Chapter 43

DESIGN AND RISK MANAGEMENT STRATEGY FOR THE SOUND TRANSIT BEACON HILL STATION AND TUNNELS

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ABSTRACT

The Beacon Hill Station and Tunnels portion of the Sound Transit Central Link Project consists of one mile of twin rapid transit tunnels, an underground station, portals and ancillary works constructed in an urban environment. Presently under construction, the station will set a number of North American firsts including deepest soft ground excavation carried out using the Sequential Excavation Method (SEM). Located in highly variable glacial soil deposits ranging from soft, water bearing sands to stiff, slickensided clays, the station complex and running tunnels proved to be technically challenging during design. Meeting these challenges necessitated an extensive geotechnical investigation, a trial shaft, and state-of-the-art numerical analyses to develop ground conditioning, pre-support, excavation support and structural design. A comprehensive risk identification and mitigation program, and innovative procurement strategy were incorporated into the design and contract bid documents.

INTRODUCTION

The Central Puget Sound Regional Transit Authority (Sound Transit) is proceeding with construction of the Central Link Light Rail Project, a new light rail transit line extending 14 miles southwards from Seattle towards SeaTac Airport. The one-mile Beacon Hill Tunnels and Station, shown in Figure 1, located just south of the downtown area will be the only tunneled portion in this initial segment.

Figure 1. D710 contract horizontal alignment

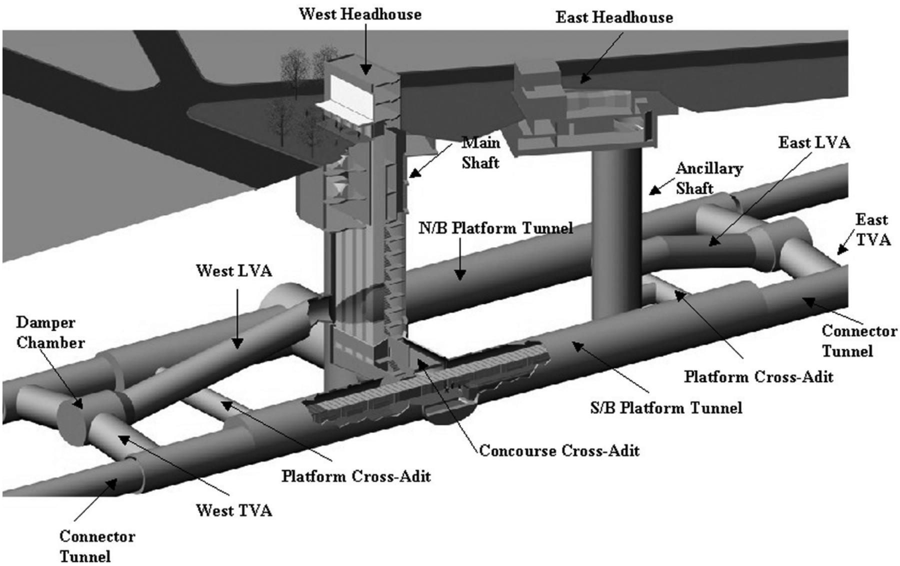

Figure 2. Beacon Hill Station general arrangement

Twin bored running tunnels, just under one mile in length, will be constructed from the west portal at the I-5 viaduct to the east portal at 25th Avenue South using an earth pressure balance tunnel boring machine (EPBM). These tunnels will be lined with gasketed and bolted precast concrete segmental linings.

The underground station, shown on Figure 2, will be constructed approximately mid-point of the running tunnels, and will consist of twin shafts and a complex configuration of vehicle, pedestrian and ventilation tunnels ranging in size from 16 to 46 feet in diameter. The invert of the platform tunnels will be 47.5 m (156 ft) below ground surface. The 14 m (46 ft) diameter main shaft will contain four high-speed elevators, emergency stairways and an emergency ventilation plenum. Tunnels will be excavated by the Sequential Excavation Method (SEM) with shotcrete liner and excavation in a head-bench-invert sequence with ground conditioning and pre-support where needed. The final liner will be cast-in-place steel fiber-reinforced concrete, with conventional bar reinforcement at junctions.

A joint venture team of Hatch Mott MacDonald and Jacobs Civil, Inc.(HMMJ) completed the final design of the Beacon Hill (D710) segment of the project between 2002 and 2004. Dr. G. Sauer Corporation was awarded a sub-contract by HMMJ for the design of the station platform and concourse tunnels. Puget Sound Transit Consultants, a joint venture of Parsons Brinckerhoff Quade & Douglas Inc., Earth Tech Inc. and URS Corporation, performed the preliminary engineering and provided program management services as an integrated team with Sound Transit.

Previous papers (Phelps, Tattersall and McAllister 2003; Gildner and Urschitz 2004; Tattersall, Gregor and Lehnen 2004) have provided an overview of the design of the Beacon Hill Station and Tunnels and the test shaft program. This paper focuses on the risk management program implemented during design and final design.

GEOLOGICAL CONDITIONS

Based on extensive drilling and testing, it was determined that the running tunnels, and station shafts and tunnels, would be constructed through an extremely complex sequence of glacially overridden deposits consisting of very dense and hard clay, silt and sand, gravel and cobbles. Multiple ground water levels were detected in granular deposits, typically due to perched groundwater overlying clay and till units. In order to simplify the descriptions of the large number of geologic units identified, and reduce complexity of the geologic profile, geologic units were grouped into five major ground types. Detail is provided by Robinson et al., RETC 2005.

- **Soft to Very Stiff Clay and Silt**: soft to very stiff, silty clay and clayey silt.
- **Till and Till-Like Deposits**: dense to very dense or hard mixture of silt, sand and gravel and varying amounts of clay. Water-bearing silt and sand lenses are expected to be present.
- **Very Dense Sand and Gravel**: non-glacial (interglacial) fluvial deposits of very dense sand, gravelly sand, and sandy gravel and lenses of gravelly cobbles.
- **Very Dense Silt and Fine Sand**: non-glacial (interglacial) lacustrine deposits of very dense to hard silt, fine sandy silt, silty fine sand, and clayey silt.
- **Very Stiff to Hard Clay**: glaciolacustrine deposits of very stiff to hard clayey silt and silty clay. Slickensided fractures and shear zones are expected to be encountered in the Very Stiff to Hard Clay, especially in the high plasticity clays.

The complex layering of these units, particularly thick zones of sands and silts, were particularly challenging for the design of shafts and tunnels for the station.

RISK MANAGEMENT STRATEGY

A comprehensive risk management strategy was implemented throughout the design of the Beacon Hill Tunnel and Station to proactively reduce risks associated with construction. While the focus of risk management was on the more complex station shafts and tunnels, the process was also an important element in the development of the design of the running tunnels. At the conclusion of the design process, a quantitative risk assessment was completed of the final configuration of the project and was used to confirm cost and schedule contingencies. Results were summarized in a Sound Transit February 2, 2004 Risk Assessment Technical Report. The following provides a review of systematic identification, avoidance, reduction and mitigation process used throughout the design phase.

Geotechnical Investigations. Sound Transit initiated a substantial subsurface site investigation program during each of the design phases. During conceptual design, a total of 44 borings (total length: 8070 ft) were made. These were augmented by an additional 32 borings (total length: 3880 ft) during the preliminary engineering and final design phases, plus a test shaft during final design.

Review Panels. Sound Transit established an independent Tunnel Peer Review Panel (TPRP) to analyze geotechnical information and monitor design development for underground elements. The TPRP includes nationally recognized tunneling experts from engineering consulting firms, contractors and academia. The panel was involved from the outset and met every 6 months on average during the design process and participated in risk assessment workshops. The design consultant also established an internal Design Review Panel (DRP) to provide advice to the design team on underground elements of the project. The Panel includes internationally recognized members of the design firms participating in the design and was involved from the outset and participated in risk assessment workshops.

Alignment Modifications. Contaminated groundwater, from a site previously used by a dry-cleaning business, was identified at depths of up to 100 feet at the originally planned location for the station main shaft. The contamination presented a significant risk to the construction of both the station shaft and tunnels due to potential construction delays. This risk was avoided by relocating the station away from the contamination plume and realigning the running tunnels to suit.

Initial soil borings indicated that the station and tunnels would be excavated through a complex sequence of clay, silt, sand and gravel. The track elevation through the station as selected to place the large station concourse and platform tunnels in the most favorable ground, which was the stiff clay deposit, thereby reducing risks for tunnel construction.

Station Configuration. The configuration of the station tunnels was selected to reduce the potential impacts of tunnel excavation on adjacent tunnels. Platform tunnels were configured in a 'binocular' configuration as shown on Figure 2, to provide the maximum practical separation between tunnels and to permit construction of inclined ventilation tunnels between. Further, tunnels were configured to facilitate excavation proceeding from large to small tunnels, which is more desirable than the reverse order.

Geotechnical Baseline Report. A Geotechnical Baseline Report (GBR) was prepared by the design consultant with input and review by the project geotechnical consultant. The GBR defined the ground and ground water conditions that should be anticipated and how the ground should be expected to behave. Because the geology was so variable with location, conditions and behavior were defined separately for running tunnels, station shafts and individual station tunnels.

Test Shaft. During the final design phase, an 18 ft diameter, 150 ft deep exploratory shaft was constructed at the location of the Station main shaft in order to expose the complex geology; provide bidders an opportunity to inspect the ground in which station complex would be excavated; and provide practical experience in excavating and supporting the ground through which the shaft and tunnels would be constructed. The sequential excavation method was selected for the test shaft in order to reflect the proposed methodology for the main contract, but it proved to be very difficult in sandy material despite extensive pumping from external wells and internal vacuum lance installations.

Two Stage Procurement Process. Recognizing the highly specialized nature of the tunneling expertise required for these tunnels, a two-stage procurement process was selected. The first stage was contractor pre-qualification in which prospective bidders were required to respond in detail to specific questions on qualifications of the proponent and senior staff. Mandatory inspection of the test shaft was required from any party intending to submit pre-qualifications. The objective of this initial step was to ensure that only contractors fully experienced in excavating tunnels of similar size, in similar ground conditions using the specified excavation methods would be eligible to bid on the construction contract, thereby reducing the owners risk. The second stage of the procurement process was bidding of pre-qualified contractors.

Dispute Review Board. Sound Transit elected to engage the dispute review board process in the construction contract to expeditiously and fairly resolve disputes, thereby minimizing lengthy and costly claims that could disrupt or otherwise threaten the project schedule.

Building and Utility Inventory. Potential ground movements due to construction activity were estimated by analytical methods. An inventory of all buildings and underground utilities was completed within the zone of influence of the running tunnels and station construction. Where there was concern about potential impacts from construction activity, a mitigation program was developed and implemented. An extensive ground surface and utility monitoring program was developed and will be implemented during construction to determine if there are ground movements that could potentially cause damage.

Geotechnical Instrumentation and Monitoring. A geotechnical instrumentation and monitoring program during construction had been planned from the conceptual design stage, however, as geologic conditions and tunnel design both became more complex, the scope of the instrumentation and monitoring program was broadened to include more closely spaced ground monitoring instruments.

Exploration During Construction. It became apparent that the geology was so complex that it would not be possible to define ground conditions sufficiently during the design phase to determine the extent of ground conditioning prior to construction of station tunnels. Additional exploratory holes were specified in the construction contract and this proved to be of significant benefit. Additional test holes drilled at the start of construction have indicated more extensive sand deposits at the eastern end of the station tunnels. Additional ground conditioning was specified and will significantly improve tunneling conditions and reduce construction risk.

Design Consultant in RE Support Role. Sound Transit elected to engage the design consultant to provide on-site engineering services for the sequential excavation method mining work at the Station. Involvement of the designer in construction of observational method tunnels has been proven to be beneficial in other projects.

Risk Registry. During the final design phase, two risk management workshops were convened with the design team, Sound Transit staff and members of the Tunnel Peer Review Panel and Consultant Design Review Panel. With the assistance of a facilitator, design and construction risks were identified and a risk registry was developed. Previously un-identified risks were mitigated by the conclusion of the design process.

DESIGN

As was indicated in the previous section, the design was undertaken recognizing the elevated level of risk associated with the complexity of the station arrangement, the depth and the anticipated geological variability. Sound Transit's direction was to acknowledge the risk up front by providing a more conservative and prescriptive design in the bid package than might have been provided in a less risky contract. This section discusses the key design elements of the D710 Contract, including how the design was influenced by the risk management approach.

Station Shafts

Originally designed as SEM excavations, the Main Shaft construction methodology was completely revised in response to difficulties with flowing sands encountered in the trial shaft. The groundwater regime was found to be significantly more complex than originally thought, with discontinuous aquifers containing sufficient fines to thwart stabilization measures using gravity and vacuum dewatering. These zones were found to contain sands that flowed with only slightly elevated pore water pressures and proved to be very time consuming to advance through. To mitigate the significant schedule risk, it was decided to construct both shafts inside diaphragm walls (slurry walls). This approach effectively eliminated ground behavior as a risk element for the shaft construction itself.

The shaft was designed as a plain concrete cylinder with nominal reinforcing. The required 30-in structural thickness needed to be increased to account for the tolerance in excavating to the depth required. A panel thickness of 48 inches (1.2 m) was specified based on a 0.5% tolerance. The large openings for the Concourse Cross-Adits were accommodated by increasing the vertical reinforcement in the panels adjacent to the breakouts.

SEM Tunnel Design for the Station Complex

The design of the SEM station tunnels is based on the principle that the disturbance of the surrounding ground must be minimized to mobilize the maximum self-supporting capacity of the soil and minimize the risk of excessive ground movement. This increases the safety factor of the excavation and reduces section forces in the tunnel lining, resulting in an economical and safe tunnel design. The following are the critical SEM design elements to reduce disturbance of the in situ stress field: optimized shape of the tunnel cross sections; timely ring closure; immediate installation of initial support; subdivision of excavation faces; and additional ground support; face support; pre-support; and ground improvement.

Excavation Sequences. In accordance with the prescriptive design approach, excavation and support sequences were developed and the following elements were specified in the contract documents: excavation sequence (top heading/bench/invert, single side wall drift excavation, dual side wall drift excavation); length of each excavation round; distance to ring closure; size of the individual openings; demolition of temporary support sidewalls; and breakout sequences at junctions of shafts and tunnels.

With this level of detail, the designers maintain tight control over each step of the excavation thereby mitigating the risk of instability during tunnel excavation caused by inappropriate tunnel construction. An excavation sequence with a single sidewall drift excavation is shown in Figure 3.

Deep Wells. As indicated previously, it was observed that the sand and silt strata exhibited flowing behavior under small heads of water, which led to problems during excavation of the test shaft. Since the performance of the contractor-designed dewatering was not satisfactory, the decision was made to include an owner-designed dewatering system in the contract. HMMJ designed a pattern of some 26 deep vacuum wells and 4 observation wells adjacent to the Concourse Cross-Adits in the vicinity of the breakouts from the Main Shaft and the Platform Tunnels.

Jet Grouting. Because dewatering had not proven to be effective, zones of wet sands and silts, that could be both clearly demarked and were of a significant extent,

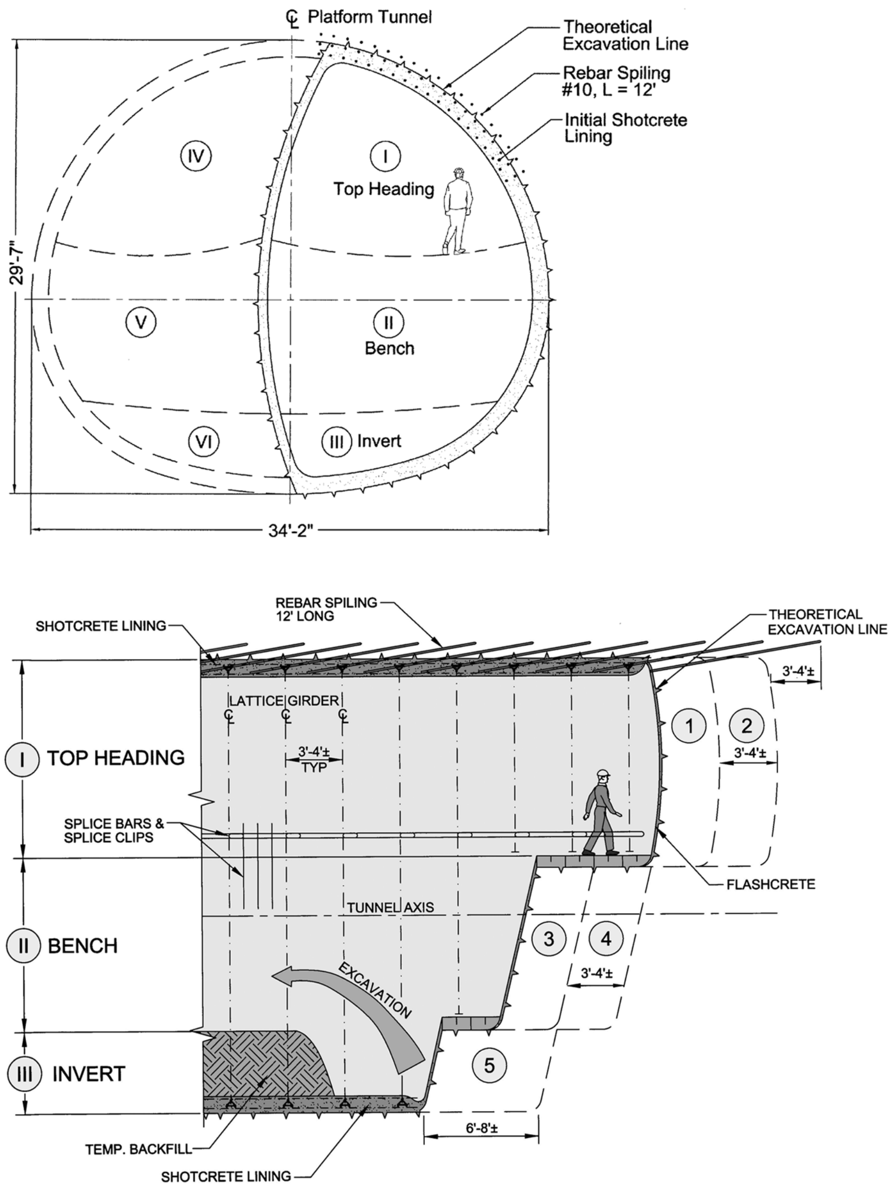

Figure 3. SEM excavation sequence

were targeted for jet grouting. At the bid stage, two such zones were believed to be the eastern half of the West Longitudinal Vent Adit and the East Damper Chamber (see Figure 1). Vertical jet grouting from the surface was called for in a performance-based specification, with a minimum bulk compressive strength of the demarcated volume of 400 psi and a maximum permeability of 3×10^{-6} cm/s.

Grouted Barrel Vault. The construction of the Main Shaft results in stress redistributions, thereby creating a zone of loosened soil around the shaft. To prevent stability problems when breaking out of the shaft and to provide pre-support during tunnel excavation, a barrel vault consisting of two rows of grouted steel pipes will be installed above the crown of the Concourse Cross Adit. A grouted barrel vault will also be installed at the NB Platform Tunnel to address the granular soils in the top heading.

SEM Toolbox Items. To address the variable ground conditions and mitigate the risk of delays occurring when the contractor encounters a situation he may not be prepared for, a comprehensive and flexible set of ground improvement and additional support measures were included in the design—the SEM Toolbox. It contains the following measures that will be used on an as-needed basis to ensure the stability of the tunnel face and the surrounding ground; these will be paid on a unit price basis:

- Pre Support Measures (Rebar Spiling, Grouted Pipe Spiling, Metal Sheets, Grouted Barrel Vault/Pipe Arch)
- Face Stabilization Measures (Face Stabilization Wedge, Pocket Excavation, Reduction of Round Length, Face Bolts)
- Ground Improvement Measures (Gravity and Vacuum Dewatering, Permeation Grouting, Fracture Grouting, Jet Grouting).
- Annular Support (Additional Shotcrete, Soil Nails, Temporary Invert)

The decision which SEM Toolbox Item to utilize will be made for each excavation round in close coordination between the contractor and Sound Transit.

Probe Drilling. To reduce the uncertainty about ground conditions ahead of the face, the systematic drilling of 35 ft long horizontal exploratory probe drill holes every six excavation rounds is specified. The results of the exploratory drilling and the assessment of ground conditions at the tunnel face will be used to determine the need for ground improvement or additional support measures (SEM Toolbox Items).

Numerical Analyses. Analysis and design of the station tunnels, including geometry, excavation sequences, initial lining and final lining thickness and lining compositions was carried out concurrently by HMMJ and Dr. G. Sauer Corp., working from a unified design approach. Junctions were analyzed with 3-D non-linear numerical analysis (FLAC^{3D} and ABAQUS). The most significant structural issue—the breakout from the Main Shaft and construction of the two Concourse Cross-Adits—was analyzed using ABAQUS and the results checked using FLAC^{3D}, with satisfactory agreement.

Station Tunnels

Because of the short tunnel lengths involved, it was determined very early in the design phase that a single base design would be provided for each tunnel. The time and cost for a contractor to change sequences to respond to a change in “class” of ground would exceed the efficiencies in material usage and sequence duration obtained. Instead, the base design would be augmented by the SEM Toolbox Items to respond to ground quality changes. The contract was written with fixed prices for the base design including the anticipated quantities of toolbox items as provided in the GBR, with unit prices for additions and deletions.

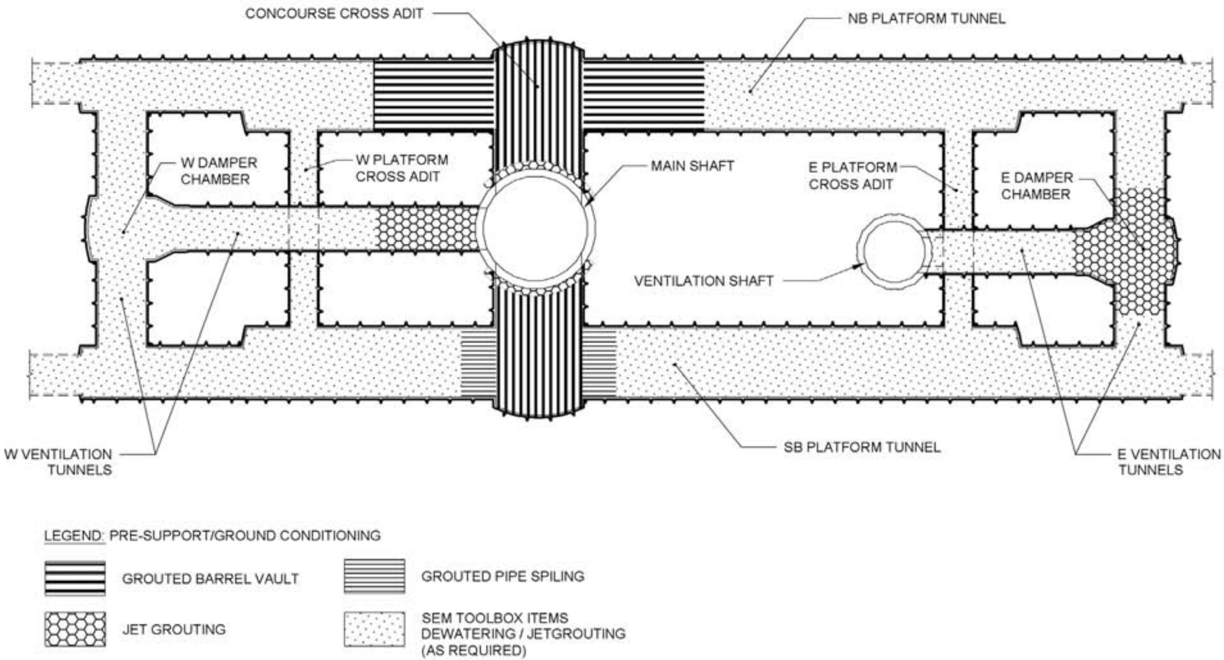

Figure 4. Pre-support and ground conditioning

Where substantial amounts of water bearing sands were predicted by the subsurface exploration program, jet grout target zones were defined in the design; they are shown in Figure 4. In addition, the barrel vaults for the Concourse Cross Adit and the NB Platform Tunnel are indicated. In the rest of the station complex, SEM Toolbox Items, additional jet grouting or dewatering by deep wells will be used as required by site conditions.

Table 1 provides a summary of the SEM designs for each of the Station Tunnels in the contract.

Concourse Cross Adit. The Concourse Cross Adit, the largest tunnel of the Beacon Hill Station, is expected to be constructed primarily in very stiff to hard clay and till and till-like deposits, with intermittent, cohesionless zones of silt and fine sand that may contain pressurized groundwater. Layers of silt and fine sand and very dense sand and gravel are anticipated at or near the crown of the excavation. Due to the large size of the opening and the difficult ground conditions, especially in the crown, the dual sidewall drift method was specified.

The construction sequence was modeled using finite element (FE) software and the numerical results were used to confirm the stability of the excavation and the excavation face as well as assess the structural performance of the initial lining, with the resulting thickness as indicated in Table 1. For the design of the final lining, it was

Table 1. Summary of SEM designs

Excavation Element	Excavated Width	Specified Excavation Sequence	Max. Round Length	Initial Lining	Final Lining
Concourse Cross-Adits	45'-4" (13.8 m)	Dual sidewall drifts with top heading-bench-invert	3'-4" (1.0 m)	14" (incl. 2-inches SFRS flashcrete)	14-inch SFRC
Platform Tunnels	36'-6" (11.1 m)	Single sidewall drift with top heading-bench-invert	4'-0" (1.2 m)	14" (incl. 2-inches SFRS flashcrete)	12-inch SFRC
Connector Tunnels	26'-10" (8.2 m)	Single sidewall drift with top heading-bench-invert	4'-0" (1.2 m)	12" (incl. 2-inches SFRS flashcrete)	12-inch SFRC
Longitudinal Ventilation Adits	22'-6" (6.9 m)	Top heading-bench-invert	4'-0" (1.2 m)	12" (incl. 2-inches SFRS flashcrete)	10-inch SFRS
Damper Chambers	33'-0" (10.1 m)	Single sidewall drift with top heading-bench-invert	3'-10" (1.1 m)	15" (incl. 2-inches SFRS flashcrete)	15-inch SFRS
Transverse Ventilation Adits	26'-0" (7.9 m)	Single sidewall drift with top heading-bench-invert	4'-0" (1.2 m)	12" (incl. 2-inches SFRS flashcrete)	12-inch SFRS
Platform Cross-Adits and Running Tunnel Cross-Passages	15'-6" (4.7 m)	Top heading-invert	4'-0" (1.2 m)	10" (incl. 2-inches SFRS flashcrete)	8-inch SFRS *

Notes:
SFRC = steel fiber reinforced concrete
SFRS = steel fiber reinforced shotcrete
* R/T Cross-Passage CIP concrete final linings sized to support R/T thrust loading

assumed that, over time, the initial lining loses 90% of its stiffness and that the full hydrostatic loading will eventually be applied to the final lining.

Platform Tunnels. The Platform Tunnels will be constructed primarily in very stiff to hard clay and till and till-like deposits, with intermittent, cohesionless zones of silt and fine sand that may contain pressurized groundwater. Layers of silt and fine sand and very dense sand and gravel are anticipated at or near the crown of the excavation in one section of the tunnel, and dry "hour-glass" sand in the invert of the Platform Tunnels in one area of the excavation. To overcome these conditions, the single sidewall drift method was specified. As for the Concourse Cross Adit, a three dimensional FE model was used for stability analyses and lining design.

Ventilation Tunnels. The geometry of the ventilation tunnels was determined based on an optimum cross-sectional area. This allowed the heights of intersection tunnels to be selected to maximize the rigidity of the junctions. Specified excavation sequences were provided for all ventilation tunnels.

Running Tunnels

The two key risk-reducing elements of the design of the twin bore Running Tunnels were a fully detailed segmental lining design and a detailed prescriptive specification for the tunnel boring machine.

Tunnel Boring Machine Specification. Geotechnical investigations for the Running Tunnels indicated that the ground would be predominantly hard clay till that, although abrasive, would provide a competent material for tunneling. However two potential problems were identified: a layer of dry sand in the lower portion of the tunnel horizon east of the station, and various zones of wet sands silts with perched water tables. To remove the potential risk of schedule delays due to using marginal equipment, the owner opted to include a prescriptive specification for an EPBM machine in the contract documents. Key performance requirements that were specified included a minimum rated EPB operating pressure of 120 psi, closeable cutterhead doors, and a minimum main bearing life.

On a previous project in similar glacial soils, HMMJ had noted that advance rates in closed (EPB) mode had been the same or better than in open mode. It was felt that eliminating the option of running in open mode would reduce risk without sacrificing production, hence EPB operation of the TBM was specified for the entire drive.

Segmental Lining Design. For the Running Tunnel linings, a proven one-pass precast bolted and gasketed segmental lining was specified and fully detailed. This design has been used on three major projects within the last 6 years with outstanding results. The only significant optimization to the design was a set of special breakout segments to be used at Cross-Passage locations.

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Chapter 76

USING STEEL FIBERS TO REINFORCE FINAL SHOTCRETE LININGS

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ABSTRACT

For short tunnels, a reinforced cast-in-place lining is not economical and shotcrete is a logical substitution. In the past, reinforced shotcrete linings have been constructed with lattice girders and welded wire fabric. For the Russia Wharf Tunnel in Boston, MA, the Contractor, with the help of the Dr. G. Sauer Corporation offered the MBTA a VECP to substitute the reinforced cast-in-place concrete lining with a steel fiber reinforced shotcrete lining. The challenge was to adequately model the ductility of the steel fiber reinforced shotcrete and to test the shotcrete prior to and during construction to assure quality control. To our knowledge, this type of numerical simulation has not been used on any other tunnels in the US.

HISTORY

Shotcrete has been used as a primary tunnel support since the 1950s. With the advent of waterproofing membranes in the 1960s, a final lining was then required to confine the membrane. In order for shotcrete to be applied to the waterproofing membrane, welded wire mesh was installed by either hanging it from embedded plastic anchors or attaching it to lattice girders. The first large cavern constructed with a shotcrete final lining was in 1987 at Washington Metropolitan Area Transit Authority's (WMATA) double-crossover at Hilderose. The next application in the early 1990s was at the Dallas Area Rapid Transit's (DART) ventilation tunnels and shafts.

Shotcrete final linings are becoming more common and have been used all over the world. The next evolution of the shotcrete lining is to reinforce it with steel fibers, thereby reducing steel fabrication costs for bending rebar or lattice girders and reducing labor cost for installing reinforcement.

PROJECT DESCRIPTION

The Massachusetts Bay Transit Authority's (MBTA) South Boston Piers Transitway is a segment of the Silver Line Phase II Busway Rapid Transit (BRT) which connects South Station to the South Boston Waterfront. Contract S0CN03, a 396 m (1300 foot) portion of Phase II, consists of a tunnel from South Station under the Fort Point

Figure 1. Contract S0CN03 layout

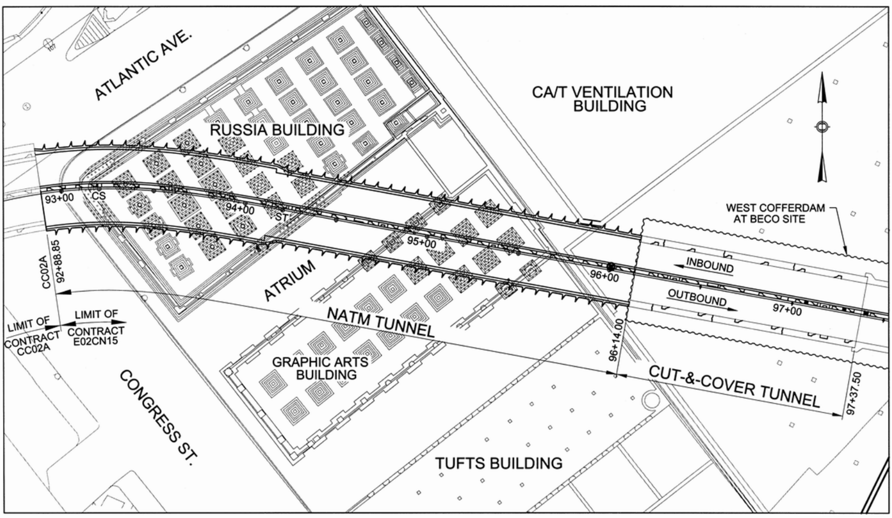

Figure 2. NATM tunnel plan

Channel to the Courthouse Station and was constructed with many different methods including the New Austrian Tunneling Method (NATM) with ground freezing, cut-and-cover, and submerged tube. This contract is broken down as follows and shown on Figure 1.

- 99 m (325') of NATM mined tunnel,
- 56 m (184') of cut-and-cover tunnel,
- 213 m (700') of submerged tunnel,
- 31 m (103') of cut-and-cover tunnel.

This paper will focus on the design and construction of a steel fiber reinforced shotcrete lining as the permanent structural lining for the 99 m (325-foot) long NATM tunnel section as shown in Figure 2.

PROJECT CHALLENGES

The NATM tunnel section crosses under two hundred-year-old-historic buildings, the Russia Building and the Graphic Arts Building which are connected with an atrium. Both buildings are resting on numerous granite pile caps, each of which transmits their load onto 32 wooden piles. As the buildings were to remain in operation during the construction, measures had to be taken to reduce the effects of the tunneling operations on the buildings. First, hydraulic jacks were installed under the buildings in order to prevent them from locally displacing and causing damage to the façade, walls, door jambs, etc. Then prior to tunnel construction, the Russia Building's foundation was transferred onto drilled mini-piles which were installed and located to either side of the future tunnel and through the future tunnel center wall. Under the Graphic Arts Building, ground freezing was used as a pre-support method to increase the strength of the soil above the crown of the tunnel, increasing the friction in the piles and therefore reducing the load at the bottom of the piles. Due to the heave associated with the soil freezing operation and the settlements resulting from the tunnel excavation process, the Graphic Arts building was lifted off of its foundation with hydraulic jacks and was adjusted and leveled as the mining progressed below. As the tunnel was excavated, the wooden piles from the Graphic Arts building were exposed, cut off above the tunnel, a reinforcement shoe was installed and shotcrete was applied incorporating the piles into the initial tunnel shotcrete lining which ultimately bears on the final structural lining as shown in Figure 3. After the tunnel construction and during the thawing process, the loads from the Graphic Arts Building were transferred onto the tunnel structure.

GEOLOGY

The geological series at the project site is, from the surface down, Fill, Organic Deposits underlain by Marine Clay and Glacial Till. The soils can be briefly described as follows:

Fill. The fill is highly variable in composition. In general the fill is described as black to brown fine to coarse sand with varying amounts of gravel and plastic fines. Brick fragments, wood, ash, and cinders, as well as organic matter and lumps of clay are common constituents. SPT N-values vary from 2 to 47 blows per foot.

Organic Deposits. The organic deposits are described as gray to black organic silt with fine sand, peat, and occasional shell fragments. The organic deposits also contain beds of fine sand and peat. Peat beds have been measured up to four feet in thickness. The SPT N-values for the organic deposits range from 0 to 17 blows per foot.

Marine Clay. This material is described as gray, low to medium plastic clay with frequent thin partings and laminae of silty fine sand. The upper part of the clay is highly over-consolidated with a yellow-brown color. SPT N-values for this desiccated crust range from 5 to 15 blows per foot. Overall the SPT N-values for the Marine Clay range from 0 to 44 blows per foot, generally increasing with depth.

Glacial Till. The glacial till typically consists of a heterogeneous mixture of sand, silt, clay and gravel, with occasional cobbles and boulders. Glacial till with a fines content of greater than 30% is classified as cohesive till, and till with a fines content of less than 30% is classified as granular till. The cohesive till typically consists of clayey silt with low plasticity. SPT N-values for the Glacial Till range from 33 to >100 blows per foot.

The tunnels are mainly driven in the Organic Deposits and the Marine Clay. The unfrozen strength parameter for this layers range from a shear strength of 36 kPa for the Organic Deposits to 48 kPa for the Marine Clays. Soil testing performed for the

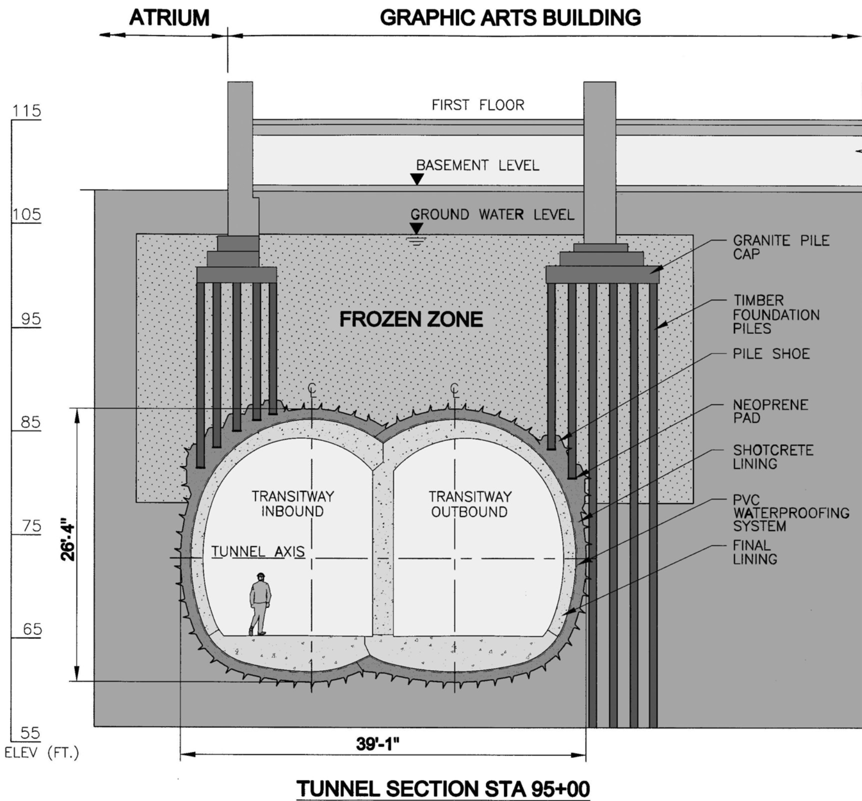

Figure 3. Tunnel cross section under the Graphic Arts building

frozen soils indicated that the shear strength in the frozen state increases to 860 kPa for the Organic Deposits and to 1130 kPa for the Marine Clays.

FINAL TUNNEL STRUCTURAL LINING

The original design specified a reinforced, cast-in-place concrete final lining ranging in thickness from 36 to 56 cm (14 to 22 inches) and typically reinforced with #4 bars each direction at 15 cm (6 inch) centers as shown in Figure 4. The concrete was specified to have a minimum compressive strength of 34,500 kPa (5,000 psi). Since the Contractor reduced the original NATM tunnel length of 126 m to 99 m (415 feet to 325 feet) with a previous Value Engineering Change Proposal (VECP) and due to the cost for concrete formwork as well as labor costs associated with erecting (or installing) rebar reinforcement, it was reasonable for the Contractor to want to use steel fiber reinforced shotcrete to Value Engineer the final lining. Since the Dr. G. Sauer Corporation designed the tunnel excavation and linings using a 3-Dimensional Finite Element model, it was more efficient for them to assist the Contractor with the Steel Fiber Reinforced Shotcrete Final Lining VECP.

The goal of the VECP was to replace the cast-in-place concrete with shotcrete, and to replace the rebar with steel fibers. Since the Contractor had already a state-of-the-art

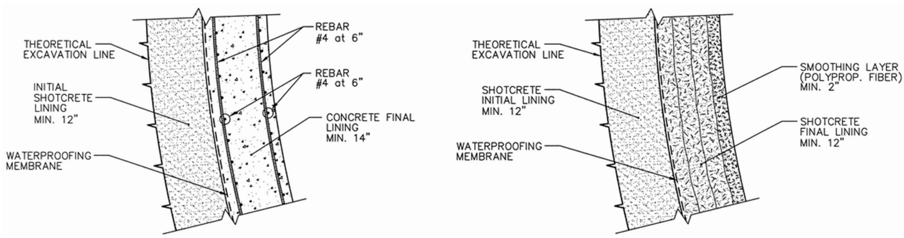

Figure 4. CIP concrete and SFR shotcrete sections

computerized batch plant on site which was consistently producing well in excess of the minimum compressive strength, the challenge was to model the shotcrete and to specify additional testing to verify that the steel fibers were adequately dosed and mixed in the shotcrete and that the shotcrete was properly applied in the tunnel.

The steel fiber reinforced shotcrete (SFRS) final lining section consisted of a minimum of 30 cm (12 inches) of steel fiber reinforced shotcrete covered by a 5-cm (2-inch) smoothing layer of polypropylene fiber reinforced shotcrete as shown in Figure 4. The smoothing layer was applied to completely cover any steel fibers which were protruding thereby protecting the public and maintenance workers against injury.

The steel fibers were specified to have a length between 1.9 cm and 3.8 cm (3⁄4 of an inch and 1-1⁄2 inches) and an aspect ratio between 45 and 85 with bent or deformed ends. And the steel fiber dosage was specified to be 35.6 kg/m^3 (60 lbs/cy) of shotcrete.

STRUCTURAL CALCULATIONS

At this stage a brief description of the three dimensional Finite Element Analyses as it was performed in order to verify the designed excavation and support sequence is given. For the definition of an initial condition, building loads are transferred by the timber piles into the ground in a layer of desiccated clay at the interface between Organic Silt and Marine Clay. The model is balanced for this situation, i.e., there are no deformations associated with this loading condition. After ground freezing the building loads are transferred from this desiccated layer into the frozen ground body as tunneling proceeds. The building loads (300 kN/m^2) are transferred to the frozen soil strata approximately one tunnel diameter ahead of the tunnel face. For this simulation piles are not assumed to act as individual members transferring discrete forces onto layers of soil or frozen soil, but as distributed loads acting on individual faces of elements of the finite element mesh.

Soils and frozen soils are modeled as elastic-plastic materials utilizing a Drucker-Prager yield criterion. Freezing of soil is modeled by increasing the stiffness and shear strength of the elements forming the frozen body in the FE mesh. The primary shotcrete lining is modeled as an ideally elastic material.

Typically, we would use this assumption for the final lining as well. However, as the Contractor wished to replace conventional rebar or mesh reinforcement with steel fiber reinforcement, a different approach was chosen. Steel fiber reinforced concrete or shotcrete displays a distinctly ductile behavior. Therefore, in order to utilize this quality, the post cracking behavior of the steel fiber reinforced concrete/shotcrete was defined and included in the material behavior for the final lining in the FE analysis. Steel Fiber Reinforced Shotcrete (SFRS) for the final lining is modeled using the “Concrete” material model included in ABAQUS. In this material model, the steel fiber reinforced

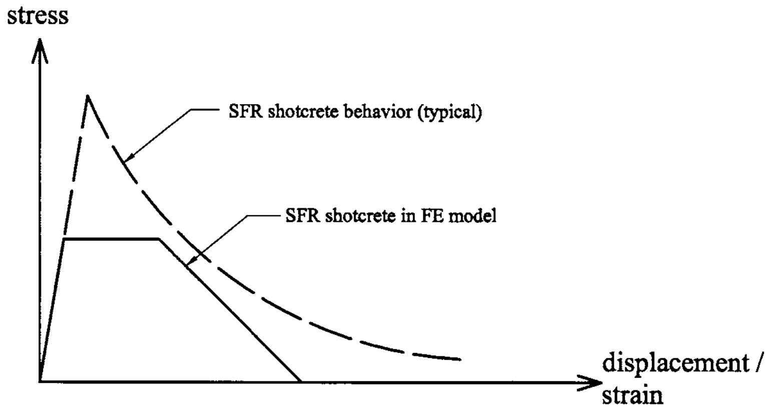

Figure 5. SFR shotcrete stress/strain curve

shotcrete is assumed to be an elastic-plastic material. Its stress-strain curve is defined as ideally elastic up to the first crack in the shotcrete. The post-cracking behavior of steel fiber reinforced shotcrete is defined as a series of stress/strain points along the plastic-behavior line (see Figure 5).

As shown in Figure 5, for reasons of numerical stability and efficiency, the stress peak at the first crack and the following drop of stress, that may be significantly sharper than that shown, is not included in the material model but replaced by a linear plastic behavior (yield plateau) following the elastic zone of steel fiber reinforced shotcrete and a succeeding linear reduction to zero flexural stress at a limiting strain.

The challenging part of modeling was to represent the behavior of the SFR shotcrete tested on site adequately in the model and achieve the required numerical stability in order to obtain reliable results. In a first approach, the requested ductility of the SFR shotcrete could not be achieved with the mix used on the site. As the shotcrete mix was also used for the initial lining support, with the requirement of high early strength (and in this case followed by very high final strength) ductility of the SFR shotcrete was low. However, after several iterations coordinated with the contractor, material properties that could be matched on site and that delivered consistent analytical results were established.

As the post cracking behavior of the material is included in the analyses and, therefore, has a major impact on the feasibility and the reliability of the results, it is essential that SFR shotcrete used in construction does perform as it is assumed in the analyses. Strict quality control during construction and performance of the installed materials according to the defined specifications is essentially required and, therefore, extensive testing prior to and during construction is necessary.

SHOTCRETE TESTING

In order to assure that the SFRS met the minimum structural requirements, shotcrete panels were shot prior to applying and during application of the SFRS final lining and tests were performed as listed in Table 1.

It was also specified as in any other shotcrete operation that the nozzelmen must be certified by ACI, have experience applying shotcrete on at least two previous projects and has successfully demonstrated to the Engineer that he is capable of

Table 1. Summary of shotcrete testing

Type of Test	Min. Required Values	
Compressive Strength		
1 day	14.5 MPa	(2,100 psi)
7 days	28.3 MPa	(4,100 psi)
28 days	34.5 MPa	(5,000 psi)
Flexural Strength at First Crack		
7 days	3.0 MPa	(440 psi)
28 days	3.7 MPa	(540 psi)
Residual Strength Factor $R_{30/10}$	70	
Toughness Indices		
I_{10}	8	
I_{30}	22	
Bond Strength		
28 days	1.0 MPa	(145 psi)
Steel Fiber Content	23.7 kg/m^3	(40 lb/cy)

applying shotcrete and adhering to the specified smoothness criteria. In addition, all shotcrete applications must be performed under the immediate supervision of a foreman with at least three years of shotcrete experience.

Regarding the shotcrete application, in order to improve the bond between the shotcrete layers, it was important to specify that all surfaces receiving shotcrete must be cleaned to remove loose material, mud, dust, rebound and other foreign material and that all shotcrete and concrete surfaces are kept moist until shotcrete is applied.

CONTACT GROUTING

In order to transfer the loads from the initial tunnel lining to the SFRS final lining, it is important for the final lining to tightly abut the initial lining. Typically with cast-in-place final linings, contact grouting has to be performed to fill up the gap at the top of the tunnel left by the concrete placement operation. However, with shotcrete, there is no gap at the top of the tunnel but instead there is a smaller gap typically around the crown of the tunnel due to shrinkage during curing. Therefore contact grout pipes were installed at 10, 12 and 2 o'clock as shown in Figure 6 and grouted with cementitious grout.

SUMMARY

Shotcrete final linings, like other construction technologies, have been used for many years and must evolve with the advent of new materials. Steel fibers can be modeled to take the place of traditional rebar reinforcement or lattice girders with welded wire mesh. The key to a successful SFR shotcrete final lining starts with properly modeling the tunnel lining and developing a proper material testing program, complimented by thorough inspection and a Contractor who emphasizes quality.

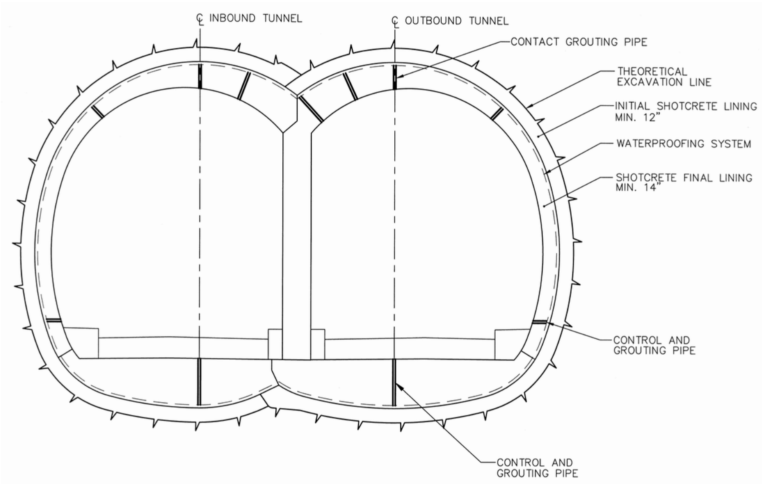

Figure 6. Typical tunnel section showing grouting pipes

Figure 7. Finished inbound and outbound tunnels

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