Design and Construction of a Support of Excavation System for a Deep Cut-And-Cover Tunnel in Downtown Boston

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ABSTRACT

This paper presents the analysis, design, implementation and construction of a Value Engineering Cost Proposal (VECP) for the support of excavation system for parts of the underground Central Artery Tunnel in downtown Boston. The excavation varies between about 130 ft and 240 ft in width and between 60 ft and 100 ft in depth. The typical structure of the tunnel consists of soldier pile tremie concrete (SPTC) walls, roof girders with a cast-in place (CIP) concrete slab and a CIP invert slab. The SPTC walls, constructed using the bentonite slurry technique, act as the temporary earth-support structure as well as the permanent walls of the tunnel. The walls are temporarily braced during the excavation prior to the installation of the roof girders and the invert slabs. This support of excavation (SOE) scheme was the target of the VECP.

The VECP was conceived to save both time and money over the original scheme presented in the contract documents, which was based on a beam on elastic foundation method of analysis to design the walls and determine line loads for bracing design. The crucial element of the VECP was to use a finite element analysis method to reanalyze the walls with fewer bracing levels. This analysis yielded lower line loads compared to the original design. The paper traces the steps leading to the implementation of the VECP, including the proposal and preliminary design, the cost and schedule negotiations with the owner, their representatives and the designer of record, the analysis and design submittals and, finally, the construction and performance of the system.

INTRODUCTION

The Central Artery/Third Harbor Tunnel (CA/T) in Boston, a multi-billion-dollar transportation improvement project, includes the replacement of the existing elevated I-93 expressway by a tunnel in downtown Boston and the construction of a new tunnel crossing Boston Harbor to extend the adjacent I-90. For the downtown I-93 section, the typical tunnel structure consists of soldier pile tremie concrete (SPTC) walls, roof girders with a cast-in place (CIP) concrete slab and a CIP invert slab. The SPTC walls, constructed using the slurry method, act as the temporary earth-support structure as well as the permanent walls of the tunnel. The steel plate girders of the roof system span between the SPTC piles and CIP walls. The invert slabs house the ventilation ducts and are rigidly connected to the SPTC walls. Cross-lot struts, and earth berm are used to temporarily brace the SPTC walls during the excavation.

Generally, contractors are encouraged to develop and submit Value Engineering Cost Proposals (VECP) to the Owners. In a typical VECP a contractor will propose a change to the base contract that will save usually money and often time over the original design. If the Owner approves this proposal, both the Owner and the contractor share the savings, which are agreed upon during negotiations of the terms of the VECP. This provides an incentive for all parties without compromising the integrity of the final product.

The VECP considered here was conceived to save both time and money over the original scheme presented in the contract documents. This original design was based on a conventional beam on elastic foundation method of analysis to design the walls and determine line loads for bracing design. The contract documents included the final wall design and the bracing line loads, with a requirement for the contractor to design the SOE based on these values. The crux of the VECP 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 lower stresses in the walls with lower line loads at fewer bracing levels, compared to the original design.

This analysis technique resulted in the design of a lighter bracing system than would have originally been required. Money was saved in the material and labor costs of the bracing and time was saved by eliminating one or two levels of bracing. In addition, a “top down” sequence of construction was adopted in many locations where the geometry allowed, wherein the roof girders were installed on the way “down” instead of the way “out” of the excavation. The girders not only substituted for a temporary bracing level, but were easier to install at this stage at a temporary subgrade rather than later from the road surface through intermediate bracing levels.

Fig. 1. SPTC wall braced by struts and walers

The engineering work on the VECP consisted of two very distinct phases. First was the analysis, which was performed on the existing wall design using modified construction stages. Although the wall was analyzed and stresses found to be reduced, no changes were made to this design. Furthermore, the scope of the VECP was limited to the temporary stages without modifications to the final structure designed in the original contract. Therefore, the changes that resulted from the analyses were in the sizes of the bracing struts. While the design of these members realized the savings in the VECP, the design of the structures was fairly conventional, with some minor innovations that will be touched upon later.

The behavior of the underground tunnel and the surrounding structures was analyzed using state-of-the-art finite element models with soil-structure interaction. Nonlinear soil models were incorporated in the finite element analyses, which modeled the staged excavation and strut installation, as well as the sequence of strut removal during the construction of the tunnel. The results of the finite element analyses were used to design an efficient temporary SOE system that consists of cross-lot struts with wale beams along the SPTC walls. The number and elevations of the bracing levels were selected such that the permanent structure stresses and deflections and the soil deformations were within the contract required limits. The potential for conflict between the SOE system and the permanent structure also influenced the selection of the bracing locations. The finite element models were updated during the construction of the tunnel in order to reflect changes in the construction sequence. Data from the instruments that monitored the SOE system and adjacent structures during the excavation and construction of the tunnel were compared with the results of the finite element analyses. These comparisons illustrate the realistic results produced by the finite element models, which are still sufficiently conservative to protect new and existing structures.

BACKGROUND

The CA/T Project was under the direction of the Massachusetts Highway Department (MHD) and is currently overseen by the Massachusetts Turnpike Authority (MTA). MHD hired a Management Consultant (MC) to oversee both the design and construction of the project. The MC prepared preliminary designs and managed final design and construction. For all portions of the work, consulting design firms (Section Design Consultants, SDCs) have prepared the final designs. These have been conventionally bid and awarded to the low bidder.

For the contract between North and Chardon Streets, the general contractor (GC) proposed the VECP detailed here. The GC enlisted the assistance of a consultant to perform the analyses and design for the proposed VECP, as the General Contractor's Consultant (GCC). The GCC here had already had similar VECPs approved on other CA/T contracts, and the process appeared straightforward.

Probably the most challenging aspect of the project, for the MTA and the MC as well as the SDCs and GCs, was its location, directly through the heart of downtown Boston. The need to maintain the city streets and buildings in a fully functional state, with minimal impacts, throughout the course of the project has been demanding in many respects. Specifically as it relates to this VECP, limiting impacts to adjacent structures from the deep excavations has been tantamount. The MC and Owner established limiting criteria which would preserve and protect all adjacent structures. These criteria were applied to the VECP work as well as the original design and were the focus of many of both the contractual and technical negotiations between the various parties. Technically, these issues can be summarized as follows:

- The original contracts included a detailed baseline analysis for performance of support of excavation. This was presented in the contract documents in terms of strutting locations and forces, and specified limits for various types of movements of walls, existing structures, and impacts to the groundwater table.
- The structures along the tunnel alignment were considered more susceptible to the adverse impacts of deep excavation and tunnel construction, and were subject to more stringent specification limits for any impacts.
- The contractor had to demonstrate to the Owner that any revision to the contract document design of support of excavation would still satisfy the baseline requirements for the SOE performance.

This last item was perhaps the most difficult part to achieve. The requirement overlapped the border between a "design" element, and contractor's means and methods. It was necessary for this sensitive construction that the design of the temporary SOE address not only impacts to the final structure, but control of impacts during construction. This area is traditionally part of the contractor's means and methods. Ensuring that the contract requirements were understood and satisfied required not just technical analysis, but detailed coordination and an education effort for all parties involved.

CONSTRUCTION ISSUES

The contractor's inspirations for initiating this change were threefold: cost savings, time savings and improvement of the intangible "constructability". The SOE redesign VECP effected two significant changes. Increasing the spacing between bracing levels, which reduced the number of brace levels, eliminated at least one stage 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.

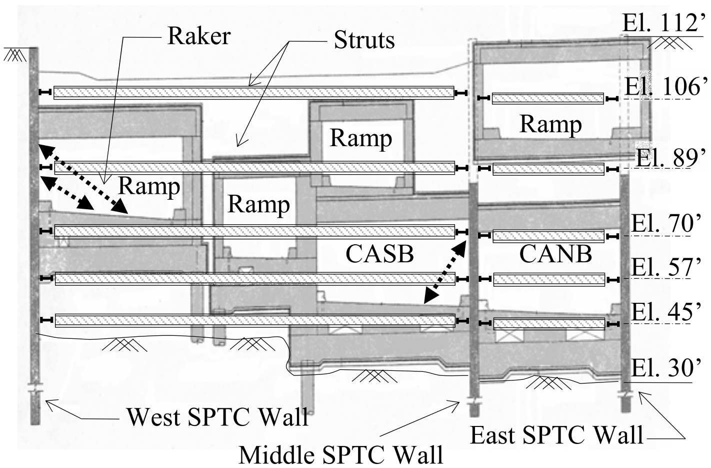

Fig. 2. SOE system proposed in the contract documents.

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, moving this lower brace level up 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. Moreover, the VECP simplified the construction sequence of the tunnel by eliminating the rakers that were proposed by the contract documents. A typical tunnel cross section with contract document SOE is shown in Fig. 2.

One concern regarding the relocation of the bracing levels was that the responsibility of the bracing levels was now entirely borne by the GC. Any impacts to excavation and subsequent tunnel construction was coordinated between the GCC and the GC. For this reason each bracing level and strut location was reviewed for potential impacts during construction.

The relocation of the bracing levels also eliminated a number of wall penetrations that were required under the original scheme. This provided a better end product for the owner and benefits to the construction staging.

In addition, at a later stage, the VECP incorporated the permanent roof girders of the northbound tunnel into the bracing scheme by having them installed during the excavation. Obviously, this was only possible in tunnel section where the roof girders span between the SPTC walls. In these locations construction was simplified by placing the roof girders typically after the first or first two bracing levels were installed. Girders were brought into the excavation using a horizontal access on large skids operating on the temporary subgrade of the incomplete excavation and were installed in their final locations. This system eliminated the more difficult operation of installing the girders from the completed base slab. In this operation, the girders would most likely need to have been lowered into position, and it would have been tricky maneuvering them past the bracing walers, typically 36" deep, along both walls. Given the horizontal spacing of the braces, turning the girders diagonally in plan was limited. In addition, bracing below the girders and above the completed base slabs would have precluded the use of any handling equipment operating on this slab.

PROPOSED SOE SYSTEM

As mentioned, the GCC had already developed, for other contracts, analysis and design methods to effect the desired changes. Working with GC to best understand the construction staging, the GCC 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 SDC. First, the GC was able to detail the actual, proposed construction staging, whereas the SDC had to make general assumptions about staging for their original analysis. Finally, the GCC 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 SDC. Many areas of the project had relatively strong and stiff soils, enhancing the effectiveness of this modeling tool.

The modifications made to the original design are highlighted in Fig. 3. At a later stage, the third level struts were replaced by the Central Artery Northbound (CANB) roof girders during the top-down construction process. Moreover, taking advantage of the soil-structure interaction and the advanced finite element modeling allowed the rearrangement of the struts below the roof girders. Furthermore, virgin earth berm was left next to west slurry wall, which allowed the elimination of the last two levels of struts between the west and middle slurry walls. The construction sequence was modified to eliminate the need for any rakers during the strut removal.

The GCC prepared a conceptual submittal, which provided a complete and detailed analysis and design for three typical sections of the tunnel. The intention of this submittal was to allow the reviewers at the MC and the SDC to see the entire scope of the analysis and design, comment and correct as necessary, in advance of the preparation of similar work on the ten odd design sections that needed redesign within the limits of the contract. This process worked fairly well, particularly to concur on the method of analysis, establish soil parameter values and evaluation criteria, including wall stresses, roof girder loads and, most importantly, both horizontal and vertical wall movements. Of course, in the various sections during the final design development stages, special conditions were encountered that needed special review and often modification.

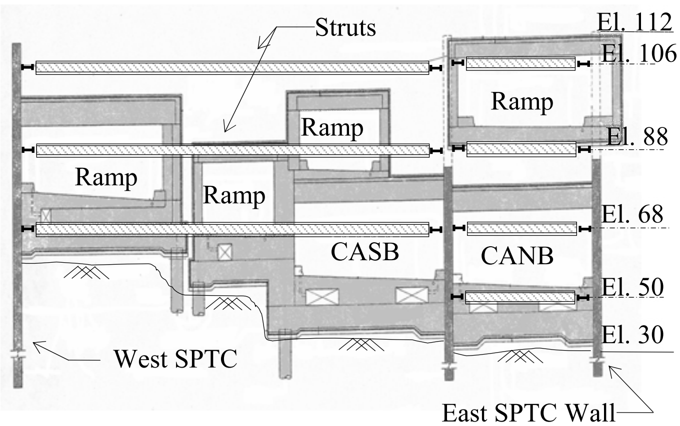

Fig. 3. SOE system proposed in the VECP.

ANALYSES METHODOLOGY

Classical Analysis

This approach uses the Rankine Theory of earth pressure for the analysis and design of braced excavations. The lateral pressure, which may include earth, surcharge, and hydrostatic loads, is applied on the active side of the wall, and a group 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. 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 are implemented by 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 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.

For the VECP, the full structure of the tunnel section was modeled using the finite element software ANSYS. The geometry of the section, 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. Several finite element models were constructed for different sections along the tunnel alignment. The finite element models accounted for the variation of the soil profile, the geometry of the tunnel, and the location of the temporary bracing levels. A sample finite element mesh is depicted in Fig. 4. The traffic decking plate-girders typically served as the 1st level braces.

SPTC walls were modeled as two-dimensional elastic beam elements, and their stiffness was derived based on the cracked transformed section. Beam elements were also used to model the roof system of the tunnel. The struts were modeled as truss elements in which no end moments were developed between the walls and the struts.

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.

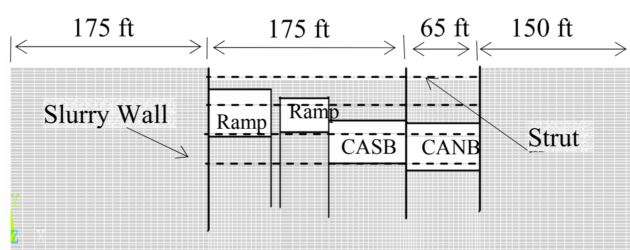

Fig. 4. Typical finite element model.

Construction surcharge was applied at ground surface on either side of the excavation. The hydrostatic pressure is assumed to be uncoupled from the mechanical behavior of the soil. This uncoupling is acceptable for temporary short-term excavations because the time-dependent behavior, such as consolidation and creep, is not significant. The prescribed hydrostatic pressure was applied on the wall, while the soil pressure was generated automatically by the finite element code using the appropriate unit weight of the soil. Inside the tunnel, the water table was assumed to be at grade level. On the other hand, the water table was maintained outside the tunnel at its design level.

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.

ANALYSES RESULTS

Sample results from the finite element analyses are shown in Fig. 5 through Fig. 7. The results are presented at several excavation and construction stages. Negative moment is observed at the location of the bracing struts; refer to Fig. 5. In this project, the slurry walls are toed in the bedrock, hence, large negative moments are observed in the slurry wall at top of the bedrock. Also due to this fixity at the toe of the wall, the positive moment is not relieved during the excavation process. Generally, some relieve in the positive moment might occur if the toe of the wall experienced some lateral deflection towards the excavated site.

The maximum lateral deflection at the toe of the wall was in the order of 0.1", compared to about 1.75" above the bottom of excavation, refer to Fig. 6. Note that this figure shows the results for the excavation of the northbound tunnel. Similar curves were generated for all stages within a particular analysis.

The use of the finite element model gave the analyst the opportunity to predict the free-field soil deformations within different locations in the vicinity of the excavated area. All historic buildings in the vicinity of the excavation zone were evaluated for potential damage as outlined by Boscardin and Cording (1989). In particular, two major parameters, namely the angular distortion and the horizontal strain, were used in the evaluation process. The goal was to maintain the estimated damage level below "slight". With this goal in mind, the locations of the struts were revised several times during the analysis stages.

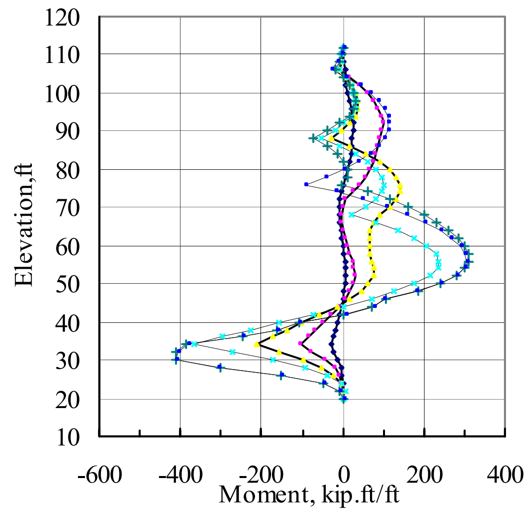

Fig. 5. Bending moment diagrams.

An example of the free-filed vertical displacement of the soil due to the excavation is shown in Fig. 7. The maximum value of the excavation-induced settlement occurred at about 20 ft from the face of the slurry wall. Utilities located next to the slurry walls were designed to tolerate such settlement. Note that at a distance from the slurry wall equal to about the depth of excavation the free field settlement was about 0.3".

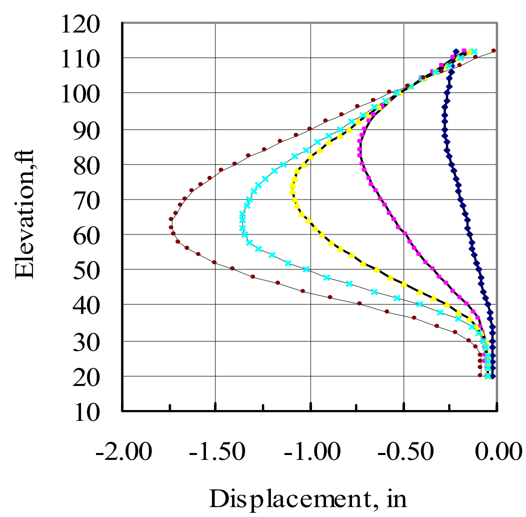

Fig. 6. Lateral deflection of the slurry wall.

One of the main objectives of the VECP was to produce reduced lateral loads for the design of the SOE system. A comparison between the design loads provided in the contract documents and those produced by the finite element models is shown in Table 1. The soil-structure interaction models not only enabled the GC to eliminate some strut levels, but also allowed the reduction of the lateral design forces for the struts. The VECP analyses resulted in slightly higher axial forces in the first level braces when compared with the contract document forces. This higher axial force was primarily due to removal of braces below this first level. As mentioned earlier, the first level braces served as traffic girders, which were typically 5 to 6 ft deep plate girders designed to support traffic and construction equipment at street level, and the axial force was not a major factor that controlled the design. In fact, the end connection of those plate girders was detailed such that the axial force would be beneficial to the behavior of the plate girder.

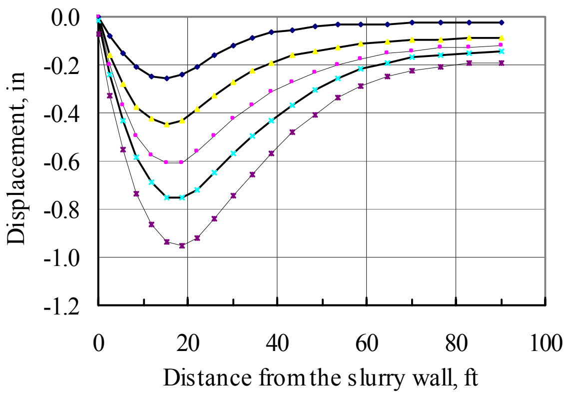

Fig. 7. Vertical displacement next to the slurry wall.

Table 1. Strut axial forces, kip/ft

	CASB tunnel strut level					CANB tunnel strut level				
	1	2	3	4	5	1	2	3	4	5
Contract	20	80	73	59	30	20	80	73	59	30
VECP	28	62	46	---	---	29	42	52	27	---

As part of the VECP agreement, all of the analyses needed to demonstrate that the proposed changes had no adverse effect on the final structure. The finite element analyses allowed for introducing these structural elements into the analyses and for finding the temporary stresses imposed upon them during the various construction stages. Of particular interest were the northbound tunnel roof girders, as they were being used as temporary struts during the excavation. The GCC was responsible for calculating the loads on these members and, by manipulating the excavation and construction staging as necessary, insuring that no undue loads were placed on them. The finite element analyses also had to demonstrate that the stresses in the walls did not exceed allowable design stresses. In fact, it was demonstrated that the stresses were actually lower than those originally predicted. Other element used to temporarily brace the SPTC walls, such as earth berm and lean concrete fills, were also investigated. The target of the investigation was to determine the geometry and the required strength parameters for those elements. Finally, although not a major concern, the analyses were able to demonstrate that the base slabs also were not overstressed in the proposed redesign.

DESIGN OF THE SOE SYSTEM

Generally, the support of excavation consisted of steel wale beams and cross-lot struts, refer to Fig. 1. Two wide flange sections were connected using batten plates to form the built-up cross-lot struts. The struts were designed as conventional built-up beam-column pinned at both ends. The design of the struts accounted for thermal and self-weight stresses. Due to close proximity of historic buildings, the axial stresses in the struts were limited to 12 ksi. This axial stress limit was relaxed to 15 ksi for struts located farther from existing buildings. However, the majority of the long struts (longer than 100’) were not affected by the 12 ksi stress limit since the design of these struts is generally controlled by the slenderness of the strut.

Some struts were in excess of 140’ in length, economical and practical design of such struts required providing lateral and vertical supports at intermediate points. To accomplish this goal, two options were presented for the design of struts longer than 110’. The first option required the use of pin piles to limit the maximum unbraced length of the strut to about 100’. The pin piles provide lateral support as well as vertical support to reduce the effect of the strut self-weight. The second option proposed to support the struts at two additional points within the strut span. Pin pile frames were used for this purpose. The pin pile frames were designed as moment frames to provide adequate lateral support for the struts. All struts were pre-loaded, using hydraulic jacks, to 50% of their design force. The pre-loading process was intended to insure a tight fit between the SOE elements and the SPTC wall, and to reduce the elastic deformation of the struts as excavation progressed below these struts. This particular project was located in the heart of downtown Boston where a maze of utilities was buried in the ground, and right below the existing Central Artery Viaduct. The underpinning of the viaduct footings and the support of the existing utilities created a “forest” of steel which complicated the design of the SOE system. The layout of the struts and walers was dictated by the underpinning elements, furthermore, the layout of the SOE elements intended to eliminate or minimize interferences with the permanent tunnel structure. Those factors resulted in struts spaced horizontally as far as 30 ft apart. The tunnel cross section geometry varied rapidly due to the presence of many ramp structures that are intended to circulate the traffic. Therefore, tunnel cross sections were generated electronically at 20 ft interval along the length of the 1200 ft tunnel. Each section showed the exact tunnel geometry and the exact location of the SOE elements. Those sections helped in ensuring that all interference issue with permanent structure are addressed ahead of the actual installation of the SOE elements.

Walers were designed to support the SPTC walls, and they spanned between the cross-lot struts. Hence, walers were designed as beams supported by struts and loaded laterally by soldier piles. The lateral load of the soldier piles was 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 SPTC wall between the cross-lot struts.

TECHNICAL NEGOTIATION

Once the conceptual submittal was prepared and presented, all parties, the MC, the SDC, the GC and GCC, 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. However, as noted above, the MC was charged with upholding certain limits to preserve the integrity of the surrounding structures. While the analyses did not violate any original contract criteria, they significantly reduced the SOE that would be installed. It was clearly evident that the GC could design and install an SOE that is adequate to safely support the SPTC walls during excavation. While the owners took the position that the stiffness of the SOE system should abide to a certain criteria.

Another factor that influenced these negotiations was the MC's comfort level with the new analysis methods and the extent to which it could be justified to their client, the MTA, who ultimately had a responsibility to the public and private owners of the surrounding structures. 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 MC against the proposed time and cost savings promised by the VECP. The SDC was charged with performing concurrent analyses using FLAC, a geotechnical finite difference software. The results of the SDC analyses were comparable with finite element analyses performed by the GCC.

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. The position of the MC and GC regarding these issues is outlined below. It should be mentioned here that the GC was allowed to pursue the VECP given that the contract specified movement thresholds, which limited the impact on adjacent structures, are satisfied by the proposed VECP.

The constitutive model of the soil, used in the finite element models, is the single most critical input parameter. Since 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 MC and the GCC, but included the SDC and their geotechnical consultant as well. The most important issue was that the soils in the Boston area had not been modeled in this manner and the soil parameters prescribed in the original contract were not readily translated into the finite element constitutive model. Values of soil parameters were agreed upon that gave the MC and their client 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 produce savings over these conventional models.

The strut spacing was perhaps the most debated topic between the various parties, as the GC took the natural position that any bracing that could be eliminated should be. This presented the MC with conceptual designs that sometimes eliminated up to three levels of bracing. 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 MC was faced with defining a compromise that would preserve the value of the change while still providing a system that could be justified to owners with structures impacted by the excavation process.

The MC allowed the substitution of the northbound roof girders, where applicable, for one level of struts, given that they were installed in a top-down sequence, and the elimination of one other level of bracing. Virgin soil berm and lean mix concrete fills were allowed as a brace at the southbound side. A maximum vertical spacing limit was also given at 20 feet. These criteria still enabled time and cost savings in the final installation and did, indeed, create a system that performed within allowable limits. While it may be debated that further bracing could have been eliminated, the risk that working with a relatively unproven analytic method did not warrant a more aggressive system, particularly in an area as sensitive as the heart of downtown Boston.

The horizontal spacing of the struts, prescribed in the contract at 18 feet maximum, was also investigated by the GC and GCC to determine if this could be increased as well. The MC was not particularly receptive to this proposal, as the original limits were imposed to minimize wall movement, and the consequent surface settlement, between struts. With deflection criteria of L/1200 for the walers, this did not become a likely place for economizing the SOE system: the walers would have been excessively large. However, as mentioned above, the layout of the struts was dictated by the presence of viaduct underpinning steel elements and utility corridors. This resulted in up to 25' to 30' horizontal space between some struts.

Finally, contract design parameters mandated that strut stresses should not exceed 12 ksi given concerns for elastic shortening of the struts during loading in the staged excavation, and the consequent potential for wall movement. However, strut stresses were allowed to increase to 15 ksi, given that wall movement predictions still fell within allowable limits. This increase allowance was limited to struts within specific areas farther from the abutting structures. Although in some instances the GC and GCC maintained that higher stresses would not result in additional movement, in the majority of the cases, the unbraced lengths of the struts governed the allowable stress, often with a value less than 12 ksi. Any change in the criteria by the MC would have resulted in negligible savings, again, not enough to justify the additional risk.

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 investigated by both the GCC for potential savings and by the MC to assess the impacts of these proposed changes. First in sizing and detailing the longest of the struts, some up to 140 feet long, the GCC proposed different methods to reduce the unbraced span of the sections. While other methods were suggested, the method ultimately employed in the field consisted of moment frames that were formed by pin piles and cross beams. In addition to supporting the struts, those frames supported the traffic bridge at street level. The detailing of the walers was also debated, from issues of stability to details of support at the soldier piles in the walls.

INSTALLATION OF THE SOE SYSTEM

The development of the final SOE design, as detailed above, proved to be rather complicated and evolving task, from both technical and contractual perspectives. Although the design was much more efficient, field work remained relatively unchanged, with the exception of the reduced number of struts. However, it is worth discussing the adaptability of the analyses to responding to changed field conditions. The original VECP called for the concurrent excavation and construction of the southbound and northbound tunnels using the open-cut excavation technique. However, the VECP was modified to allow the accelerated construction of the northbound tunnel ahead of the southbound tunnel. Under this modified scheme, the northbound tunnel was proposed to be constructed using the top-down technique while the southbound remained to be excavated using an open-cut excavation method. With seemingly great ease, the finite element models were 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 GC's proposed method for handling an unforeseen condition, whether there be an unexpected utility or other conflict.

PREDICTED versus ACTUAL BEHAVIOR

Because of the concern for the integrity of the surrounding structures during the CA/T 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 a network of subgrade geotechnical instruments. Inclinometers measure wall movements, while observations wells and piezometers measure groundwater levels and heave gauges monitor soil movements. The SOE struts themselves have been instrumented with strain gages to monitor the changes in the strut forces through the stages of excavation. Through the collection of data from these instruments, the MC has been able to closely monitor the impacts of the excavations at all stages of this work.

The monitoring program has also provided an opportunity to evaluate the accuracy and reliability of wall analysis and design methods. Here, for a typical section in the area of the VECP, the predicted and the measured behavior of the wall are compared in Fig. 8. Comparisons are made at the final stage of excavation and are based on several different measurements. The actual measured movements are still below the predictions and well within the allowable threshold values established to preserve the abutting structures.

The curves presented in Fig. 8 indicate that the analytical behavior of the wall has a trend similar to that of the actual behavior. However, 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. The stiffness of the walls was calculated based on the properties of the steel soldier piles alone, while the actual SPTC wall might posses some composite behavior, hence, stiffer actual walls would deflect less. 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 in downtown Boston is one of the major tasks of the CA/T 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.

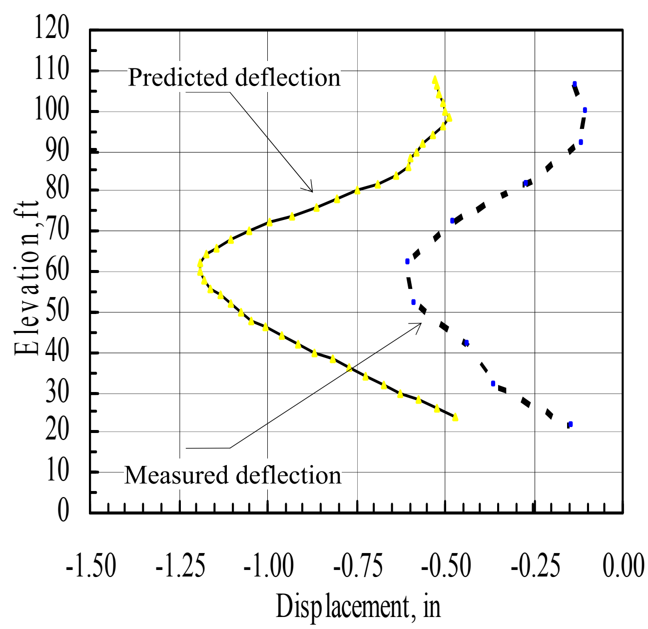

Fig. 8. Predicted vs. measured lateral deflection.

In summary, it appears as if the analytic models very closely and somewhat conservatively predicted wall movements and surface settlements. This is an encouraging result, as the models produced more economical and practical bracing designs than conventional analysis methods, without any compromise for the safety and integrity of the surrounding structures.

SUMMARY

The VECP for the analysis and design of the SOE system was a mutual effort between the concerned parties to satisfy the limits imposed by the design specifications and to address the contractor's interests for improving the constructability of the tunnel structure to support means and methods. The finite element analyses was proven to be a crucial tool in the evaluation of not only the behavior and design of the tunnel structure during excavation and construction, but also in the evaluation of its impact on abutting buildings, many of which are historic structures. By working together to develop a design incorporating these techniques, the owner and contractor were able to realize shared time and cost savings in the installation of the temporary support of excavation system.

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