commitment to improving quality and safety.

Once the procedures and results had been rigorously tested against the required performance criteria for the final permanent linings a separate specification was developed for the use of PAC to delineate this method from Cast in Place concrete or shotcrete as final lining. Each specification has similar performance criteria but the testing and demonstration requirements are tailored to the specific method of application. If anything the requirements for the PAC method are more onerous than the other methods as the project was determined to ensure a high quality, durable final lining with whatever method was adopted.

LIMITATIONS—NOT SUITABLE FOR ALL APPLICATIONS

It should be understood that the discussions and technical information within this presentation represent cutting edge applications of Structural Shotcrete in high performance applications of infrastructure construction and repair. In all cases special Engineering Considerations should be carefully addressed before the Pneumatically Applied Concrete method can be utilized. This includes but is not limited to special testing, mock ups, applicator qualifications and experience applicable and appropriate to the work being designed or considered for Structural Shotcrete Methods.

To date on East Side Access PAC has been used for the following applications:

- Vertical Shaft walls
- Inclined escalator shafts
- Fish mouths between CP's and bored tunnels
- Wye caverns where cross section changes continuously
- Cross passages and other restricted locations
- Final lining of the Northern Boulevard Crossing

CLIENT BENEFITS

For the Client the benefits to using the PAC method are mainly associated with schedule and quality.

As no forms are required there is no need to go through a drawn out process of design, fabrication, delivery installation and removal of forms. As such the PAC method can be used throughout the duration of the project enabling the final lining to be installed relatively quickly after excavation. This can enable follow on contracts to enter into these completed sections for access or for completion work earlier than would be the case with a CIP lining. In addition the lack of forms does not block access routes through the area to be lined although the scaffolding required to install the control wires and undertake the concrete placement may case some blockage but is of a more limited duration.

The finished space is not now limited by the need to build and install forms. Continuously changing, cross sections can be developed that minimize excavation, lining thickness and schedule as the PAC method can be used to match the lining to the space requirements and the challenge is now back with the designer to economize on these elements knowing that PAC is a tool in his armory.

With regard to Quality the finished product can be seen as the work progresses, there is no waiting until the form is struck to discover, voids, honeycombing etc. these are fixed as the work progresses with PAC thereby minimizing the need to go back and undertake remedial works in completed sections of tunnel, freeing up the completed structure earlier. In fact the PAC method has been used on ESA to rectify areas where problems were encountered with the use of formed concrete.

CONCLUSIONS

Cast in place concrete will continue to be the prime method of placement of final linings in underground structures. For repetitive lining operations such as lining a TBM tunnel over several thousand feet this is in reality the most practical method of concrete placement. The PAC method offers a viable alternative placement method for use in non uniform cross sections, shaft and other areas where the installation of a form would be problematic. It is certainly not a panacea and requires a rigorous engineered approach to the design of the structures and methods to take advantage of its flexibility and quality benefits. The challenge now lies with the designers to take advantage of this method to provide efficient and economic designs that take onto accept the limitations and benefits of the PAC placement method.

MASSIVE ANNULAR GROUT AND LONG-DISTANCE PUMPING AT THE SEYMOUR-CAPILANO PROJECT

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Alex Lowson ▪ Mott MacDonald Limited

Kevin Banek ▪ Frontier-Kemper Constructors ULC

ABSTRACT

The Seymour Capilano Twin Tunnels Project in North Vancouver, British Columbia has two water tunnels, each 7.2km × 3.8m in excavated diameter. The tunnels receive a steel lining at each end that is up to 1400 m long. The annular space between the lining and the tunnel rock is filled with a grout required to meet stringent strength, shrinkage and peak temperature requirements. The development of the special grout used for backfilling the liner, and the methods and equipment developed for placing the grout over the full 1400m lengths of liner in a continuous operation is described. The test method developed to ensure the grout met structural requirements for the liner, what those requirements are and how liner capacity is affected by grout properties is also described. The paper gives recommendations for the specification of backfill materials with a focus on fluid properties which are critical to a successful backfilling operation.

GENERAL LAYOUT AND CONTRACT REQUIREMENTS FOR ANNULAR GROUT

The Seymour Capilano Twin Tunnels Project in North Vancouver, British Columbia has two water tunnels, each 7.2km × 3.8m in excavated diameter. Access at the west end is via two separate 4.1m dia. shafts 265m deep and at the east end by a single 12m dia. shaft 179m deep. The owner is the Greater Vancouver Water District and the designer is Hatch Mott MacDonald. The project is being completed by the Seymour-Cap Partnership (SCP) which is a joint venture of Frontier-Kemper Constructors ULC, Aecon Construction Group, and J.F. Shea Construction Inc.

Each tunnel receives a steel lining at each end that is 3m in diameter and up to 1400m long. The lining varies in thickness between 34 and 25mm depending on location in the tunnel. The annular space between the tunnel and the lining varies between 470 and 240mm. Typically, void spaces between the steel liner and the tunnel wall are filled with a cellular grout prepared and placed by a specialty contractor. On this project, the contract specifications effectively precluded the use of cellular grout.

The contract specifications called for a grout with a minimum strength of 15 mPa unconfined compressive strength (UCS) in 28 days, a maximum liner temperature during grout hydration of 27°C, and a maximum drying shrinkage of 0.05%. The drying shrinkage requirement was especially difficult to meet as the shrinkage test required that the 75mm × 75mm × 250mm specimens be dried at a relative humidity of 50% and at a temperature of 23°C for 28 days before taking the shrinkage measurements.

DEVELOPING THE ANNULAR GROUT MIX

SCP Additional Requirements for the Grout

SCP required that the grout be designed for 21mPa UCS to assure that the 15mPa contract requirement would always be met. SCP required that the grout not exceed the 27°C requirement on the hottest day of the summer. In addition the grout was to be made with the crushed sand available at the jobsite. The mix had to be pumpable in a 125mm dia. pumpline for up to 1400m and had to stay alive in the line and placeable for up to 6 hours. The target pumping rate and pressure was 30m^3/hr at a pressure not to exceed 90 bar. The pressure and rate limitations were set after looking at typical concrete pumping equipment and pumpline. To prevent large uplift forces from developing, the grout had to be placed in layers. The liner pipes only had a single 100mm dia. grout port about every 11 to 12m so the grout had to flow almost level between grout ports without the assistance of vibration.

Past experience and review of the literature suggested that a successful mix would have a high Type F flyash content to slow down the grout hydration, lower the heat generation, and provide an easy pumping mix. As interruptions in pumping could be expected, need for the grout to be pumpable after an extended period of standstill in the grout line was paramount. Minimal segregation and bleeding of the grout mix was required. Although the literature provides detailed methods for calculating pumping flow rates and pressures for concrete, these methods proved of no use for sanded grout.

Grout Mix Lab Development and Field Testing

The initial mix designs were attempted using various local materials that included different cements, flyashes, and aggregates. The initial hopes were to use locally available Type F flyash, cement and crushed sand with a minimum of admixtures. The local flyash was somewhat unusual in that the specific gravity of 1.96 was rather low. Normal flyashes have a specific gravity in the 2.1 to 2.4 range. The crushed sand was readily available but crushed sand is not recommended for an easy pumping mix. In order to keep the mix simple and minimize material handling, the mix was to have only one type of sand as an aggregate.

A series of mix designs were tried using an onsite laboratory set up by SCP. Mix designs were subjected to shrinkage tests per CSA A23.2-21C and calorimetry testing. A sample of the grout was put into a calorimeter and the heat generation signature was recorded. Using simulation software, the anticipated parameters for concrete batching and placement were input and the software charted the calculated temperature development of the grout at the liner and at the grout interior. A concrete rheometer purchased for the project was used to record the viscosity and cohesion of the test batches. Test cylinders and shrinkage prisms were cast for each batch. The shrinkage test was an impediment to rapid mix development since the entire testing cycle took a minimum of 35 days from casting the prisms to getting the shrinkage results.

The first pumping test was organized after SCP developed a mix that seemed to meet the requirements. The test circuit was about 700m long, half the length of the maximum pumping distance anticipated in the tunnel. The circuit was in the form of a loop so that once the pumpline was filled up the pumping could continue for an extended period of time. The test was intended to determine what actual pumping pressures and flow rates could be achieved and verify the pumping rate and pressure calculations. The setup also allowed for the stopping and resumption of grout pumping after a set period of time (Figure 1).

The test results were disappointing. The grout mix used a lot of time dependent admixtures to control flowability, slump, and set times. The mix proved to be “touchy” in that just the right amount of water had to be placed in the mix. Not enough water and the grout mix was more difficult to pump. Adding just a little too much water made the

Figure 1. 700m long pumping test circuit using 5 inch dia. pumpline

mix segregate and bleed. The grout also changed rheology significantly, being more viscous when it came out the other end of the pumpline. The lightweight flyash was thought to be the culprit as the grout seemed to have a higher water demand and produce a lower density paste than other Type F flyashes. The water/cement ratio had to be kept very low, in the 0.36 range, to barely meet the shrinkage requirement. Finally, priming and cleaning out the line proved to be as tricky as pumping the grout.

A second, third, and fourth round of mix development was done. By this time SCP knew the maximum cement content that could be allowed without exceeding the temperature specifications and the cement and flyash required to meet 28 day UCS requirements. The problem was getting the pumping pressures down, being able to pump reliably after interruptions, eliminating the high sensitivity to W:(C+F) ratio, and meeting the shrinkage criteria. The second round of mix development including testing various ratios of crushed to natural sand, slag cement, and additional flyashes and cements. Bentonite was tested for provide line lubrication and bleed reduction. Additional admixtures were tested including anti-shrinkage admixtures.

Each round of grout mix design was followed by a full scale pumping test. Some improvement was shown but the results were still disappointing. The pumping pressures were still too high. Cleaning out the line in a timely manner proved difficult. Major promise was shown by a flyash from Washington state. This Type F flyash had a specific gravity of 2.6 (heavy for flyash) and resulted in a more stable mix, less sensitive to variations in W:(C+F) ratio.

SCP felt that the grout mix designs were getting close but were still not good enough for reliable production pumping and placement. Another round of grout mix development ensued. The first key breakthrough was the substitution of natural pea gravel for some of the crushed sand. The pea gravel reduced the amount of paste required to coat the sand particles and freed up the paste to make the grout easier to pump. The amount of gravel used in the mix was kept low enough to keep the mix flowable. The pea gravel particles did not touch each other but floated in the matrix. The second breakthrough was the development of the paste that would suspend the sand and pea gravel, in other words the aggregate had to float in the paste. Two separate grout mixes were successfully designed to meet all HMM and SCP criteria. The crushed sand was retained and the natural sand was deleted in favor of the natural pea gravel. The grout with the local light flyash was increased in cohesion by adding diutan gum to the mix. The grout with the heavy flyash was able to float the aggregate by virtue of its high specific gravity at a higher W:(C+F) ratio. In the end the heavy flyash mix was preferred as it required less admixtures, showed less sensitivity to W:(C+F) ratio, and was considered to be a mix more amenable to mass production.

Large-Scale Shrinkage Test

To determine the actual shrinkage, SCP set up a large scale test in the tunnel. A steel form was designed to bolt up against the tunnel wall and allow grout to be placed from the top. The form was 750mm long, 695mm high and 300mm wide (tunnel radial direction). The form size was picked so that the grout would have enough mass to heat up internally in the same way as it would against the pipe. Four vibrating wire strain gauges were installed in the form. Two of the gauges were installed radial to the tunnel, one was installed parallel to the longitudinal tunnel axis, and one was installed chord-wise to the tunnel cross section (90° to the longitudinal tunnel axis). The data logger was set to record data from all four channels every 30 minutes. The light flyash mix was used as this mix had the higher shrinkage potential in laboratory testing. A discussion of the results is the section "Design of the Grout and Steel Liner—Application to Future Work."

Lessons Learned and the Final Mix Design

A few lessons were learned along the way. The concrete rheometer proved to be useless for testing grout because the instrument was not sensitive enough for the low viscosity required. The spread test, using an inverted slump cone, proved to be a useful, quick, and consistent test although it did not measure some important rheological properties. The spread test was not useful for comparing one grout to another in terms of predicted pumping pressure. The test later proved most useful during production batching to determine the correct consistency before sending the grout to the pump. Making 4 × 8 test cylinders and slicing the hardened cylinders lengthwise for examination gave the best indication of bleeding and segregation.

Bentonite eliminated bleed and improved pumping but didn't help with shrinkage and strength. Anti-shrinkage admixtures made bleeding worse. Slag cement made grout that spread nicely in testing and looked great but proved difficult to pump because the viscosity was too high. Air content had to be controlled using a de-air entrainer as the fluid grout trapped too much air during mixing due to the high paste content.

The best way to proportion the pea gravel was to start with a 50/50 mix of sand and gravel. Then increase the gravel as much as possible without having the individual gravel particles contact or interact with each other. Any more gravel created a mix that would easily plug in the line.

The paste fraction of the mix was found to work best around 53% by volume of the total ingredients. More paste would have lowered the pumping pressure but would also have increase shrinkage. Paste contains the most expensive ingredients so the paste proportion needed to be kept as low as possible. Increasing flyash lowered viscosity. Increasing cement content increased the paste density and reduced segregation.

Lessons were also learned about priming and cleaning the pumpline. The long pumpline had to be clean as a whistle before introducing the prime grout. Any rust, aggregate, or paste built up in the long pumpline was sure to accumulate in advance of the grout and create a plug. The line needed to be kept clean by running a several wire brush and wiping pigs through the circuit. This was true even if the pumpline was new with only a light coating of rust.

Water from a high pressure pump was used rather than compressed air to push the grout out of the line during cleanout. Keeping the water separated from the grout proved to be essential if no plugs were to occur. Once a plug did occur, the extra time required to clean out the line could and did result in a completely plugged pumpline on one occasion.

Table 1. Grout mix proportions

Paste 53% by volume	Aggregates 47% by volume
Cement—Type GU Flyash—Type F Water Hydration stabilizer Superplasticizer Superplasticizer extender Entrapped air remover Viscosity modifier Entrapped Air 3%	Coarse aggregate—pea gravel Fine aggregate—crushed sand Grout Density 2245 kg/m^3

The final grout pumping test was done with a 1400m long 125mm dia. pumpline. The heavy flyash mix was successfully pumped through the line in a loop and a pumping stoppage of 1¾ hours did not present a problem. Line pumping pressures were under 90 bar at 30m^3/hr. By the time SCP had a satisfactory grout designed, over 70 mixes had been tested requiring many months of effort and significant cost. The final mix design using the heavy flyash is presented in Table 1. In the end, the best approach was to use the correct basic ingredients, maximize the amount of water, and minimize the use of admixtures.

PLACING THE ANNULAR GROUT

Diversion of Water from Upstream Sources and Within the Steel Lining Area

During the placement of the liner pipe and the subsequent annular grouting, there was still a need to pass through up to 8L/sec of water that entered the tunnel upstream of the liner pipe. Local water in-flows up to 3L/sec within the length of the liner pipe also needed to be drained while the grout was being placed. These areas were later grouted off.

Drainage Channel and Panning Within the Steel Liner Area

Prior to liner pipe placement, a drainage channel was created by placing metal decking on top of the steel ties in the space between the track rails as shown in Figure 2. The decking was covered with a 90mm thick layer of 35MPa concrete up to the bottom of the rail head. Testing and observation showed that the track drainage channel would be able to handle the water flows without pressurization. All water producing areas within the length of the liner pipe were panned or intercepted and the flows directed to the drainage channel using plain dimpled PVC drainboard. The panning was also necessary to keep dripping water off the steel pipe joints that were to be welded. A complete ring of panning with a geotextile face 1200mm wide was placed every 100m along the length of the liner pipe in the flat grade sections to give water a path the flow to the drainage channel. A panning ring was installed just uphill of every bulkhead on the sloped sections. Panning rings were placed with the geotextile facing away from the tunnel wall. The panning ring was intended to prevent the accumulation of water that was not captured by the plain drainboard. The composite panning ring allowed water to pass down to the track drainage channel while excluding the grout.

Installation of Flow Through Drain Lines

In order to bypass water flows generated upstream of the liner pipe, two 3 inch diameter PVC drain pipes were placed before liner pipe was set and welded in place. Each pipe could handle about 4L/sec. The pipes were concreted in the invert at the same

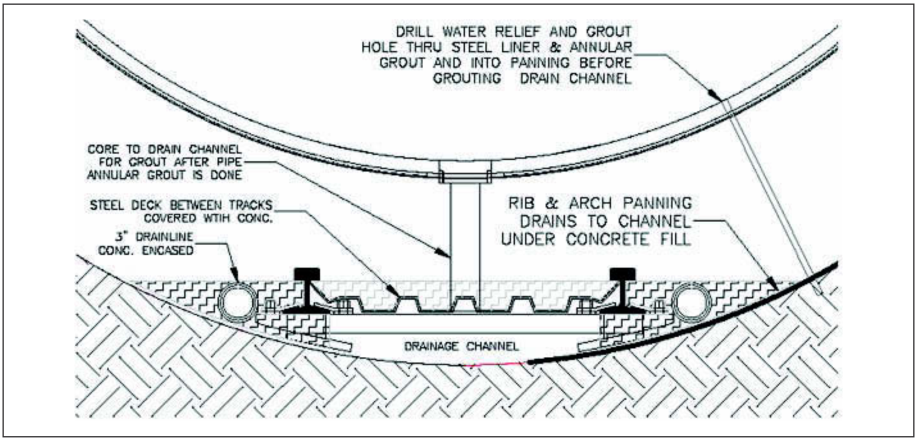

Figure 2. Drainage system for upstream flows and panning in steel liner area

time that the track channel was created. The pipes were covered with 35MPa concrete up to the bottom of the rail head. All panning within the steel liner area that drained to the tunnel invert was placed prior to placing the concrete fill. The drain lines conducted all upstream drainage water past the liner pipe. The central drainage channel was reserved for panning water to eliminate the introduction of silt from upstream. The drain lines continue to by-pass water until all work is completed inside the liner pipes and will be grouted off at the conclusion of the work.

Placement Scheme

The basic scheme was to place the grout in seven horizontal lifts around the pipe. Lift 1 was used as a sealing layer to make sure there were no leaks to the drainage channel below. Lift 2 was a bedding layer under the pipe was designed to just float the pipe. Subsequent Lifts 3 through 6 were intended to add only lateral pressure to the pipe. Lift 7 was intended to fill the top space including any broken out areas. The scheme required a finite element analysis of the pipe loading and deflection to make sure that subsequent grout layers would not displace the pipe laterally so much that the grout could flow under the pipe and refloat the pipe with greater force. The high density of the grout, about 2.5 denser than a cellular grout required that the placement scheme be closely followed. The multiple lift scheme helped keep the liner pipe heating from grout hydration below 27°C. Up until the placement of Lift 7, the placement of multiple lifts provided an annular space that could be used to dump grout anywhere along the liner pipe in the event of a pump failure or pumpline blockage thus preventing the necessity to dump grout inside the liner pipe.

A grout placement scheme was designed that allowed the placement of multiple layers of grout anywhere along the length of the pipe. The scheme also had to provide a backup to the concrete pump to clear out the slick line in the event of pump failure. A 125mm dia. grout pumpline was placed inside the entire length of the pipe. The pumpline had turn-outs for tapping into the line spaced every 76m. Grout was distributed from a 75mm dia. rubber hose through the liner pipe at various injection points. Every liner pipe section had a single 100mm pumping port at the noon position and on average they were spaced 11m apart. Grout was injected through these ports into the annular space. The 7th and final layer of grout was held back until the entire length of liner pipe had been grouted to the top of the pipe. Then the last layer of grout was

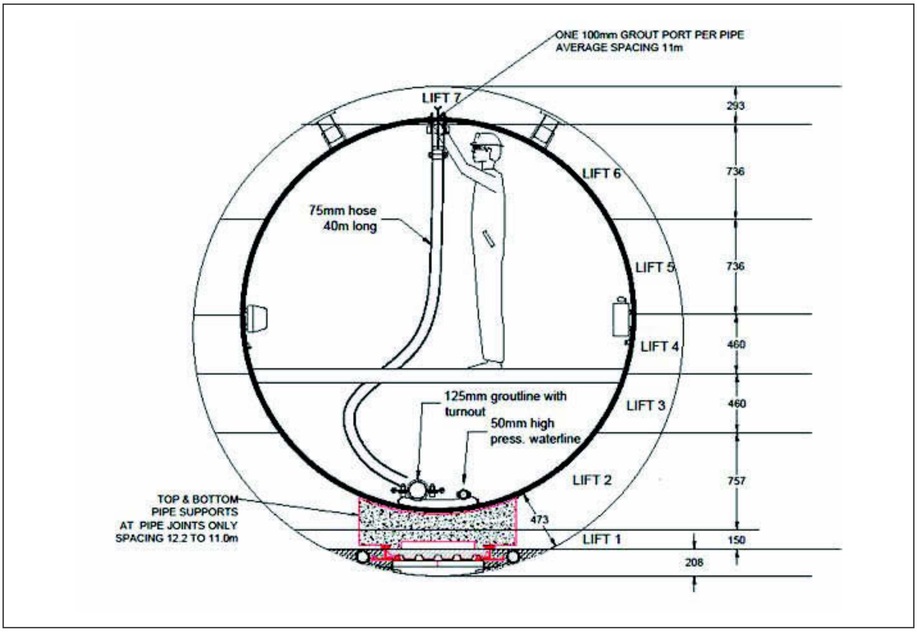

Figure 3. Grout placement sequence and lift heights

injected through the lining and pushed through the annular space to get the best possible filling of the arch (Figure 3).

Prior to grouting up the drainage channel, water relief holes were drilled through the steel lining, through the grout, and into the panning as shown in Figure 2. These holes later served as panning grout holes. Holes were cored through the liner invert grout ports into the drain channel to relieve water pressure as the grout was forced up the drainage channel. The drainage channel below the pipe was grouted up last starting at the lowest position in the tunnel. Then the top arch was contact grouted followed by the panning.

Pressure Monitoring

Pressures in the pumpline were monitored by electronic pressure transducers that reported to a central display at the grout pump operator's station. The pressure sensors were located approximately every 76m along the 125mm dia. grout pumpline. Up to 21 sensors could be monitored. The information from the sensors was displayed on a touch screen that had a vertical bar chart style output that displayed the current pressure. The display also had a strip-chart style output that showed the pressures recorded for the last 5 minutes. The pressure monitoring system allowed the pump operator to know what was happening along the line. The system also aided in tracking down any plugs occurred in the pumpline and was also able to detect grout leaks at pipe joints. The system proved useful in tracking the passage of the cleaning pigs at the end of the day and was remotely viewable at the site office.

Equipment Layout

Concrete for annular grout was produced by the batch plant at the top of the shaft. The grout pump was set up at the bottom of the shaft and pumped to whichever tunnel was receiving grout. The pump was supplied with grout via a 125mm dia. 265m high drop line in the shaft. The dropline terminated in an energy absorbing boot structure. The grout dropped from the boot into a 3m^3 surge hopper and then into the concrete pump. A hydraulically operated diversion valve after the pump allowed rapid switchover to a piece of pumpline preloaded with cleaning pigs. At the conclusion of pumping the valve switched over to the pipe preloaded with the pigs. Cleanout water was supplied by a water tank and high pressure water pump at the surface supplying water to the shaft bottom that pushed the pigs. The high pressure water line was also able to clean out the line in the event of a breakdown of the diesel powered grout pump. A high pressure 50mm water line ran the length of the pumpline to provide water pressure anywhere along the line to push grout out of the pumpline.

Progress Rates and Daily Routine

As of January 2013, SCP has nearly completed the placement of the grout around the first and longest (1400m) of four steel tunnel liners. A typical day begins with the batching of 2 to 3m^3 of prime grout which has the same flyash:cement ratio as the regular grout but just enough sand to ensure good mixing in the drum mixer. A cleaning pig is pushed by the prime grout to prevent the mixing of water in the line with the grout. Then annular grout is introduced to push the prime grout. Production and placement of the annular grout proceeds for the rest of the day shift and part of the swing shift. Production varies depending on which layer is being placed. Pumping rates up to 25m^3/hr have been recorded. Grout placed for the second and thickest layer is regulated by the production capacity of the batch plant. Grout placed for subsequent layers is restricted by the amount of time required to move around and connect to every grout port. Three hours before the end of swing shift, grout production ceases and a series cleaning pigs is pushed into the pumpline at the shaft. Grout placement continues until all the grout is pushed out of the line. This is not a small matter as there is 16m^3 in a 1400m line.

As of January 2013, the grout performance has met all contract specification and SCP requirements. Grout temperatures have been monitored by a combination of temperature data loggers and thermal camera imaging and, during the winter, have not exceeded 20°C. The primary problem has been the wear on the dropline in the 265m deep shaft followed by a need to minimize grout spillage inside the steel liner pipe.

DESIGN OF THE GROUT AND STEEL LINER—APPLICATION TO FUTURE WORK

Test Results

The results of the field trials on the backfill grout are shown in Figure 4. The test results indicated the maximum strain resulting from shrinkage due to a combination of temperature changes (cooling down from peak temperature during curing) and loss of volume due to autogenous and drying shrinkage was about 200 microstrains (ms). It was noted that measured shrinkage parallel to the rock surface i.e., parallel to the axis or circumferential shrinkage was much less than the radial shrinkage. This was assumed to be due to restraint from the rockmass and agrees with the approach used below in calculating the resulting gap outside the liner.

A maximum temperature of 21°C was reached around 3 days after grout placement, after which the temperature fell to 13°C. Based on a measured coefficient of expansion of 10 microstrains per degree the 200ms measured shrinkage is made up of 50ms due to cooling and 150ms for autogenous and drying shrinkage. Alternatively the temperature plot the strain that occurs after the sample reaches ambient temperature is about 100ms. A value of 150ms has been used below in calculating the effects on liner capacity.

The specified criteria for the mix were a maximum drying shrinkage of 500ms (0.05%) and a maximum temperature during curing of 27°C. The mix appeared to easily satisfy these criteria if it was accepted that the shrinkage that was relevant was that which would occur in practice.

Compliance with Design Intent

The gap that forms between the steel liner and the backfill material is important in design because it reduces the buckling capacity of the liner under external water pressure. The assumption made in design is that the backfill material remains bonded to the surrounding rockmass and so when it shrinks it gets thinner, increasing the gap. Based on an assumed minimum liner temperature during some future inspection of 10°C the resulting gap is then made up of a combination of thermal contraction of the liner, and thermal contraction and any shrinkage of the backfill material. The idea that drying shrinkage should reduce the thickness of backfill material that is permanently submerged in groundwater is questionable but assuming the material shrinkage somehow equals the specified limit the design gap is then calculated as:

- Thermal contraction of 1500mm radius. steel liner cooled by 17°C: 0.33mm
- Thermal contraction of 300mm backfill cooled by 17°C: 0.05mm
- 500ms shrinkage of backfill: 0.15mm
- Total Shrinkage: 0.53mm
- Shrinkage as% of radius: 0.035%

In practice the maximum temperature of the liner during backfilling was 21°C so the actual gap is calculated as:

- Thermal contraction of 1500mm steel liner cooled by 11°C: 0.21mm
- Thermal contraction of 300mm backfill cooled by 11°C: 0.03mm
- 150ms shrinkage of backfill: 0.05mm
- Total Shrinkage: 0.29mm
- Shrinkage as% of radius: 0.019%

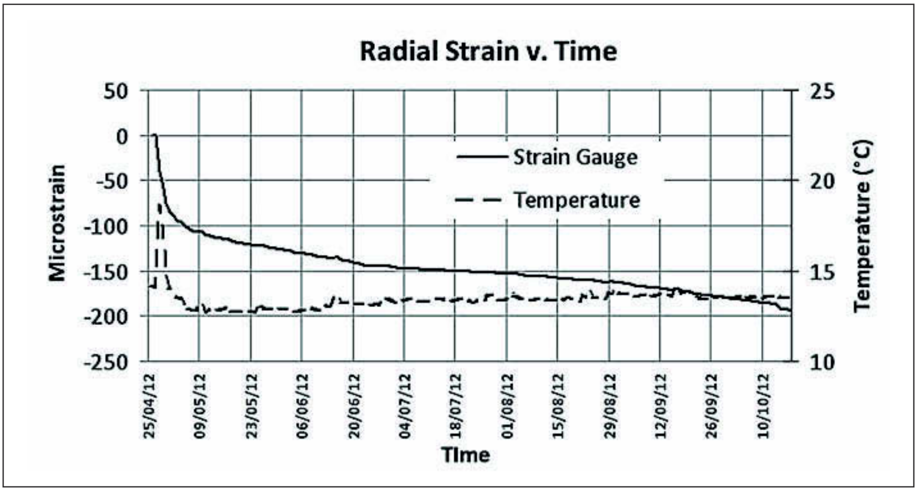

Figure 4. Shrinkage test results

On the basis of this calculation the expected shrinkage value is less than the design value so the original design calculation is valid.

Structural Design of Steel Liners

The buckling capacity of steel tunnel liners under external pressure is normally calculated using the equations given either by Jacobsen (1974) or by Amstutz (1970). The model of behavior used by both methods is that the liner is a loose fit inside the backfill. This looseness of fit arises partly from the gap resulting from material shrinkage and thermal contraction quantified above, and partly from elastic compression of the liner under the external pressure. Both calculation models then assume the liner moves to be in contact with the backfill on one side, giving space for the liner on the other side to distort and buckle inwards. Neither calculation is exact because the mathematics used to derive the buckled shape, first developed by Amstutz, makes the assumption that the deflection is small enough that curvature can be based on the second derivative of radius with respect to distance along the line of the undistorted liner rather than on rate of change of direction with respect to distance along the deformed liner. The assumed buckling condition is when extreme fibre stress reaches the yield value. For a very thick liner or one with external stiffeners this calculation of curvature is reasonably accurate but for unstiffened liners of normal proportions it is not, as deflections will be a significant fraction of liner radius when the liner starts to yield. However numerical studies by Frey and Rebora (2002) found that, while both the deflected shape and the pressure at first yield were quite different from those used in the Jacobsen or Amstutz calculations, the Jacobsen method gave results that were very close to the plastic collapse pressures predicted by their numerical models.

Both methods for calculating buckling capacity are in common use worldwide although the Jacobsen method is preferred by most designers as it is considered to be more accurate and if nothing else it gives lower buckling capacities. The Amstutz method seems to be preferred by manufacturers, perhaps because it is easier to solve the equations and possibly because it results in a thinner lining. While some serious failures of steel liners designed using a version of this method were experienced in the 1960s, these may have used relatively low factors of safety. The results of both sets of equations can be represented in semi-dimensionless form for a given yield strength of steel liner. The equations give different capacities for a given liner, but for either set the collapse pressure is a function of the ratio between liner radius and liner thickness—designs made using either equation are scaleable. Figure 5 shows the buckling capacity according to both methods for a yield stress of 386MPa (as used on the Seymour Capilano Project) for a range of radius/thickness ratios.

From the above the following points are notable:

- Radius : thickness ratios for the liners used at Seymour Capilano vary between 45 and 60 and so the reduction in capacity with increasing gap is about 7m per 0.01% gap which is a reduction of about 3½%. So capacity is sensitive to gap but the reduction is not very great, and with the normal safety factor of 1.5 used in design it would require a big change to jeopardize liner stability. So if, say, maximum temperature did exceed the specified limits the reduction in structural capacity would probably not be critical.
- Based on the tests done on site the actual gap will be 0.016% less than the value implied by the Specification, so the actual buckling capacity will be about 11m higher than the maximum allowable gap would provide.
- The Amstutz method gives capacities that are between 6% and 12% higher than the Jacobsen method.

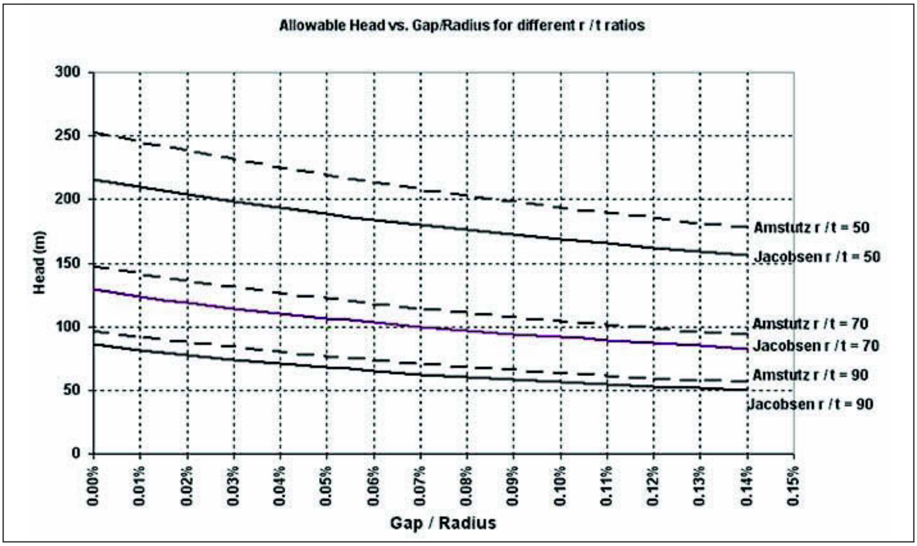

Figure 5. Buckling capacities for 386MPa yield stress for a range of radius/thickness ratios

It should be noted that the difference in calculated capacity between the two methods varies with yield stress. At very high yield stresses of 750MPa or more associated with high strength alloy steels the two methods give very similar results. However at a yield stress of 235MPa associated with lower grade plate the Amstutz method gives capacities up to 28% higher than the Jacobsen method.

Recommendations for Future Specifications

The specification given by the designer for the Seymour Capilano Project was developed for normal concretes and follows standard industry practice: The temperature limit was based on an experience of what was achievable in the field as was the shrinkage limit. The limit on shrinkage may be to some extent unnecessary since the backfill will probably never dry out and is anyway not of great thickness. However for a drill&blast situation with possibly a lot of overbreak and potentially long period between concreting operations a limit on drying shrinkage is arguably necessary because of the thickness of concrete that can be required in some locations. Moreover placing a limit on shrinkage in a normal concrete is a good way to avoid problems with segregation and workability generally, as a low-shrink mix will be a cohesive one with minimal oversanding.

However for the sanded grout the contractor needed to use, the shrinkage limit had some undesirable effects. To comply with it with a sanded grout meant developing a mix with a very low water/cement ratio. This reduced shrinkage but resulted in a mix prone to setting up in pump lines potentially causing disruption to the placing operation. Following the insitu tests it was accepted that drying shrinkage was not a problem in practice. Freed of this restriction, and focusing more on flow characteristics, the contractor was able to develop mixes that performed far better in practice, which could be pumped long distances without pipe blockage and placed without difficulty.

As demonstrated earlier in this paper the temperatures reached in practice combined with actual shrinkage levels meet structural requirements and there is no reason to think there is any problem with the liner as constructed. For future projects the following recommendations are made:

- High backfill strength is not necessary to support a steel liner. On many projects Low Density Cellular Concrete with strengths around 400psi (2.7MPa) is often used. On this project the backfill was required to provide permanent support to the ground which was easily achieved by the specified strength of 15MPa. Specifying a higher strength than necessary can cause problems with pumpability.
- Workability and ease of placement are of paramount importance and should be the main focus of the development program. Blocked pump lines and flash-set problems are likely to lead to workmanship and morale problems, the possible result being voids in the backfill or floated liners, and a high accident rate.
- Segregation or bleed of the mix must be avoided: Segregation causes pipes to block when pumping is stopped for any period. The formation of bleed water or slush on top of lifts of backfill material is a recipe for large uplift forces potentially leading to flotation or a damaged liner. A stable mix is essential.
- Drying shrinkage is not the most important criterion for a material that will be permanently submerged, and it should not drive mix development.

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BRIGHTWATER CONVEYANCE SYSTEM, CENTRAL CONTRACT: OVERCOMING THE CHALLENGES OF INSTALLING PIPE AND FINAL LININGS IN LONG BORE TUNNELS

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ABSTRACT

The Brightwater Central Contract is comprised of two tunnel sections, BT-2 and BT-3. The BT-2 tunnel is 3,536m (11,600 ft) long with the final lining consisting of three separate pipelines that run the tunnel's entire length. The BT-3 tunnel is 6,127m (20,100 ft) with the final lining being a 3.22m (10.58 ft) steel pipe running the first 1,400m (4600 ft) from the North Kenmore Portal. The piping in both tunnels is backfilled with low density cellular concrete (LDCC). In BT-2 the backfill concrete is capped with a structural slab that slopes towards the pump station shaft. In BT-3 the backfill concrete fills there entire annulus between the steel pipe liner and the tunnel segmented liner. Challenges common to both tunnels were placing straight pipe sections in curves, maintaining specified clearances between the tunnel liner and pipe, and conveying the LDCC long distances from the surface batch plant. Each tunnel had specific challenges due to their configuration also. This paper will review the methods used to overcome these common and specific challenges to successfully complete the Brightwater Central Project.

INTRODUCTION

The Brightwater Conveyance System is designed to meet the future sewage treatment needs of north King County's and south Snohomish County's expanding population. The entire system includes approximately 13 miles of bored tunnels that start at the treatment plant in Woodinville and end in Point Wells where there treated effluent outfalls into the Puget Sound. As indicated in Figure 1, the Central Contract included two tunnels, BT-2 which is a 3,536m (11,600 ft) long tunnel that runs from the North Kenmore Portal to the Influent Pump Station (IPS) in Bothell and BT-3 which is a 6,127m (20,100 ft) long tunnel that runs from the North Kenmore Portal to the Ballinger Way Portal. The tunnels are currently owned by King County Department of Natural Resources and Parks Wastewater Treatment Division. Both tunnels were initially lined with precast concrete segments which had the same finished inside diameter of 4.37m (14.333 ft).

The final linings of BT2 and BT3 differ as shown in Figure 1. BT2's final lining consists of three separate pipelines with different diameters. Two of the pipes are fiberglass reinforced pipe (FRP) and the other is a ductile iron pipe (DIP). The largest diameter pipe is the 1.83m (6 ft) FRP that conveys the treated effluent followed by the 1.37m (4.5 ft) diameter FRP that conveys raw influent. The smallest diameter pipe is

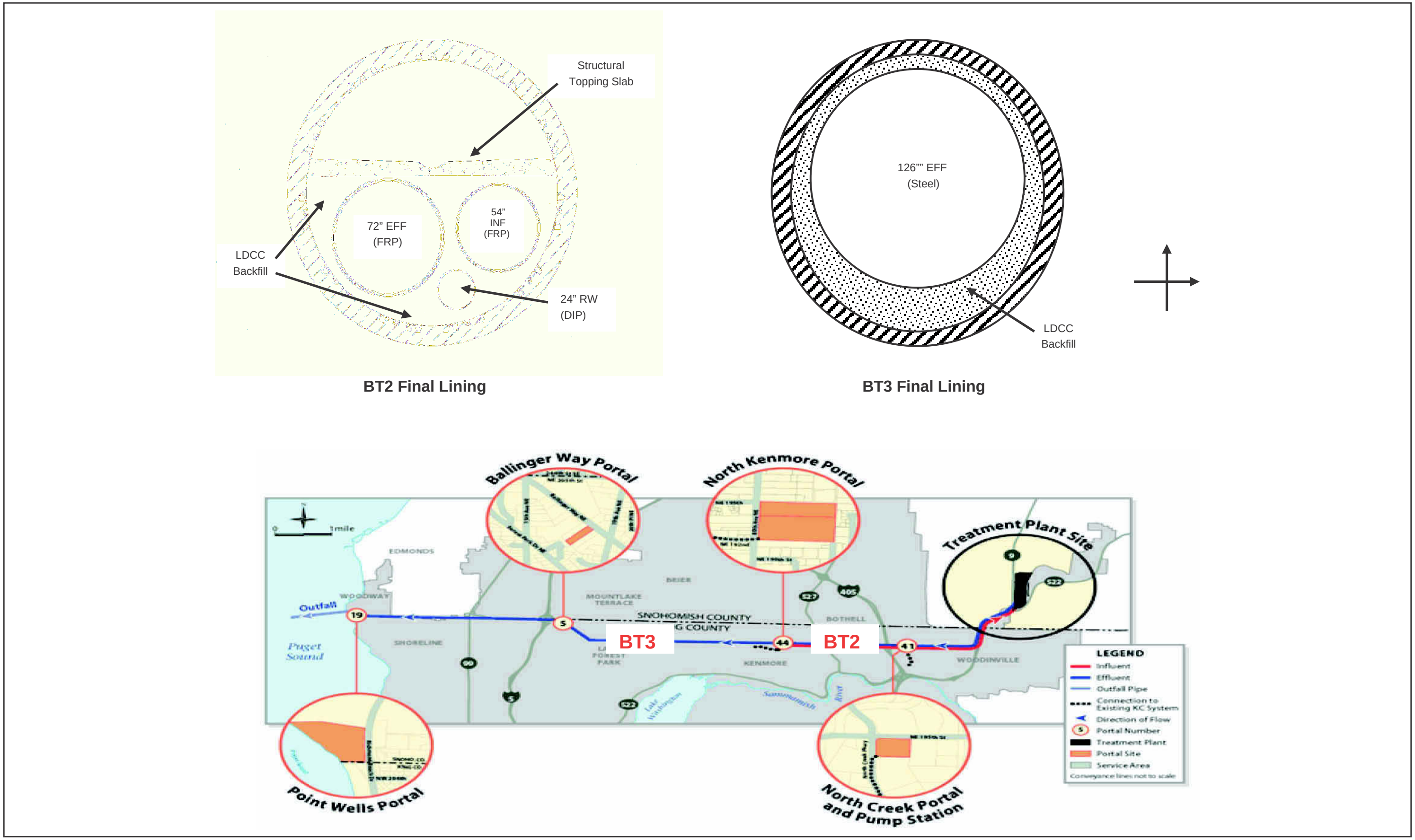

Figure 1. Project alignment, BT2 and BT3 cross sections

the 0.61m (2 ft) DIP that conveys reclaimed water. All three pipes are then backfilled with low density cellular concrete (LDCC) to just over the crown of the highest pipe. Then a structural slab is placed to cap the LDCC backfill. The slab's profile has both sides sloping to a drain trench at the center that slopes 0.253 percent from the North Kenmore Portal (NKP) to Influent Pump Station (IPS). The structural slab also included an embedded 4" conduit to run two fiber optic cables the length of the tunnel. For a majority of BT3 the precast segments are used as the final lining, for the remainder of the tunnel a secondary steel liner is installed and backfilled with LDCC. The liner is 3.22m (10.58 ft) diameter steel penstock that runs 1,400m (4600 ft) into the BT3 tunnel from NKP.

The final linings were completed sequentially with BT2 beginning and finishing before the completion of mining in BT3. The lessons learned in completing BT2 were used to complete BT3 where applicable.

BT2 PIPE AND FINAL LINING

The inclusion of a structural topping slab in the final lining of BT2 dictated the sequence of the pipe and lining installation. The project schedule called for pouring 152.4m (500 ft) of structural slab each week. Conveying the structural concrete to the placement area was the major challenge. Many options were considered, and the method chosen was to use an in-tunnel pump with a retractable slick line and Moran cars to deliver the concrete from the shaft to the pump. The pump had a maximum pumping distance of 762m (1500 ft). This limitation defined the how other two activities, pipe installation and pipe backfill were completed.

All three activities (pipe installation, pipe backfill, structural topping slab) were done concurrently and in 152m (500 ft) sections. A staggered start for each activity was used. The first two weeks only pipe installation occurred. In week three, pipe was installed in the third 152m (500 ft) section and backfill was performed in the first section. In week four, pipe was installed in the third section and backfill was completed in the second section. Week four had all three activities, pipe install in the fourth 152m (500 ft) section, backfilling of the third section and the topping slab was placed in the first section, taking place. Figure 2 shows the activity breakdown in each section. This sequence was repeated for 26 weeks to complete the BT2 final lining.

Prior to pipe being set in each 152m (500 ft) section the tracks were removed, the tunnel invert mucked out and the tunnel liner cleaned. The tracks were dismantled in 10m (33 ft) lengths, lifted onto a flat car using the Tunnel Portal Crane (TPC) and transported to the shaft for removal. The tunnel muck was placed into bucket on the same train as the tracks being removed. The TPC was an overhead monorail crane that was advanced towards the shaft and had 50m (165 ft) lifting range. This allowed removal of three to five sections of track before resetting of the monorail beam. After rail removal and invert cleaning the pipe was brought into the heading via train cars that had specialized brackets to hold the different diameters of pipe. Once at the heading, the pipes were lifted off the cars and set using the TPC. The pipe installation was complicated by a few major factors; placing straight pipe sections in tunnel curves, maintaining negative slope in the raw influent line (Φ1.37m FRP) and keeping the required clearances around the pipes for the backfill grouting.

To mitigate the challenge of placing straight pipes in a curved tunnel a couple of methods were used. Firstly, the two larger diameter FRP pipes were fabricated in three different lengths; 7.32m (24 ft), 4.88m (16 ft) and 3.05m (10 ft). The shorter pipe lengths were used specifically at the approach, in and exiting the curved portions of the tunnel. Using the shorter pipes allowed for deflecting the pipe joints just enough to get around the curves without breeching the manufacturers one degree allowable deflections at the joints. Secondly, the pipe supports used at the curved tunnel sections were

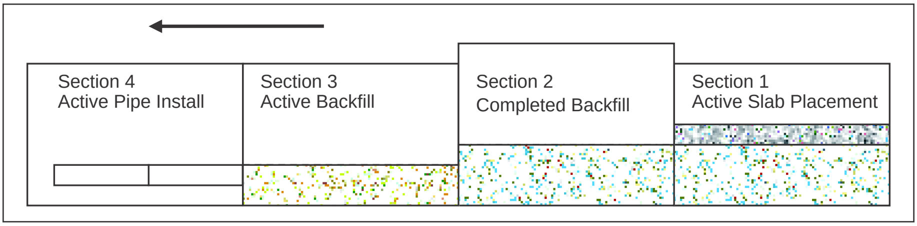

Figure 2. BT2 pipe install/final lining sequence

Figure 3. BT2 pipe install

adjustable in height which allowed the pipe to be "rolled" into the direction of the curve. The adjustable supports also aided in pipe joining as they could be lowered down to get the pipes aligned for joining and then readjusted back to the appropriate height and/or roll. Figure 3 shows the BT2 pipe cross section, the supports and the TPC monorail beam. Lastly, the entire pipe layout was prebuilt in Auto CAD using the as-built tunnel and the actual lengths of the fabricated pipe. This exercise produced the specific sequence of pipe placement required to ensure the pipe would be able get around the curves. It also produced the shipping order of the pipe from the factory to the site. The North Kenmore site had a small storage area, it only allowed for two to three weeks of installation. The shipments from the factory had to match the planned sequence of pipe installation.

For the raw influent line (Figure 3 right side) the adjustable supports were also used to achieve the required negative slope from NKP to the IPS. At the start a rotating laser was used to set the support elevations until it was found to produce too much variability. The method was adjusted to marking an elevation line, using a Total Station, of the influent line pipe bottom on the tunnel segment for the entire length of the tunnel. The pipe setting crew would then use a four foot level off the elevation line to set the support heights. The simplified method gave excellent results for maintaining the grade and it increased the efficiency of the pipe setting.

As seen in Figure 3, the clearances around the pipes were very tight. The project specifications called for a minimum of 15cm (6 in) between the pipes and 23cm (9 in) between the pipes and the tunnel liner. The specifications were there to ensure that the pipe backfill could flow between the pipes and liner. The project team was able to get the owner to reduce the clearance requirement to 5cm (2 in) after providing a mock up of the LDCC backfill operation showing the high flow ability of the mix.

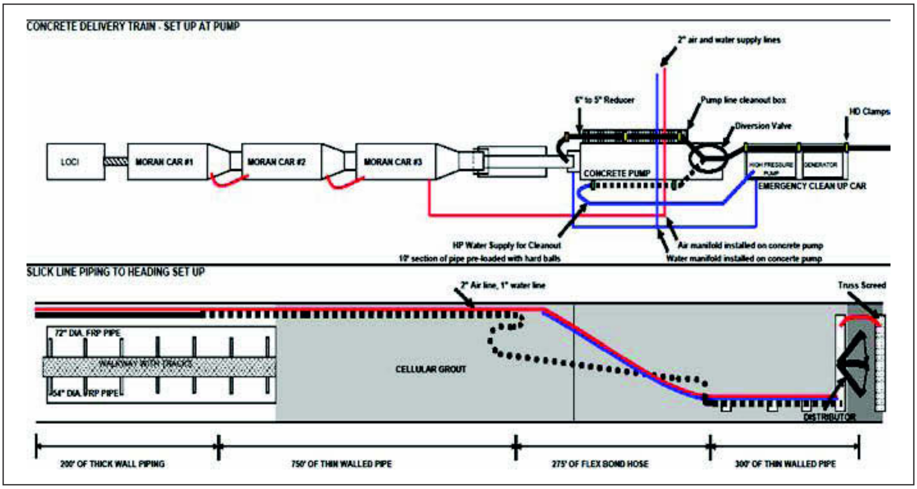

Figure 4. Concrete delivery system

The next phase, after a 152m (500 ft) section of pipe was set, was the LDCC backfill. A foam bulkhead was installed at the end of each section that prevented the backfill from seeping into the current pipe setting section. The bulkhead was not removed, instead it was left embedded in between each section. The backfill operation was initially set up in five separate lifts (one per day) for each section. The lifts were designed to reduce the buoyant force acting on the pipes which was a major concern for the project. The weights of the two FRP pipes were not enough to counter the buoyancy force so 5.1cm (2") nylon ratchet straps were used to hold them tight the supports. The five day backfilling sequence was eventually reduced to a two day sequence performing two lifts a day. Accelerator was added to the mix to achieve the proper curing before placing the second lift on top of the first.

Another challenge for the backfilling operation was the long distance pumping. Originally the plan was to put the batch plant in the tunnel, but this would have made it impossible to set pipe ahead at the same time backfilling was taking place. To keep on schedule the batch plant was moved to the surface and a 3" hose was placed in the tunnel to convey the LDCC. The first half of the tunnel backfill was batched from the Influent Pump Station side and the second half was from the North Kenmore Portal side. When the batch plant moved to NKP the lining operations were not quite at halfway which made for some long distance pumping. The longest pump was 2,133m (7,000 ft). To reach this distance the accelerator in the mix was replaced by a retarder admixture.

The last operation to complete a 152m (500 ft) section was the structural slab placement. The challenges in this phase were how to convey the concrete and how to attain the trench drain and the slab profile. Figure 4 shows the set up of the concrete conveyance. The concrete went from the ready mix trucks down the shaft chute into the Moran Agitator Cars, once all 3 cars were filled (13.8m^3) the train went to the heading. The Moran cars then dumped the concrete onto a conveyor that transported it to the pump hopper. The pump then sent the concrete through 5" slick line system that included a 91.5m rubber hose that allowed the system to retract during a pour. The total length of the 5" slick line system was 762m (2,500 ft). The original method to achieve the desired slab profile was to use a vibrating truss screed that ran on pre-set rails and had a tail form attached to form the drain. With the length of the slick line system the proper slump could not be achieved to make the truss screed effective at forming the

Figure 5. Steel drain forms and concrete distributor

drain. The slump was too high to hold the drain profile and it could not be lowered or the slick line would plug repeatedly. A steel form was used to form the drain. The forms were held to elevation and down using a packer and PVC conduit. Figure 5 shows the set steel drain forms and the concrete distributor cart.

The project team worked together to devise methods to overcome the identified challenges in the planning phase and the collaborative effort was instrumental in quickly solving the challenges faced in the field.

BT3 PIPE INSTALLATION AND FINAL LINING

In BT3, as was the case in BT2, placing straight pipe in a curved tunnel was a major challenge that was made even worse by the size and length of each pipe section (Figure 6). Each section of steel liner for BT3 was 12.2m (40 ft) long which combined with its large diameter made passing through a curved section of tunnel quite difficult. For schedule and cost reasons it was determined that using shorter lengths of pipe in the curved sections, like BT2, was not feasible. Instead, each pipe section that entered, was in, or exited the tunnel curves had one end or both ends mitered at specific angles to allow the pipe to "bend" around the corners. The required angles were determined by using the as-built tunnel CAD drawings and placing each pipe section in the tunnel with the end centerlines as close to the tunnel centerline as possible. Mitering the pipe created a subsequent challenge for the installation. The pipe had to be placed in a predetermined sequence that could not be altered. This created similar storage and shipment challenges to the BT2 installation operations and similar methods were used to solve them.

Another method used to mitigate placing straight pipe sections in the tunnel curves was the adjustable pipe supports. Each pipe was set on four independent supports that were preset to the proper grade. Initially the supports were only to set grade, but during the setting operation it was realized that the adjustment of the shaft side supports could be used to "roll" the pipe end into a better position for the next fit up. This adjustment allowed for minor corrections to get the pipe back to the as built centerline of the tunnel. One of the support bases can be seen on the left hand side of Figure 7. The base was attached to the tunnel liner using the segment bolts and then a flat plate welded to an all thread rod was inserted into the base to support the pipe sections.

The weight of the pipes presented an issue also. At 14,061kgs (31,000 lbs) the site crane did not have the capacity to lower the pipe at the shaft center. Instead the pipes were lowered onto the pipe bogie that sat on a hydraulically powered turn table as close to the south side of the shaft wall possible. Once set the turn table was aligned

Figure 6. BT3 pipe lowered onto hydraulic turn table

Figure 7. Pipe section at BT3 tunnel eye

with the tunnel eye to allow the pipe to be transported to the heading. Figure 6 shows a section of pipe being lowered and set on the turn table. Figure 7 shows how tight the clearances of steel pipe sections were to the tunnel liner were.

The small clearances were exacerbated by the ventilation system required to provide the proper air flow to the heading. Unlike BT2 which was open at each portal, BT3 was a dead end tunnel at the time pipe installation took place which created the need for a more sophisticated ventilation system. To provide the required air flow four (two per side) 0.51m (20") diameter air ducts were installed from NKP to the heading 1400m (4600 ft) away. In Figure 7 only three of the four vent ducts have been installed, but it gives a good picture of the little room there was to transport the pipe through the tunnel. In some instances, mostly in the tunnel curves, the pipe needed to be shifted horizontally to make it past the vent ducts. This was accomplished by using the hydraulics on the pipe bogie. The pipe bogie was designed with two independent saddle carts that could be shifted horizontally 0.07m (3") either way off centerline and lifted or lowered 0.14m (6").

The biggest challenge during pipe setting was in trying to obtain the specified weld gap at the pipe joints. In many cases the welding tolerances were not compatible with the manufacturing tolerances allowed for the pipe. This was especially the case with the mitered end pipe sections. The allowable waviness of the beveled miter cut of each pipe end when combined with another pipe end made the root opening in the weld wider than allowed in the welding specifications. There were two solutions to this problem; (1) field cut the ends to a tighter tolerance, (2) qualify a weld with a larger root opening than specified. Option 2 was used as Option 1 could not guarantee that the field cuts could be done with more precision than the shop cuts. The new weld

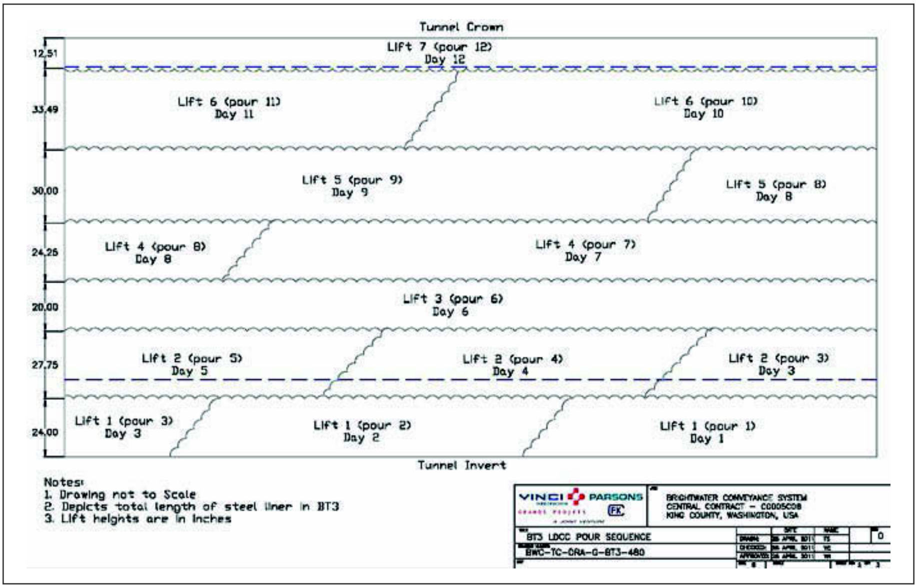

Figure 8. BT3 staged grouting sequence

procedure allowed for up to a 25mm (1") root opening between two pipes. Using the new procedure increased the amount of welding by approximately 15% for the entire pipe line. The added welding cost was well worth the time saved to field cut the pipes as it allowed installation to continue. If the field cutting option was used the installation of pipe would be on hold until the operation was complete because each pipe was set sequentially. Field cutting would potentially have created a domino effect by changing the alignment and creating a need for additional field cutting as installation continued.

After all 114 steel pipe sections were welded together, the annulus between the steel liner and tunnel liner were backfilled with the same LDCC mix used in BT2. Unlike BT2, BT3 backfill pours were limited by the volume of backfill that could be batched in a day. The upper limit that could be placed in a shift was 917.5 cubic meters (1200 cubic yards). At this upper limit the entire operation took 13 days to complete. The challenge of pumping the long distance was already overcome during BT2 backfilling and was not an issue during BT3 backfilling. The only difference was that the supply line could not be blown clean inside the pipe, so a return hose was added to allow clean out of the line to be on the surface. The mitigation of the uplift forces was quite different in BT3 compared to BT2. In BT3, crown blocking was placed every 12.2m (40 ft) near the welded joint to prevent the pipe from lifting, but the blocking created a potential for pipe deformation as the uplift on the pipe pressed against the blocking. The potential deformation was eliminated by using a staged grouting sequence which also took into account the limit of 917.5 cubic meters (1200 cubic yards) per shift placement rate. Figure 8 illustrates the staged grouting sequence used. This method of backfilling proved to be very successful with no discernible deformation on the pipe and meeting the scheduled completion date for this phase of work.

The last major challenge in completing the BT3 tunnel piping was applying coating on the welded joints and repairing construction damage to the shop applied liner. The coating required specific environmental conditions for application to be allowed with the biggest one being that the area needed to be completely dry throughout the application and curing of the coating. This was a concern because BT3 sloped down to the NKP shaft and there was water infiltration in the 4,752m (15,000 ft) of tunnel up slope from the start of the pipe. To divert this water, a dam was made to prevent it flowing in the pipe. A sump pump was then placed in the water and a 50mm (2") PVC pipe was ran from the dam to the NKP shaft to create a bypass system.

The coating also required the bare steel to be sandblasted to white metal (SP10) before applying the coating. This created a challenge to "capture" the blast media without contaminating the rest of the pipeline. This was done by creating an "air lock" structure using bulkheads to control the ventilation in the section being lined. An exhaust fan was plumbed into the bulkhead with a sand trap to collect the sand before exhausting the air into the pipe.

SUMMARY

The completion of the BT2 and BT3 final linings included over 12,192m (40,000 feet) of pipe, 23,701 cubic meters (31,000 cubic yards) of LDCC backfill and over 4,587.3 cubic meters (6,000 cubic yards) of structural concrete. The two tunnel configurations shared numerous challenges to complete, but each also had specific challenges to successfully complete their final linings. The major factor in allowing a successful completion was the time allocated to preplanning the activities and the collaboration between the engineering staff and the production staff. The collaborative engineering environment also led to quick solutions for issues in the field which helped to deliver a quality finished product to the owner. The on time completion of the tunnel final linings allowed for the shaft and site build out too start and be completed in time for the conveyance system to be turned over for testing.

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