Design and Construction of a Support of Excavation System for the Silver Line Subway in Boston

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ABSTRACT

This paper presents three (3) case histories associated with the construction of the Support Of Excavation (SOE) system for parts of the new underground (Phase II) Silver line subway system in downtown Boston. The first case study addresses the design and construction of the cantilevered sheet piles at the Fort Point Channel. The design and construction of a drift tunnel below the Russia Wharf complex is presented as the second case history. Finally, the third case study presents the analysis and design of the support of excavation system for the construction activities at the West Cofferdam.

To allow the construction of the perimeter walls for an underground garage using the bentonite slurry technique, a 25 to 30 ft thick flowable fill material was placed at the west shore of the Fort Point Channel. The placed flowable fill was retained using an HZ pile system that was constructed to cantilever 25 to 30 feet. The performance of the HZ piles was within design expectations.
The second case history addresses the construction of a drift tunnel located adjacent to the exterior wall of an existing tenant occupied historic building. This was undertaken to allow the permanent underpinning of said exterior wall.

The third case history addresses the design and construction of a cofferdam for the construction of a cast-in-place transition between the immersed tube tunnel sections under the Fort Point Channel and the NATM tunnel under the Russia Wharf complex. A Value Engineering Cost Proposal (VECP) was proposed and implemented to eliminate 90 feet of NATM tunnel by extending the cofferdam and to reduce the number of bracing levels from five to three. The underground tunnel and the surrounding structures were analyzed using three dimensional finite element models that utilized nonlinear soil-structure interaction.

INTRODUCTION

This paper presents engineering work associated with Phase II of the Silver line tunnel owned by the Massachusetts Bay Transportation Authority (MBTA). A two-lane subway tunnel designed for immediate use of electric buses will be constructed with the ability in the future to convert to light rail. The Phase II alignment begins at South Station, travels north along Atlantic Ave, turns east and travels under the Russia Wharf Complex, through the Boston Edison Company (BECo) Site, continues through Fort Point Channel (FPC) to South Boston, and ends near the Boston's World Trade Center. Following the VECP the contract alignment consists of 325 feet of NATM tunnel, approximately 300 feet of cut & cover tunnel, and 700 feet of immersed tube tunnel. Refer to Fig. 1 for the project plan

The coastline in the 1800's passed through the BECo site, where several wharves and bulkheads were constructed. Most of these structures were buried and abandoned in place during subsequent harbor filling operations. The subsurface at the BECo site contains an old stone seawall of unknown configuration and several generations of abandoned wooden wharves supported on timber piles. The BECo site is to be developed as an underground garage with reinforced concrete slurry wall along its perimeter. The construction of the slurry walls required the installation of 25 to 30 ft cantilevered HZ sheet piles. These sheet piles are the subject of case history no. 1.

The New Austrian Tunneling Method (NATM) in conjunction with ground freezing is used to construct the binocular silver line tunnel below the Russia Wharf Complex, which is a group of three seven-story buildings constructed in the early 1900s and supported on timber piles with granite block pile caps. The construction of the NATM tunnel requires the underpinning of some pile caps that support the historic buildings. A drift tunnel, presented in case history no. 2, is excavated below the building to facilitate the underpinning of the pile caps.

Precast concrete immersed tubes are used to construct the tunnel below the Fort Point Channel, and cast-in-place concrete tunnels are used as a transition between the immersed tubes and the NATM tunnel. The cast-in-place structure is constructed within a cofferdam, discussed in case history no. 3, on the shores of the Fort Point Channel.

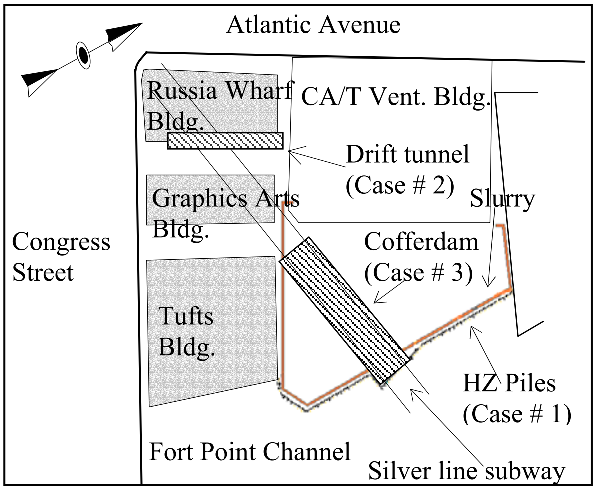

Fig. 1. Project plan.

The crux of the engineering design performed for those case histories was to use a finite element analysis method to reanalyze the SOE system and take advantage of the analysis of more detailed staging determined in the construction phase. The results of the analyses demonstrated deformation within allowable limits, and lower line loads at fewer bracing levels, compared to the original design.

ANALYSIS METHODOLOGY

Classical Analysis

This approach uses the Rankine Theory of earth pressure for the analysis and design of braced excavations. Other methods might use apparent pressures instead of the Rankine theory. The lateral pressure, which may include earth, surcharge, and hydrostatic loads, is imposed on the active side of the wall, and a series of springs are used to model the passive resistance of the soil, hence, such models are referred to as "Soil Spring Models".

Generally, Soil Spring Models are simple to formulate (SEI/ASCE 2000), and can be analyzed using relatively simple computer software. Analysts and engineers tend to assign conservative soil parameters for the modulus of subgrade reaction, this leads to conservative estimate of the support of excavation stresses and displacements. The Rankine Theory assumes that the lateral pressure on the wall is independent of the wall displacement. Furthermore, the excavation impact on adjacent structures and the soil deformations cannot be easily inferred from the classical analysis. "Stick" models that implement the classical approach cannot capture the impact of the soil heave and elastic deformations at the toe of the wall on the behavior of the wall (Hagh *et. al.* [2001]). These shortcomings of the classical approach were among the driving factors that motivated the development of more sophisticated finite element analyses.

Finite Element Analysis

Finite element analysis methods can be implemented with a variety of commercially available software. The important difference between this approach and more conventional, classical methods is that the models incorporate not only the structural system, but the surrounding soils and adjacent structures (as surcharges) as well. These systems work together as the soil models both load and support the structural elements. Furthermore, by incorporating the constitutive non-linear equations for the various soils, the models more closely imitate the true behavior of the soils than the separate systems of loads and springs in conventional beam on elastic foundation models.

Soil Models

The soil is modeled as four-noded plane strain elements, in which the strain normal to the plane of the section is assumed to be zero. Soil material is generally modeled as either (a) Multilinear Isotropic, or (b) Drucker-Prager. Multilinear isotropic materials, used for cohesive soils such as clays and organics, contain the hyperbolic stress-strain relationship developed by Filz, Clough, and Duncan (1990). The primary soil parameter for this material model is the undrained shear strength. The Drucker-Prager model, used for cohesionless soils such as fills and glacial till, describes materials whose strength increases with depth. The primary soil parameter for this material model is the friction angle. Good quality rocks are modeled as elastic materials. The soil parameters used in the finite element analyses were derived from the geotechnical report prepared by the geotechnical consultant.

CASE no. 1: CANTILEVERED SHEET PILES

Site and Work Description

The development of the BECo utilizes a perimeter reinforce concrete slurry wall. Approximately 307 feet of this wall runs straight along and parallel with the Fort Point Channel. The panels vary in depth from 50 to 90 feet. Land needs to be reclaimed to construct this wall. The original design calls for sheet piles, supported by battered piles, to retain the reclaimed land.

A typical cross section at the cantilevered sheet piles is shown in Fig. 2. The original design called for the installation of the sheet piles and the supporting battered piles inside the Fort Point Channel. The organics "muck" material behind the sheet piles is removed through a dredging process, and then a flowable fill is placed between the sheet piles and the existing sea wall. Reinforced concrete slurry wall panels, 10 ft wide, are excavated and constructed using the bentonite-tremie technique. Later in the project, the width of the slurry wall panels was increased to 12 ft.

The construction sequence, refer to Fig. 2, required driving 16" ϕ

battered piles to the glacial till soil to develop a design load of 125 tons. Sheet piles are driven behind the battered piles, and gaps are left in the sheet piles in order to eliminate any differential hydrostatic pressure on the sheets. Once the walers are installed and the sheet piles are connected to the battered piles, the gaps are close. During the next stage the organics are dredged for about 5 to 6 feet behind the sheet piles and controlled density fill (CDF) is placed in layers of 5 ft thick. Subsequent layers of CDF are placed after the previous layer had set and cured, thus eliminating the fluid pressure of the previous layer. The placement of the CDF would be performed sequential in 30 ft wide slots. Finally, the slurry wall is constructed using the tremie bentonite technique, and the panel width would be limited to 10 ft. The installation of the battered piles and their connections to the sheet piles would require working off a barge located in the FPC. Such work would be greatly impacted by the tidal effects. Moreover, installing the battered piles would have made it difficult to construct the Tufts and BECo Wharves at a later date. The wharves are supported on a grid of drilled piles that would be driven around the battered piles.

Due to those constructability issues associated with battered piles, it was logical to explore other venue that would eliminate them, hence, the cantilevered sheet pile idea was conceived. Preliminary engineering studies indicated that the stiffness of the Support of Excavation System (SOE) rather than the strength would control the design of any SOE system. The HZ pile system selected for this project consist of combination of a 30" deep wide-flange "king" pile and AZ piles. Further more, the width of the slurry wall panels was increased from 10 ft to 12 ft, thus increasing the efficiency of the slurry wall construction.

The HZ wall system was installed using a 12 ton vibratory hammer supported from a barge crane on the Fort Point Channel. Installation of the HZ wall system was difficult due to deep obstructions.

Analysis and Design of Sheet Piles

The analysis of the sheet piles should account for all forces that are applied to sheets piles at different stages of construction. The sheet piles had to retain the flowable fill, once the fill sets, a differential hydrostatic pressure due to the tidal effect will generate on either side of the sheet. Finally, the sheet piles had to retain the hydrostatic pressure due to the bentonite and the subsequent wet concrete. Classical analyses techniques could not be applied easily to such a problem, and one had to consider finite element of finite difference modeling. Now the issue becomes whether to use a simple two-dimensional plane strain analysis or a more complex three-dimensional modeling. While plane strain analysis could capture the effect of the pressures due to the flowable fill and the differential hydrostatic pressure, this analysis failed to simulate the construction of the slurry wall.

Three-dimensional models were used to analyze the proposed SOE system. Shell and brick elements were used to model the sheet piles and soil mass, respectively. Those models simulated the installation of the sheet piles, the dredging process, the backfilling with flowable fill, and finally the excavation and pouring of the concrete for the slurry wall panels.

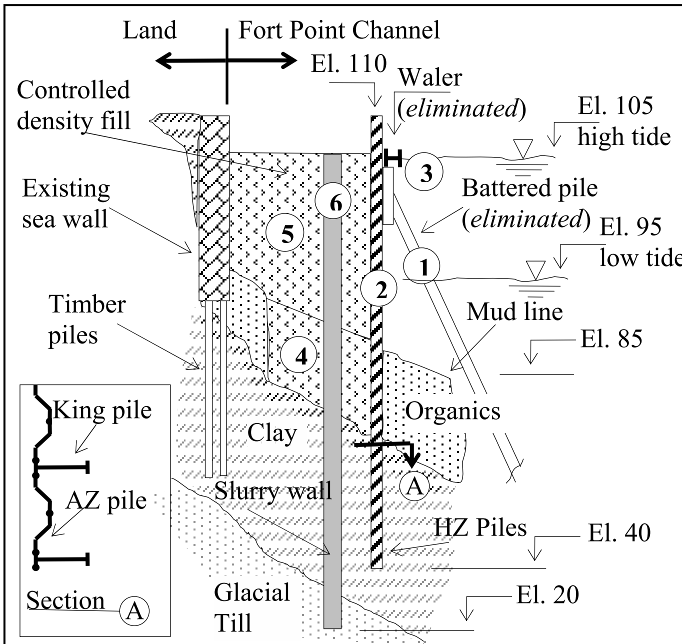

Fig. 2. Typical cross section at HZ piles.

The results of the finite element were used to size the sheet pile system. Those confirmed that the stiffness rather than the strength would control the design of the sheet piles, as predicted by the preliminary engineering studies. The size of the sheet piles was selected such that the deflection at top of the sheets is in the range 2" to 3".

The analyses indicated that the pouring of the concrete for the slurry is the most critical stage of construction.

Actual Behavior of Sheet Piles

Deformation monitoring points were established at top of the sheet piles to monitor their lateral deflection. During the concrete placement at the slurry wall, some sheet piles showed deflections that exceeded the 3" value predicted by the finite element analyses. Review of the excavation logs of the slurry wall panels just behind those sheets indicated that battered timber piles, as long as 18 ft, were extracted during the excavation of the slurry wall panel. Such timber piles were concealed below the dredged organics in the clay layer; therefore, they were not extracted during the dredging process. The extraction of such large piles during the excavation of the slurry wall panel disturbed the soil at the toe of the sheet piles causing reduction in the passive pressure mobilized behind the cantilevered sheets. Furthermore, direct communication between the sheet piles and the slurry wall panels might have occurred, causing more direct fluid pressure on the sheets. This was evident by the over-pour recorded for those panels.

The contract documents' limitations on the lateral deflection of the sheet piles were intended mainly to insure that the verticality of the interior face of the slurry wall is within tolerable limits. A 20 ft deep test pit was excavated in front of the slurry wall panels that were supported by sheet piles that exceeded the predicted deflection limits. The test pit allowed visual examination of the joint between adjacent slurry wall panels. Also, the verticality of the slurry wall panel was measured. The slurry wall joints were found to be sound, and the verticality of the slurry wall panels was within contract acceptable limits.

CASE no. 2: DRIFT TUNNEL

Site and Work Description

To facilitate installation of the permanent underpinning system along the east façade of the Russia Building, it was necessary to gain access to the existing building foundation. To gain this access a tunnel had to be constructed such that the Atrium would not be negatively impacted. The tunnel would be 60 feet long to clear the Atrium and 14 feet wide by 20 feet deep to accommodate installation of the mini-piles used to underpin the building foundation. An almost vertical approach shaft led to the 20 ft deep drift tunnel. The approach shaft was used as an access hole to lower construction equipment and remove excavated material out of the tunnel, refer to Fig. 3. Prior to the existence of the Atrium, which is located between and serves as an entrance to both the Russia and Graphic Arts Buildings, a cobble stone road existed. The contract documents proposed installing steel soldier pile and lagging system. The contractor considered several options including shield jacking, drilling a horizontal jet grout curtain and artificial ground freezing. After careful evaluation, steel soldier pile and lagging system was chosen as the preferred tunneling method, refer to Fig. 4. The contract documents indicated that the Atrium floor was a 12 inch concrete reinforced slab. Field evaluation determined that the slab thickness varied and was more in the range of 5 to 6 inches thick. Since the floor slab would form the top of the tunnel a maximum horizontal spacing of 4'-0" was established for the steel elements supporting the tunnel and the above structure. The tunnel being 20 feet in depth required that it be excavated in two headings, a top and a bottom. The top and bottom headings would be excavated simultaneously with a stagger. The steel sets were designed and fabricated such that the upper portion could be installed and temporarily supported in the top heading. Once the bottom heading arrived the remaining lower portion of the steel set could be installed to complete the drift. Tunneling began at the Congress Street side of the Atrium. The actual excavation of the drift tunnel was executed as planned. With delays due to obstructions encountered, excavation of the tunnel took approximately 6 weeks to complete.

Constructability Issues

The major construction concern was the uncertainty regarding the exiting conditions. As-built conditions of the granite pile caps and the existing obstacles and structures buried below the Atrium slab required much field fit-up work to install the steel elements. During excavation, an abandoned brick masonry manhole was encountered, which required abatement prior to removal. A significant amount of timber cribbing was also encountered. This timber cribbing most probably are the remains of preexisting wharf structures that predated the Russia Wharf complex. The close proximity of this project to the Fort Point Channel created a ground water flow issue into the tunnel which required continuous water pumping during the excavation.

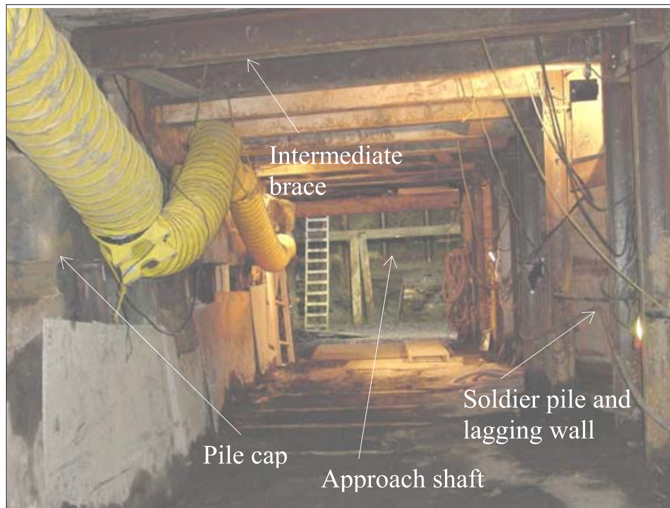

Fig. 3. Drift tunnel.

Design of the SOE System

The contract documents provided the lateral loads for the design of the various elements of the bracing system. The gravity loads were estimated based on the available drawings and occupancy of the structure.

Due to the size restriction of the drift tunnel, all steel elements had to be designed such that they can be handled manually. Light W8, W10 and W12 structural steel beams were used for the soldier pile and lagging wall and the intermediate braces. The steel elements were designed with bolted connections, and splices to reduce the weight of individual elements.

Timber blocks were used as footings to support the soldier pile and lagging and distribute the axial load, due to underpinning, to the ground.

During the excavation of the drift tunnel, the design of the support of excavation system was modified at certain areas to account for unforeseen conditions that were revealed during excavation. The design modifications were quick and prompt, since the excavation can not be left for extended periods without support.

Movements within the Atrium due to the tunneling were kept to within allowable limits. No apparent distress to the concrete floor slab and stairs due to tunneling were observed.

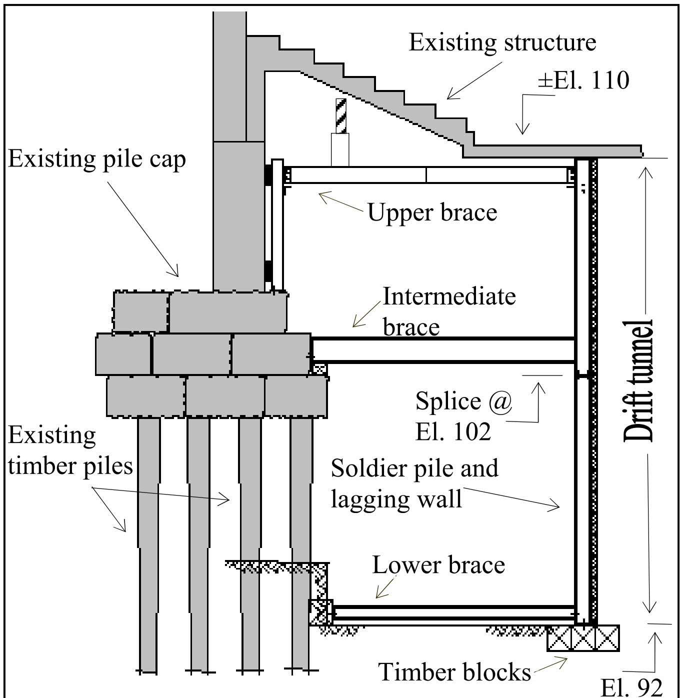

Fig. 4. Typical section at drift tunnel

CASE no. 3: COFFERDAM CONSTRUCTION

Site Description

As shown in Fig. 1, this project is located on the west shore of the Fort Point Channel. The site of the project contained a massive 9 ft thick concrete slab supported by a dense grid of timber piles spaced at 2 ft on center. This structure was constructed during the early 1900's to support power generators in station no. 3 of the BECo. Preliminary investigation indicated that the upper 20 ft of the soil mass to be contaminated which would require special treatment before commencing mass excavation. The decontamination process was performed after driving the sheet piles.

The soil profile at the site of the project consisted of about 5 to 15 ft of various fill materials followed by 5 to 10 ft of soft organics. A 40 ft thick clay layer underlain the organics and is supported by about 15 ft thick Glacial till deposits. The excavation of the cofferdam was approximately 50 ft wide and 60 ft deep, extending through the fill and organic soil and into the clay stratum.

Contractor-Proposed VECP

The contractor's inspirations for initiating this change were threefold: cost savings, time savings and improvement of the intangible "constructability". The VECP effected two significant changes. Increasing the spacing between bracing levels, which reduced the number of brace levels, eliminated at least two stages of the excavation and the associated bracing installation process. While the excavation volume remained the same, the operation was more efficient as larger equipment could be used in the hole. Reducing the line loads at the bracing levels served to lighten the bracing members, saving steel, and also the bracing connections, saving installation time and the costly labor associated with this work.

While not measurable, the redesign also enhanced the constructability of the overall support system. In the original design, brace levels were, in some instances, so close to structures of the final tunnel that, once the actual brace size was accounted for, there was very little clearance to work. The best example of this is the lowest contract brace level. Once these struts were sized, the contractor realized that there was virtually no room under these struts to finish the concrete base slabs of the tunnel. Therefore, eliminating this lower brace became essential to not only an efficient operation, but to a quality finished product. The lowest brace level in the VECP redesign allowed for the necessary equipment to pass beneath the struts.

The proposed VECP extended the cofferdam by about 90 feet towards the west, thus increasing the volume of tunnel to be cast-in-place. As shown in Fig. 6, the original design called for elaborate temporary horizontal truss system to support the fill atop the tunnel. Since the VECP proposed to increase the length of the cast-in-place tunnel, it was possible to provide a stable slope for the backfill without the need for the horizontal truss system. Since the contract documents did not allow penetrations through the tunnel, the struts within the tunnel envelope needed to be removed before casting the tunnel walls. The contract documents proposed vertical trusses, refer to Fig. 5, to support the wall after removal of the lower struts. The vertical truss was connected to the 18" working slab, and this connection needed to be designed to resist a reaction of 1100 kip. The VECP eleminiated those trusses, and designed the sheet piles to span the distance between the strut above the tunnel roof and the working slab. The original design of the east bulkhead wall showed AZ sheet piles supported by five levels of semi-circular W14 beams. The VECP proposed to use the already existing HZ piles, present in case history no. 1, to replace the semi-circular east bulkhead wall.

The contractor developed a conceptual design for the VECP, based primarily on using the finite element method of analysis to study the soil and structure simultaneously. This conceptual design took advantage of several opportunities not available to the original designers. The contractor was able to detail the actual, proposed construction staging, whereas the original designers had to make general assumptions about staging for their original analysis. Finally, the contractor was able to employ structural analysis models that augmented the contractor-proposed construction staging, thus taking additional advantage of soil-structure interaction behavior, again, an option not available to the original designers.

The modifications made to the original design are highlighted in Fig. 5. Taking advantage of the soil-structure interaction and the advanced finite element modeling allowed the rearrangement of the struts within the excavation section. Hence, a total of two temporary bracing levels were eliminated; namely struts at elevation 92 and 58. Additionally, the horizontal and vertical trusses were eliminated.

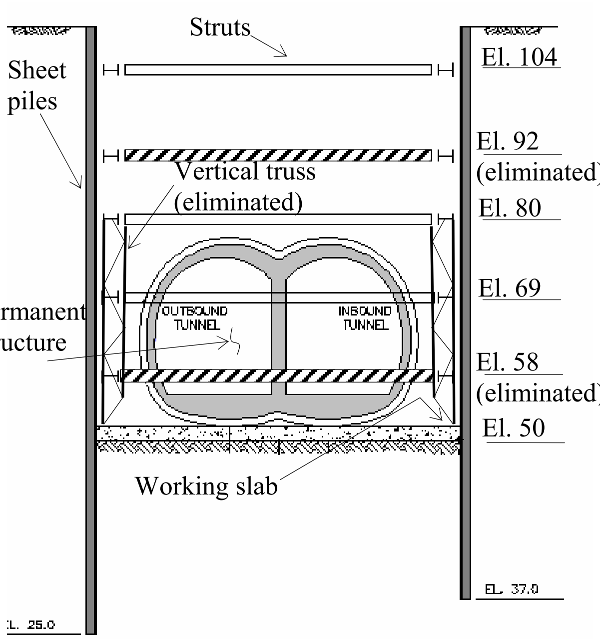

Fig. 5. Typical excavation cross section.

Impact on Adjacent Structures

As mentioned earlier, the cofferdam was extended by 90 ft, along the alignment of the Silver line tunnel, towards the historic buildings. This extension placed the historic buildings within the influence zone of the excavation for the cofferdam. Furthermore, the VECP proposed to reduce the number of bracing levels and use a flexible wall system (sheet piles) to support the excavation. Therefore, there were concerns that such VECP might result in deformations that exceed the contract limits set for protection of the abutting historic buildings. Excavation-induced movements might cause total settlement, differential settlement and angular distortion. Vertical settlement might be accompanied by horizontal strain in the adjacent structures. The relationship between the horizontal strain and angular distortion, as presented by Boscardin *et. al.* [1987], was used to assess the impact on adjacent historic structures.

Finite element analyses were performed to establish the SOE wall deflection and allowable differential settlement criteria based on tolerable amounts of angular distortion and horizontal strain that the building can withstand. Due to the orientation of the excavated area relative to the historic buildings, a three-dimensional finite element analysis was necessary. This analysis reflected the staged excavation as well as the sequence of strut removal. Actual strut and waler sizes and location were included in the model. Note that the SOE system was already designed based on the more conservative two-dimensional finite element analyses, as well be shown later in this paper.

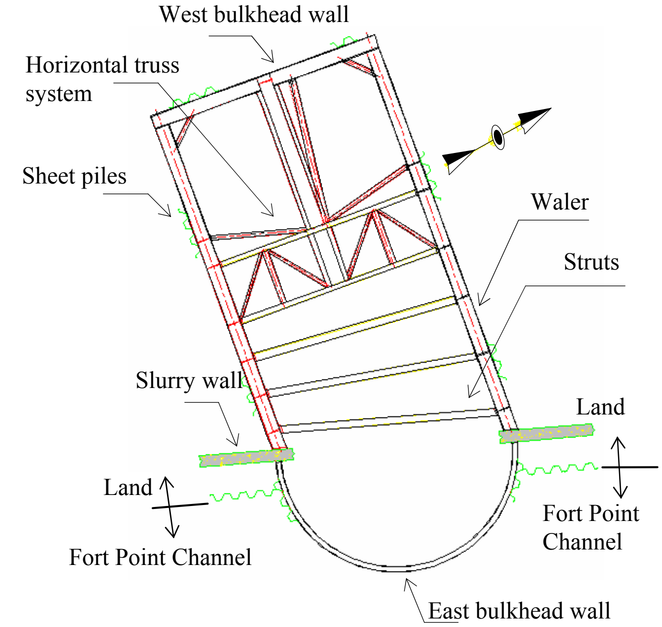

Fig. 6. Contract document SOE layout.

The results of the three-dimensional analysis indicate that the soil deformations at the historic buildings were within acceptable limits. This analysis was only used to investigate the excavation-induced soil deformations at the location of the historic buildings. The analysis of the SOE system was performed using two-dimensional finite element analyses.

Analysis of the SOE

The full structure of the tunnel section was modeled using the finite element software ANSYS. The geometry of the sections, including locations of various structural members such as walls, tunnel roof, and slab, was taken from the contract drawings. In addition, the soil profile for each section was determined from the geotechnical interpretative report.

In general, the width of the finite element mesh is at least equal to twice the depth of excavation. This dimension is sufficient to fully develop whatever active pressures are generated due to excavation. The total depth of the mesh is equal to the depth of the deepest wall plus approximately 20 feet. The vertical dimension of each two-dimensional soil element is 2 feet, and its width generally decreases as it approaches the walls. Such a mesh satisfies the refinement necessary to capture stress/strain concentrations near the walls, as well as staged excavation and construction. A sample finite element mesh is depicted in Fig. 7. The traffic decking typically served as the 1st level brace.

Sheet pile walls were modeled as two-dimensional elastic beam elements. Also, beam elements were also used to model the roof and the walls of the tunnel. The struts were modeled as truss elements in which no end moments are developed between the walls and the struts. The soil beneath the base slab provided an elastic support for the slab. The analytical models calculated the stiffness of this elastic support based on the given soil properties.

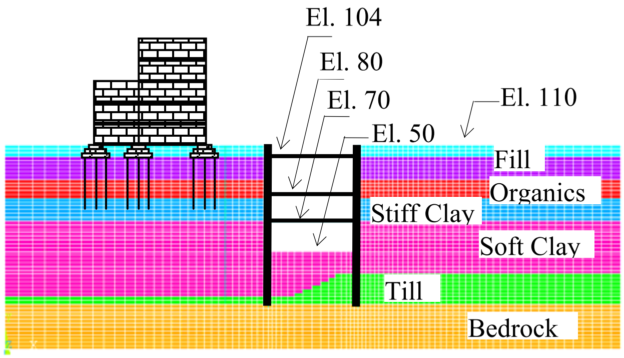

Fig. 7. Cofferdam finite element model.

Staged excavation analysis is performed by deactivating appropriate soil elements that were excavated. Staged construction analysis is performed by activating or reactivating the appropriate structural elements, which are installed, or backfilled soil. The locked-in stresses in the structural elements due to the different stages of excavation and construction are automatically considered in the nonlinear finite element model. The analysis is intended to simulate the excavation and construction of the tunnel in several load steps "stages". The first stage of analysis approximated the in-situ stresses, and the existing building loads were applied in the second stage. The tunnel excavation was started in the third stage.

Analysis Results

Sample of the analysis results are presented in Fig. 8, Fig. 9, and Table 1. Since the sheet piles were driven into the glacial till layer, small lateral displacement was observed at the toe of the wall, and the maximum lateral deflection of the wall at end of excavation was about 1.75" at El. 55. The removal of the lowest level of struts, after placing an 18" concrete working slab, resulted in an additional ¾" lateral deflection.

The lateral deflection curves of Fig. 8 indicate that no additional lateral deflection occurs at strut level as the excavation proceeds below the strut. Such results would yield stiffer struts during the design of the SOE elements, which is preferable since no strut preloading was specified for the majority of the struts.

A sample of the moment diagrams for the sheet piles is shown in Fig. 9. Note that negative moment indicates tension at interior face of the sheet piles, while positive moment indicates tension at soil side of the sheet piles. Due to the weakness of the clay layer below the bottom of excavation, larger moment was observed below the BOE and above the top of the glacial till. A bending moment of the same order was observed at the location of the second level struts after the removal of the last level of struts.

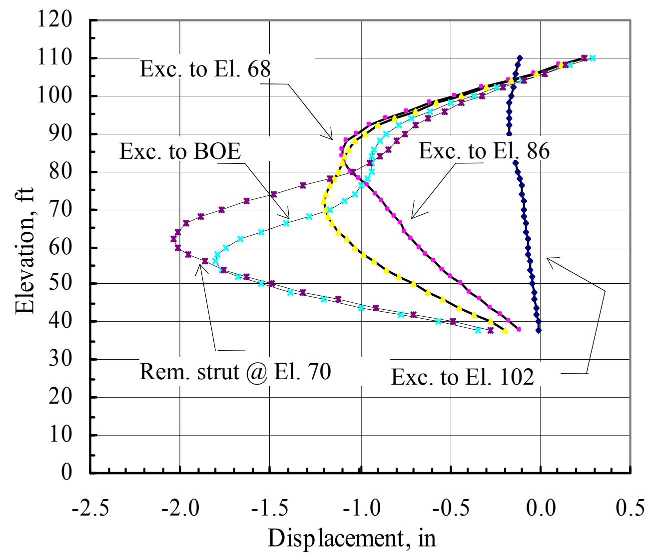

Fig. 8. Lateral deflection of the sheet piles.

Table 1. Strut Forces, kip/ft

Stage	Strut at El. 104	Strut at El. 80	Strut at El. 70
Exc. to El. 78	20.59	--	--
Exc. to El. 68	19.18	43.85	
Exc. to BOE	17.67	39.27	83.71
Rem. El. 70	15.30	84.70	--

During the excavation stage, the maximum axial force in the strut occurs as the excavation proceeds below this strut and just before installing the next level of braces, refer to Table 1. After removal of the last level of braces, the axial force in the second level of struts almost doubled.

Design of the SOE system

Generally, the support of excavation consisted of steel wale beams and cross-lot struts, refer to Fig. 10. Due to the large involvement of the contractor in other projects associated with excavation for the Central Artery/Third Harbor tunnel in Boston, considerable amount of struts and wale beams were available for use in the cofferdam.

Those struts were originally designed as built-up columns of two wide flange sections connected using batten plates. For use in the cofferdam project, a list of the available struts was generated showing the struts size and capacity for the specified unbraced length. Then the struts were selected from this list based on the axial forces calculated from the finite element analyses. Only one strut end detail needed to be modified, the other end detail was preserved and incorporated in the design of the SOE element of the cofferdam.

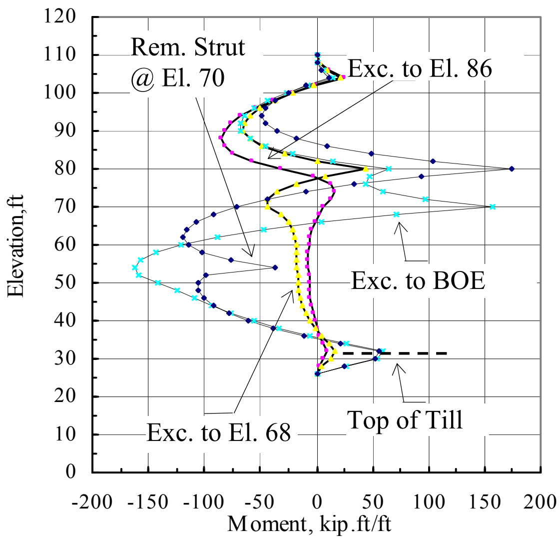

Fig. 9. Bending moment diagrams.

Few first level struts were designed as plate girders to support contractor traffic load as well as providing lateral support for the sheet piles.

Fig. 10. Excavation cross section.

Walers were designed to support the sheet pile walls, and they spanned between the cross-lot struts. Hence, walers were designed as beams supported by struts and loaded laterally by sheet piles. The lateral load of the sheet piles is obtained from the finite element analyses of the tunnel section. For economical and practical reasons it was desirable to use walers made of rolled steel beams without any web or flange stiffeners. The walers were sized to resist the bending moment and shear forces due to the load from the soldier piles. Furthermore, the lateral deflection of the walers was limited to L/1200, where L is the span between supporting points. This deflection limit was imposed to minimize the additional deflection of the sheet pile wall between the cross-lot struts. Similar to the struts, the wale beams were selected from a prepared list of available steel members that were used in other projects. In fact, some of the wale beams, used in this project, were originally designed as struts for a previous project.

The sheet piles were designed based on a combination of bending moment values, such as Fig. 9 above, obtained from the finite element analyses and axial forces due to the weight of the various SOE elements. Some sheet piles were designed to support contractor traffic load in addition to the lateral loads, and therefore, they were toed into bedrock and fitted with cover plates. Generally, the rest of the sheet piles were toed into the glacial till deposits in order to reduce the basal heave during excavation thus minimizing potential impacts on abutting historic buildings. The HZ sheet piles, installed previously for the construction of the BECo development slurry walls, refer to case history no. 1, were integrated in the cofferdam to form the east bulkhead wall, refer to Fig. 11.

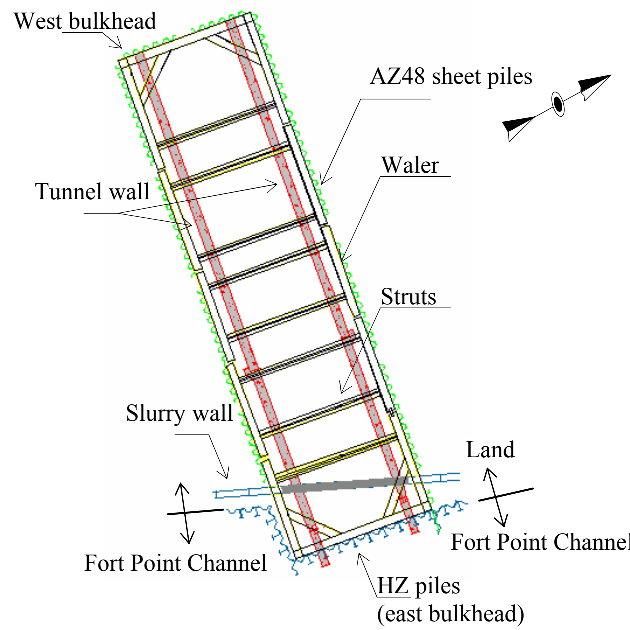

Fig. 11. Cofferdam SOE plan

The unbalanced force in the east-west direction and supporting the east and west bulkheads were probably among the most challenging issues encountered during the design of the SOE system. The east bulkhead wall was supporting hydrostatic pressure from the Fort Point Channel, and that pressure fluctuated due to tidal effect. The west bulkhead wall supported soil and ground water pressures. Both bulkheads transferred their lateral forces to the south and north sheet pile walls of the cofferdam. It was not desired to rely solely on the in-plane diaphragm action of the south and north sheet piles to resist the forces from the bulkhead walls. Therefore it was necessary to insure continuity in the load path to transfer the load between the east and west bulkhead walls. This was accomplished by providing a continuous concrete grout behind the wale beams.

This concrete grout extended the full length of the cofferdam and was designed as a column without reinforcement. Shear studs were added to the wale beams so that the longitudinal forces are transferred to the concrete grout. In fact, this concrete grout served a dual purpose; it transferred the longitudinal forces between the bulkhead walls and insured full contact between the sheet pile walls and the wale beams.

Implementation of the SOE System

While the development of the final SOE design proved to be rather complicated and evolving task, from both technical and contractual perspectives, the actual installation of the system was relatively straightforward. Although the design was much more efficient, field work remained relatively unchanged, with the exception of the reduced number of struts and elimination of the steel trusses. The only discussions regarding the installation would concern the adaptability of the analyses to responding to changed field conditions. With seemingly great ease, the finite element models could be adapted to investigate alternate sequences of work, different levels of bracing or changed soil conditions when any of these situations was encountered. Within a matter of a few days, a reanalysis would be ready to present to the owner, detailing the contractor's proposed method for handling an unforeseen condition, whether there be an unexpected utility or other conflict.

It is worth mentioning that during the removal of the struts, it was desired to accelerate the construction of the tunnel within the east half of the cofferdam. This would lead to interrupting the load transfer mechanism between the east and west bulkheads. Therefore, it was necessary to revisit the issue one more time and try to transfer the longitudinal forces from the west bulkhead wall to the soil along the north and south walls. This task was accomplished and it was determined that the SOE system within the first 100 ft from the west bulkhead would be needed to support the west bulkhead loads. This gave the contractor the flexibility to remove the eastern struts without worrying about the unbalanced forces in the east-west direction.

Negotiation with the Owner

A conceptual submittal was prepared and presented to the owner's engineer summarizing the proposed VECP. Following this, the parties entered into a series of negotiations to agree on all the various parameters of the model, its analysis and the resulting design. The initial conceptual model presented an aggressive plan, increasing the strut level spacing and reducing the line loads to nearly the limits of acceptability within the analyses. While some analyses showed that the conceptual submittal did not violate any original contract criteria, they significantly reduced the SOE that would be installed and, therefore, it became necessary to mediate a "happy medium" that still provided savings under the premise of the VECP, while providing a product with quality comparable to the original design. For a support of excavation system, the measure of this quality is primarily the stiffness of the system, that is, the bracing intervals and sizes, the very target of the VECP redesign.

Another factor that influenced these negotiations was the owner's comfort level with the new analysis methods. While the methods of analysis were recognized as accurate and sophisticated, they had not been in use long enough to be well validated by empirical data from excavations on completed projects. This factor had to be weighed by the owners against the proposed time and cost savings promised by the VECP.

The various technical parameters that became the subject of negotiations during the various revisions of the initial conceptual submittal included the soil models, the strut spacing, both vertically and horizontally, the allowable strut stress and various issues regarding the detailing of the SOE system. Each of these is touched on below.

For the finite element models created, perhaps the single most critical input parameter is the constitutive model of the soil that is used. As no loads, besides the hydrostatic, are applied in the staged analyses, the soil model itself generates both the loads and reactions. The discussions surrounding the selection of these parameters involved not only the owners, but included their geotechnical consultant as well. The crux of the matter was that the soils in the area had not heretofore been modeled in this manner and the soil parameters prescribed in the original contract were not readily translated into this constitutive model. Ultimately, values were agreed upon that gave the owners a comfort level for safe and prudent design, while still taking advantage of the inherent strength of the soil, usually not recognized in conventional analyses, to enable the finite element models to effect a savings over these conventional models.

The strut spacing was perhaps the most ardently debated topic between the various parties, as the contractor took the natural position that any and all bracing that could be eliminated should be. This presented the owner with conceptual designs that sometimes eliminated up to two levels of bracing, and using flexible SOE walls "sheet piles". This aggressive design raised the question of comparable quality. Despite the refinements that an analytic method can present, eliminating over half the actual bracing material seemed to present a system of lesser quality, regardless of the fact that movement predictions were still within contract allowable limits. The owner was faced with defining a compromise that would preserve the value of the change while still providing a system that could be justified to abutters with structures impacted by the work.

Finally, during the design portion of the VECP, that is the sizing and detailing of the bracing struts themselves, there were various criteria that were scrutinized by both the owners for potential savings and by the contractor to assess the impacts of these proposed changes. The detailing of the walers was also debated, from issues of stability to details of support at the sheet piles in the walls. In addition, the final waler design allowed for cantilevered walers, which eliminated the field fit up difficulties of meeting two walers with a single strut. Systems of jacking and monitoring preload in the struts were also scrutinized by all parties. While the above items were not necessarily a result of the redesign focused on here, they are further examples of the carefully crafted end product that resulted from the ongoing dialogue between the designers, the owners and the contractor

Actual Behavior of the SOE System

Because of the overriding concern for the integrity of the surrounding structures during the excavation, a comprehensive and complete system of monitoring has been installed adjacent to all excavation work. This monitoring system includes horizontal and vertical monitoring points on adjacent structures and utilities, in addition to an array of subgrade geotechnical instruments. Inclinometers measure soil movements, while observations wells and piezometers measure groundwater levels and heave gauges monitor soil movements. Through the collection and synthesis of data from these instruments, the MBTA has been able to closely monitor the impacts of the excavations at all stages of this work.

The analytical models tend to overestimate the wall deflection. This could be attributed to the conservative assessment of the physical properties of the soil and the walls. Furthermore, the ground water table level was determined from the design criteria of the project. In realty, the actual water table level might have been lower than assumed by analysis. Since protection of the historic building is one of the major tasks of the project, engineers tend to assign conservative parameters for the finite element analyses, which would eventually yield a conservative assessment of the lateral deflection of the SOE walls was anticipated. Note that the stiffness of the SOE system, rather than the strength, has significant impact on the excavation-induced movements in the soil mass.

CONCLUSIONS

Three case histories, associated with the construction of MBTA's Phase II of the silver line subway tunnel in Boston, were presented in this paper. State-of-the-art finite element analyses were performed to design the various earth retaining structures presented in this paper. Those analyses not only resulted in more efficient and robust earth retaining structures but they also helped in studying excavation-induced soil movements and the their impact on abutting historic buildings.

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