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### ABSTRACT

The innovative analysis, design, and construction of the temporary support of excavation (SOE) system for an underground garage will be presented in this paper. The site of the project is blanketed with a 10 to 18 feet thick layer of fill material, underlain with about 5 feet of soft organic deposits. The main soil deposit at the site consists of 65 to 90 feet deep marine clay, known as the “Boston Blue Clay”. The upper 10 to 12 feet of this clay is weathered and hardened to form a stiff crust that softens with depth. The majority of the excavation within the project site removed the stiff clay crust to expose the soft clay layer. In order to excavate to the required depth of about 44 feet, the contractor had to address a major challenge of controlling the basal heave as well as the lateral support of the excavation.

Reinforced concrete slurry walls were installed along the perimeter of the underground garage to serve as structural wall and water cut-off for the parking garage. The slurry wall was toed in the soft clay layer at about 12 to 20 feet below the bottom of excavation.

Finite element models that accounted for soil non-linearity were used to analyze the staged excavation and construction of the garage structure. Based on the finite element analyses, two temporary bracing levels were used to provide lateral support for the slurry walls. Because of the geometry of the underground garage and the variation of the bottom of excavation, the design and installation of the temporary bracing system was a challenging task. A close correlation between the predicted and the measured lateral deflection of the slurry wall was observed.

### INTRODUCTION

The Manulife building consists of a fourteen-story office structure, with two and one half levels of below grade parking. The building is located in the South Boston area, adjacent to the Central Artery/Third Harbor (CA/T) Ted Williams Tunnel. The construction of the underground garage requires a 44 feet deep excavation that extends well in the Boston Blue Clay layer. The underground garage has a unique geometry that resembles a trapezoid with a curved side that measures about 281 feet in length, and the rest of the sides are straight with a minimum length of about 140 feet, refer to Fig. 1. The foundation of the building consisted of 5 to 6.5 feet thick reinforced concrete mat supported on grade with a 2.5-foot thick reinforced concrete slurry wall installed along the perimeter of the mat foundation. Caisson foundations with a minimum bearing capacity of 200 ksf bearing on natural bedrock (Argillite) was proposed for foundations at the southern end for future extension of the silver line subway of the Massachusetts Bay Transportation Authority (MBTA).

#### Site and Project Specifics

The site of the project is blanketed with a 10 to 18 feet thick layer of fill material that was placed over the years to form the south Boston Marina port area. The fill varies from cohesive dredged clay to a mixture of sand, gravel, and some building material debris. The fill layer is underlain by 5 feet of soft organic deposits and up to 90 feet thick deposit of the Boston Blue Clay. The top of the clay is weathered and hardened, forming a clay crust that softens with depth. Glacial deposits, located at about 100 feet below the ground surface, consist of clayey sand and cobbles. The ground water table is located at about 6 to 10 feet below the ground surface.

#### Support of Excavation System

The contract documents called for reinforced concrete slurry walls to retain the soil during the excavation process. The slurry walls were toed in the Boston Blue Clay, and the bottom of the

wall was located at 12 to 20 feet below the bottom of excavation (BOE). The diaphragm walls served as the temporary as well as the permanent lateral support wall along the perimeter of the building. Three to four levels of internal bracing were proposed by the contract documents to support the slurry wall. Tiebacks were also proposed by the contract documents as an alternate to the internal bracing systems.

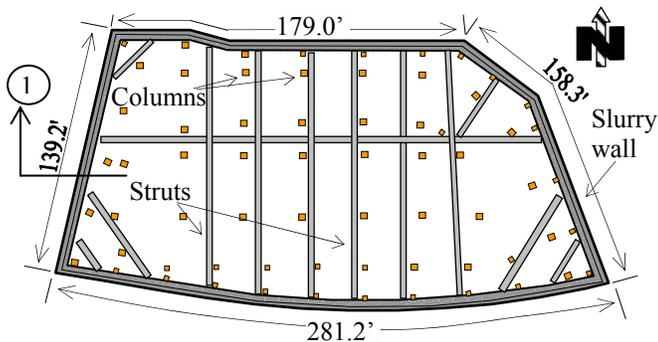


Fig. 1. Plan layout of SOE system

The design of the support of excavation system was influenced by the “up-up” construction technique implemented for the Manulife building. The original design called for the removal of the bracing system after the permanent floors are completed and reached full strength. In the completed building the slurry wall is intended as a basement wall spanning between garage floors to retain the soil. No direct connection was introduced between the slurry wall and the internal floors.

## CONSTRUCTABILITY CONCERNS

### Contract-Proposed SOE

Reinforced concrete ramps were considered to connect the various levels of the underground garage. Because of the configuration of the ramps, interference between the garage slabs and the cross lot bracing was inevitable. Additionally, the final design of the building was still in progress at the time of the bidding for the slurry wall and information on the exact location of the garage level was not available. Therefore, reducing the number of bracing levels within the garage was a primary objective of the Contractor in order to eliminate conflicts between the SOE elements and the garage floors.

As mentioned above, the original design required that the bracing system be removed after casting all slabs within the garage. This would require the use of block out forms in the concrete floors for struts that have conflict with these floors. Also, this requirement would complicate the removal of the SOE elements.

Since large machinery might not be operable inside a garage, the SOE elements had to be cut down to small pieces in order to haul them out of the garage. The Manulife building was designed to be constructed using the ‘up-up’ technique which required building the floors above ground while work is still in progress in the underground garage. This complicated the removal of the

struts because overhead permanent structure would limit the access to the underground garage.

### Contractor-Proposed SOE

Tiebacks were not considered feasible along the south side due to its proximity to the Ted Williams tunnel. Also, the owner’s technical representative had an apprehension that the tiebacks may impact schedule. The test records from earlier tiebacks on the adjacent boat section of CA/T suggested delays due to additional regrouting to improve capacity of the anchors. The option with internal raker system was not found feasible considering geology, depth and size of excavation.

The contractor-proposed support of excavation system consisted of reinforced concrete slurry wall, cross-lot struts and external walers. Two levels of internal bracing were proposed for this project.

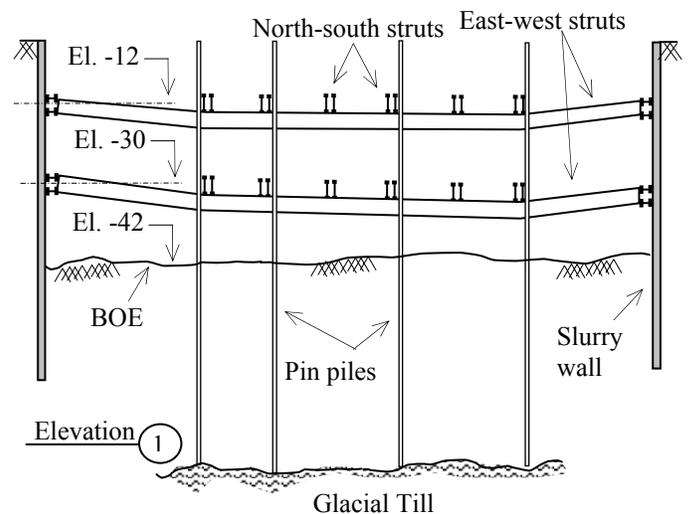


Fig. 2. Support of excavation elevation

The first stage of excavation was carried out approximately 2 feet below the bottom of the upper level bracing. After all the struts were installed and preloaded in the upper level, the excavation was resumed up to 2 feet below the lower level of bracing. The lower level struts were installed and preloaded before resuming further excavation to final subgrade.

The lower level of bracing was removed following the construction of the 5 to 6.5 feet thick mat foundation, refer to Fig. 3, and before any overhead structure was erected. To eliminate interference with the garage floor’s construction, the upper level of bracing was removed following the construction of the garage slab just below this bracing level.

### Basal Stability and Lateral Deflection

The base slab (mat foundation) of the underground garage is supported on a mud mat placed on the medium stiff clay layer at about 44 feet below ground surface. The removal of the 44 feet

of overburden soil would cause unloading of the clay layer, which would lead to bottom heave and basal stability issues. Several measures were adopted to mitigate the effects of any basal heave and lateral deflection of the slurry wall. Such measures included preloading the struts before resuming excavation, designing a stiff reinforced concrete slurry walls and sequential excavation and placing of the mud mat.

Initial analyses indicated a factor of safety of 1.25 for basal stability when the excavation reaches the final subgrade. Since this factor of safety was smaller than the 1.5 value commonly accepted by the engineering community, it was decided to modify the excavation sequence. An unrestricted excavation scheme was adopted up to the bottom of the lower level of bracing. Thereafter, excavation and construction of the bottom mat was planned and constructed in a controlled sequence (in eight segments) to increase the factor of safety for bottom stability to 1.50. Such phased excavation scheme prevented the full unloading of the entire site at the same time, hence, improving the basal stability at the bottom of excavation.

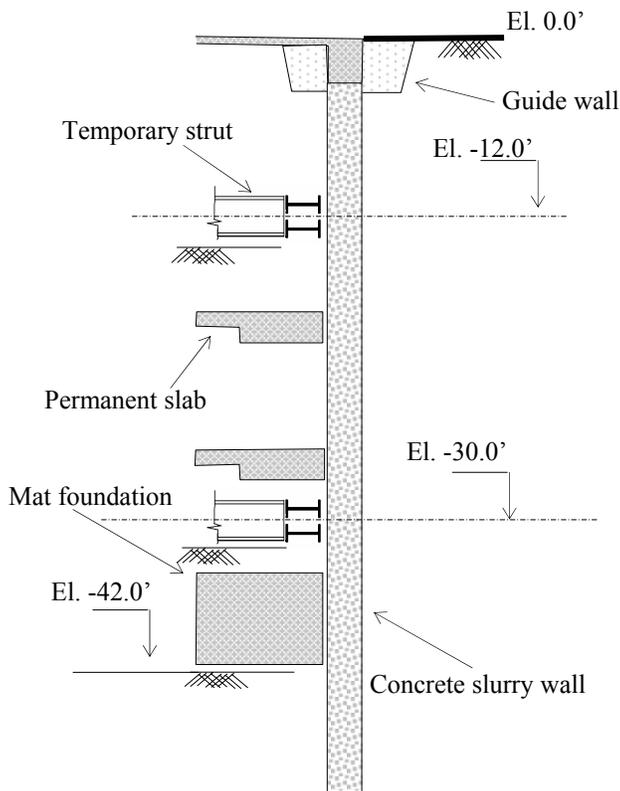


Fig. 3. Typical cross section of the slurry wall.

### Installation of the Slurry Wall

The excavation of the slurry wall was carried out with cable suspended mechanical clamshell buckets. The equipment was chosen considering geology, purpose of construction excavation, depth of slurry wall, site logistics and skill of operator. The clamshell buckets were found effective on previous projects for excavation in soft to stiff Boston Blue Clay. The excavation time was approximately 40% of total construction time.

The verticality of the panel excavation was checked by measuring the position of the cable with respect to the guide walls. Different sizes of teeth were used in the clamshell bucket to correct the verticality. At the “L” shaped corner panels the end-stops were used in the excavated panel (first bite). This procedure supported the clamshell bucket for control of verticality in the remaining bites of the corner panel.

Considering stability of trench, vertical joints, size/weight of reinforcement cages, dimensions of clamshell buckets, site conditions, the panel lengths were restricted up to a maximum of 24 feet. The panel installation sequence was dictated mainly by traffic consideration and specific project requirements. Reinforcement cages were formidable for 24 feet long panels.

The construction joints were formed using removable steel end-stop. After the initial setting of concrete the end-stops were removed. During excavation of the adjacent panels the vertical joints were cleaned to remove bentonite, soil and other impurities left between these joints to form a watertight joint.

The installation of slurry wall through 10 to 18 feet of miscellaneous fill consisting of silt, sand, gravel, ash, cinder, metal slag, brick and wood piles is often risky and challenging task. If not properly addressed, this can lead to collapse of guide wall and slurry trench causing damage of nearby structures. During bidding stages two rows of jet grouting were considered up to the top of marine deposit on either side of the slurry wall. However, considering economics and schedule, it was decided to extensively pretrench and remove the miscellaneous fill and backfill using a flowable mix having an unconfined compressive strength of approximately 100-psi.

During excavation of the slurry wall panels wooden piles were intercepted. Also, on the southern side of the project few detensioned anchors were encountered. These anchors were used earlier for temporary SOE during construction of the CA/T tunnel. These were effectively removed using special tools and adjusting the teeth of the clamshell buckets.

### ANALYSIS 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 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. Analyst 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 and Finite Difference Analyses

Finite element and finite difference analyses methods can be implemented with a variety of commercially available software. The important difference between this approach and the 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.

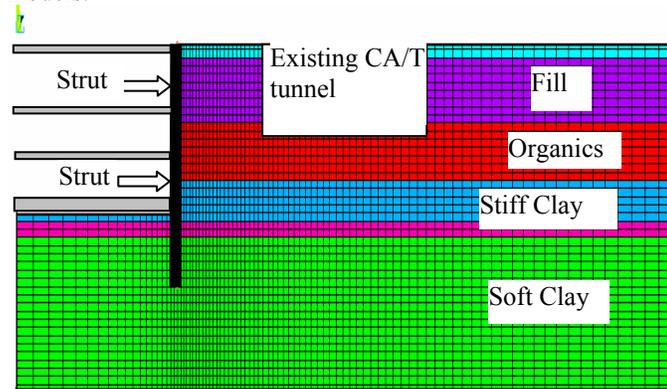


Fig. 4. Finite element model.

The finite element software ANSYS was used to model the staged excavation and construction of the underground garage. The geometry of the sections modeled using ANSYS, including locations of various structural members such as walls, intermediate slabs, and base slab, was taken from the contract drawings. In addition, the soil profile for each section was determined from the geotechnical interpretative report. A typical finite element model is shown in Fig. 4. Several finite element models were constructed for different sections along the perimeter of the slurry wall. The analytical models accounted for the variation of the soil profile, the geometry of the tunnel, and the location of the temporary bracing levels.

Considering the sensitivity of the proposed SOE system and the weak nature of the soil at the project site, a second set of analyses were performed using FLAC software. FLAC is a two-

dimensional explicit finite difference program which was developed originally for geotechnical engineering applications. This program simulates the behavior of structures built of soil, rock or other materials that may undergo plastic flow when their yield limits are reached. FLAC formulations assume a two-dimensional plane strain state, and it allows the definition of initial stress conditions and water table for the calculation of effective stresses.

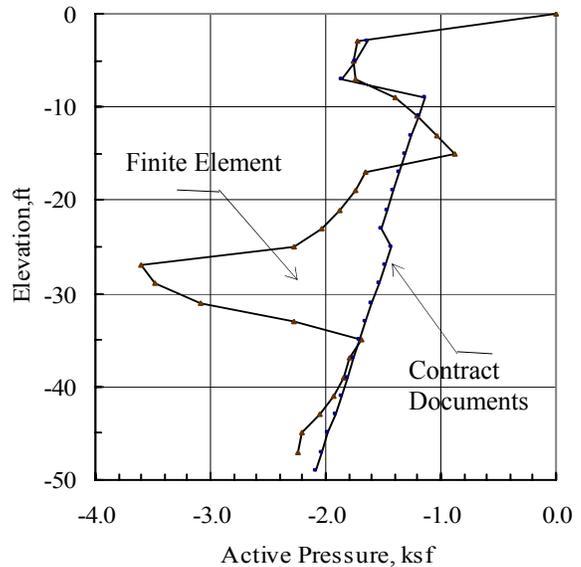


Fig. 5. Active soil pressure on the wall.

### Soil Modeling

In the finite element analyses, 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.

In the finite difference analyses, the Mohr-Coulomb failure criteria, built in FLAC, is used in modeling the soil material.

### Analytical Results

Sample results from the finite element analyses are shown in Fig. 5 through Fig. 7.

The contract documents provided the minimum lateral pressure,

due to earth and construction surcharge, for design of the lateral support of excavation system. This lateral pressure is plotted against the finite element generated pressure in Fig. 5. Generally, there was close correlation between the finite element lateral pressures and the contract document proposed pressures within the upper and lower parts of the slurry wall. However, the finite element models generated larger lateral pressures for the over-consolidated clay crust. This resulted from the apparent conservative soil parameters used in the finite element analyses and specified in the geotechnical report. This level of conservatism was acceptable and did not result in increased steel tonnage for the SOE system.

The finite element analyses considered several excavation and construction stages, however, only the results from the excavation stages are presented in this paper. Negative moment is observed at the location of the bracing struts; refer to Fig. 6.

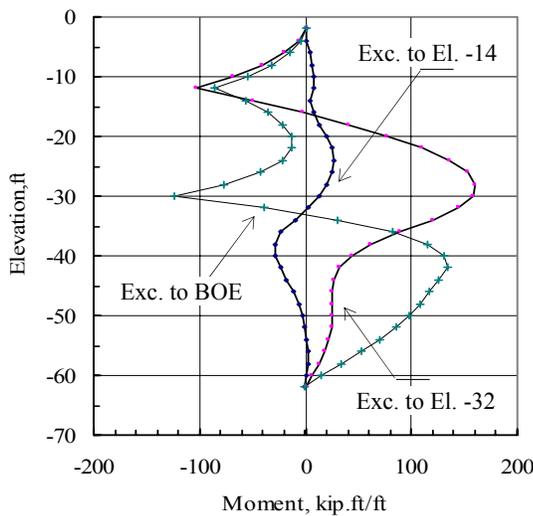


Fig. 6. Bending moment diagrams.

The maximum positive moment occurred after excavating to just below the second level of struts. Note that the soft clay provided little support at the passive side of the wall. In this case, the slurry wall spanned between the first level of bracing and some point in the clay below the bottom of excavation. Note that during the excavation stage, the lateral displacement at the toe of the wall, shown in Fig. 7, relieved the positive and negative moments in the wall.

## CONTRACTOR-PROPOSED SOE

### Design of SOE System

The actual sizing and detailing of the SOE elements was complicated due to the complex nature of the building geometry. The layout of the struts and walers was dictated by the permanent building columns which were to be erected before the removal of any bracing system. Building columns along the inside perimeter of the slurry wall were as close to the slurry wall

as 3 feet. Two options were proposed for the SOE system. The first option utilized internal walers within the concrete slurry wall. Although such option is preferable because of the limited space between the columns and the slurry wall, it imposed more restrictions on the layout of the struts and was bound to create interference issues between the struts and the building columns. A second option utilized external structural steel walers. The depth of the waler would be limited to 30 inches due to the close proximity of the building columns to the slurry wall. This imposed a limitation on the horizontal spacing of the struts. The second option with external walers was adopted.

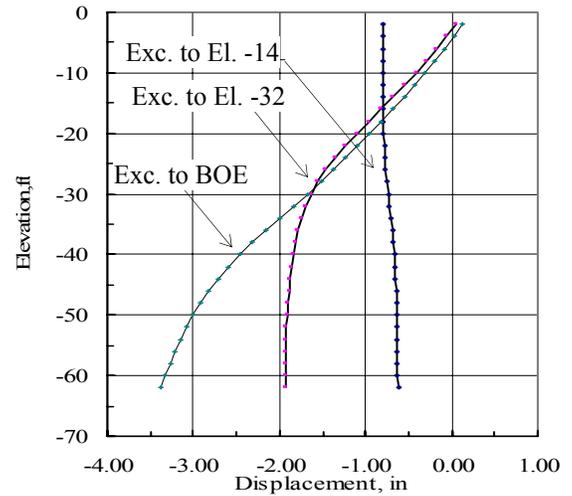


Fig. 7. Lateral deflection of the wall.

Generally, the bracing system consisted of steel wale beams and cross-lot struts, refer to Fig. 8. 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 column pinned at both ends, and were spaced up to 26 feet horizontally.

Two options were presented for the design of struts longer than 110 feet. The first option required the use of pin piles to limit the maximum unbraced length of the strut to about 100 feet. The pin piles, driven 120 feet to bear on top of the glacial till, provided 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. Diagonal steel rods attached to the slurry wall were proposed for this purpose. The first option was implemented. The east-west  $\pm 200$  feet long strut, shown in Fig. 1 and Fig. 2, was used to support the north-south struts. This long strut was supported using a series of pin piles. The design of this strut proved to be a complex task since the walers at the slurry wall had to maintain the same elevation while the strut itself had to pass just below the north-south struts. Therefore, this east-west strut was sloped from one side of the slurry wall down to just below the north-south struts and again up to the other side of the slurry wall as shown in Fig. 2. Further complications were realized because of the need to support the diagonal strut at the north-east corner. Details were developed for this special area in order to distribute the axial force in this strut to the other two orthogonal struts that meet at the end of the corner strut.

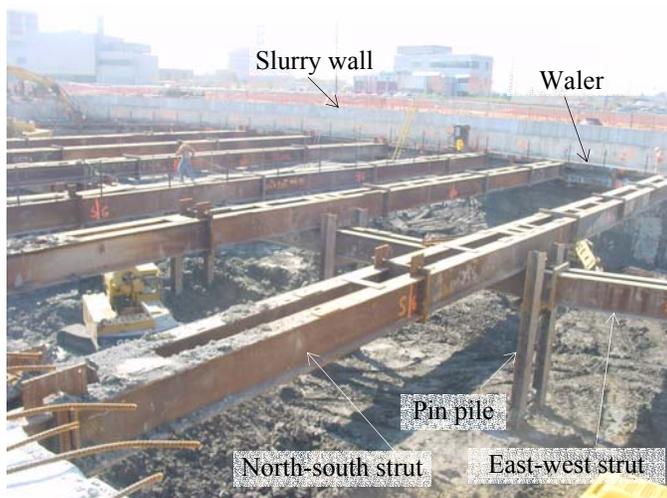


Fig. 8. Support of excavation system.

Walers were designed to support the slurry walls, and they spanned between the cross-lot struts. Walers were designed as beams supported by struts and loaded laterally by reinforced concrete wall. The lateral load of the wall was obtained from the finite element analyses of the staged excavation. Due to the configuration of the slurry walls, some wale beams were designed to resist axial forces in addition to the lateral forces from the slurry wall. In this case, the wale beams were designed as beam-column elements. Several options were considered to create a safe and economical load path for the waler axial forces. In one option, the wale beams were supposed to form a ring around the inside face of the slurry wall. This option would impose many restrictions on the design of the walers and on the excavation at the project site, therefore it was not considered feasible. Second option would transfer the waler axial forces to the reinforced concrete slurry walls through plates embedded in the wall and welded to the wale beams. The embedded plates, with shear studs, were lowered in the concrete slurry wall with the reinforcing cage before placing the concrete. During excavation, those plates were exposed and structural steel elements were used to attach them to the wale beams. This second option was implemented.

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 wall between the cross-lot struts. Because of the limitation on the wale beam depth and the desire to space the struts as far apart as possible, some wale beams were fitted with flange cover plates to increase their flexural strength and reduce lateral deflection.

#### Design of Slurry Walls

The reinforced concrete slurry wall served a double purpose: it

provided the temporary lateral support of the soil during the excavation stage, and it served as the permanent wall of the underground garage. The contract documents specified a 2.5 foot thick reinforced concrete slurry wall. Furthermore, the contract documents stated that the thickness and toe embedment of slurry wall and size of reinforcement were determined for the permanent building conditions. The contractor had to design the slurry wall for the temporary conditions before installing all intermediate slabs that would support the wall in the permanent stage.

The stage-by-stage finite element analyses enabled the contractor to design the slurry wall accounting for all loading conditions during the construction stage. The slurry wall thickness was maintained as specified in the contract documents, however, additional reinforcing rebars were added to account for the stresses during the temporary construction stage.

Although some analyses indicated that the toe embedment of the slurry wall could be reduced, it was decided to maintain the same toe embedment specified in the contract documents. Reducing the toe embedment of the slurry wall might result in basal stability risks that outweigh the savings due to the reduced toe embedment.

The wall analyses and redesign of the SOE system 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 intermediate slabs and the ground floor slab. The finite element analyses also had to demonstrate that the stresses in the walls did not exceed design limits, and, as mentioned above, it was demonstrated that the stresses were actually lower than those originally predicted. Finally, although not a major concern, the analyses were able to demonstrate that the base slabs also were not unduly loaded in the proposed redesign.

### IMPLEMENTATION OF THE SOE SYSTEM

#### Design Development

Once the conceptual design submittal was prepared and presented, the contractor and the owner and its technical consultants, entered into a series of consultations to agree on all the various parameters of the model, its analysis and the resulting design. The initial conceptual model increased the strut level spacing and reduced the line loads to nearly the limits of acceptability within the analyses. Therefore, it became necessary to have a comparable SOE system, 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.

Another factor that influenced these negotiations was the owner's comfort level with the new methods of analysis.

Although the methods of analysis were recognized as accurate and sophisticated, these had not been used long enough to be well validated by empirical data from excavations on completed projects. This factor had to be weighed by the owner against the proposed time and cost savings.

The various technical parameters that became the subject of review during the various revisions of the initial conceptual submittal included the soil models, the strut spacing, both vertically and horizontally, and various issues regarding the detailing of the SOE system.

For the analytical 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 pressures, are applied in the staged analyses, the soil model itself generates both the loads and reactions. In essence, due to the lack of experience with this type of modeling in the area prior to this project, the soil parameters prescribed in the original contract were not readily translated into this constitutive model. Ultimately, values were determined that gave the owner and their geotechnical consultant 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, and enabling the analytical models to effect a savings over these conventional models.

#### Excavation and Installation of the SOE System

With the development of the final SOE design, it was equally important to use suitable construction technologies to maintain movement of the SOE system within acceptable limits (Clough *et al.*, [1990]). Movements of in situ walls are not only a function of stiffness of the support system, but also depend on the selected construction method. Proper slurry wall construction, along with ground water control, mass excavation sequence, bracing erection and placement of base/mud mat are equally important in deep excavation works in soft soils. If any of these issues is not addressed correctly, the SOE can exhibit creep and undesirable movements.

The deployment of experienced and qualified site personnel ensured an efficient installation and performance of the SOE system.

Due to the close proximity of the building columns to the slurry walls, it was important to perform as-built survey of the slurry wall at the level of the bracing to insure that the space between the face of the slurry wall and the building would be enough to fit the wale beams. This was particularly crucial at the lower level bracing along the south wall. In fact, some of the walers along this wall were redesigned with reduced depth in order to avoid interference with building columns.

#### Adjustment of Analyses for Field Issues

The struts had the same elevation from one end to the other end while the underground garage varied in elevation because of the

presence of the ramps. This caused interference of the struts with the garage permanent slabs. Several iterations of analyses were performed to explore the most feasible sequence of strut removal. As the building design was being finalized, the interference issues with permanent structure were identified. It was necessary to modify the construction sequence to allow the removal of some struts before casting the permanent slabs. This resulted in new design conditions for the slurry wall, which required modification of the finite element models.

With seemingly great ease, the finite element models was 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 from the recognition of this conflict, a reanalysis was prepared to account for the modification and detail the proposed solution. This flexibility and adaptability facilitates field changes for both the contractor and the reviewer.

#### Predicted vs. Actual Behavior of the Slurry Wall

Because of the overriding concern for the integrity of the surrounding structures and the excavation site during the excavation, a comprehensive and complete system of monitoring has been installed adjacent to excavation work. This monitoring system includes horizontal and vertical monitoring points on the slurry wall and on adjacent structures and utilities, in addition to a host of subgrade geotechnical instruments. Inclinometers were used to monitor wall movements, while observations wells and piezometers measured groundwater response and heave gauges monitored soil movements. Through the collection and synthesis of data from these instruments, the performance of SOE was closely monitored during 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, we compare the predicted behavior of the wall and SOE system with the actual measured behavior. Comparisons are made at the final stage of excavation and are based on several different measurements. Inclinometer plots are compared to predicted wall movements in Fig. 9. In fact, the actual realized movements are still below the predictions and, consequently, well within the allowable threshold values established to preserve the adjacent structures.

The curves presented in Fig. 9 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 slurry walls, in the finite element models, was calculated based on a cracked concrete section with 75% of the gross moment of inertia. Actual flexural stiffness of the wall might be higher than the assumed stiffness; hence, the actual wall experienced smaller lateral deflection. Furthermore, the ground water table level was determined from the design criteria for the project. In reality, the actual water table level might have been lower than assumed by analysis. 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. Note that the stiffness of the SOE system, rather than the strength, has significant impact on the excavation-induced movements in the soil mass.

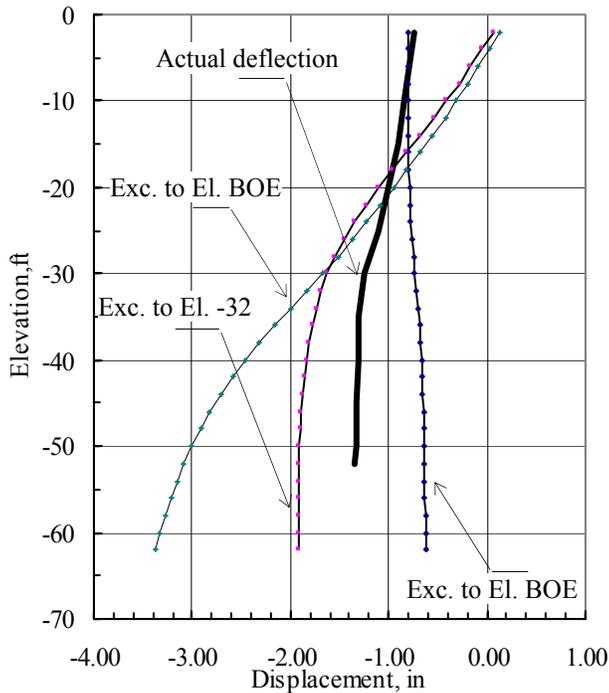


Fig. 9. Theoretical vs. actual lateral deflection.

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

## CONCLUSIONS

The design of the SOE system was performed to satisfy the limits imposed by the design specifications and to address the contractor's desire for improving the constructability of the underground garage structure. The finite element analyses have proven to be vital in the evaluation of not only the behavior and design of the garage structure during excavation and construction, but also in the evaluation of its impact on adjacent structures. The final product was an effective design solution which took advantage of both theoretical and project specific opportunities and, along with good workmanship and proper installation of the SOE system, limited construction induced movements for a deep excavation in soft clays.

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