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SYNOPSIS: Construction of a 21-foot wide, 28-foot deep braced excavation in Detroit soft clays has been completed. In order to protect an existing 50-year old tunnel adjacent to the excavation, a semi-rigid, tangent wall earth retention system was constructed to minimize the soil movements. The tangent wall was formed by 118 drilled piers with 42-inch in diameter and 41-foot long. The maximum soil lateral and vertical movements adjacent to the excavation were controlled below a magnitude of 2.0 inches, while bottom of the excavation experienced about 3 inches of heave. This paper presents the design considerations and construction performance of the retention system based on geotechnical instrumentation data. Prediction of maximum soil lateral movement based on a finite element analysis and a semi-empirical method conformed well with field measurements. Experience learned from the design and construction will be valuable for future construction of braced excavation systems in similar soil conditions.

INTRODUCTION

The City of Detroit Water and Sewerage Department has undertaken a major construction project to expand the Detroit Wastewater Treatment Plant (WWTP). Currently, the plant serves the Southeastern Detroit area and many surrounding suburbs. The expansion will make the Detroit WWTP one of the nation's largest wastewater treatment plant. As a part of the project, a junction chamber, called the Oakwood Junction Chamber (OJC), was designed to convey wastewater flow from an existing 12'-9" inside diameter tunnel, called the Oakwood Northwest Interceptor (ONI), to a new pumping station. The ONI tunnel was constructed in the late 1930's with an unreinforced concrete block (O'Rourke block) primary liner and a cast-in-place unreinforced concrete secondary liner.

The OJC is a soil supported, L-shaped, reinforced concrete structure with a rectangular cross section. The construction of the OJC required a design of an earth retention system to provide a 28-foot deep, 21-foot wide and approximately 180-foot long excavation. The perimeter of the braced excavation system was a semi-rigid tangent wall formed by 118 drilled piers. The concrete-filled drilled piers were 41-foot long, 42-inch in diameter with a W36x230 wide flange steel member embedded in the concrete. The internal bracing comprised five levels of struts and wales. Figures 1 and 2 show a plan and a typical section, respectively, of the OJC retention system and the ONI.

The flow in the ONI could not be diverted or interrupted for construction. Since the ONI was only 21 feet away from the OJC excavation, minimizing soil movements to protect the ONI became a top priority during the design and construction of the OJC. Several analyses were undertaken to predict soil movements and their impact on the ONI. The analyses included a soil-structure interaction finite element model, and a semi-

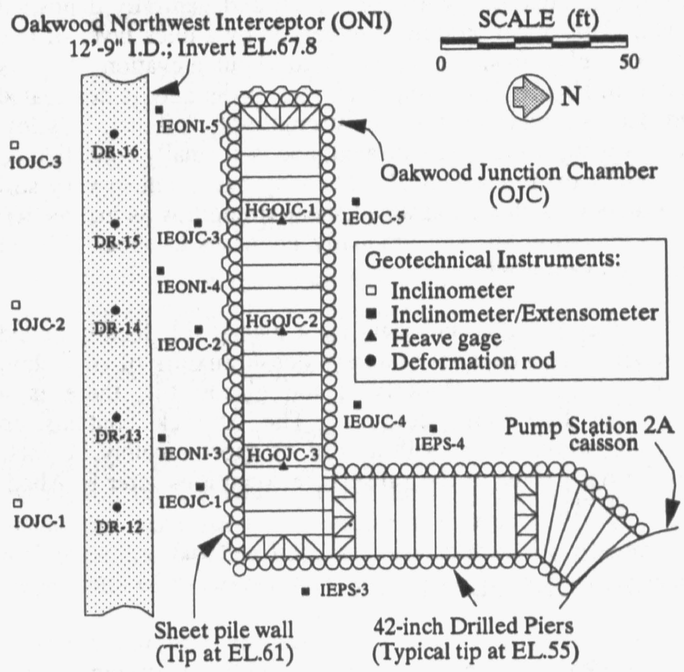

Figure 1 Project site and locations of geotechnical instruments

empirical model based on the factor of safety against basal heave and bracing system stiffness. To compare the predictions with the actual soil movements, an extensive geotechnical instrumentation program was specified and implemented to monitor the soil movements as the excavation progressed.

This paper presents details of the OJC retention system construction history. Design considerations and experience gained from the construction performance are provided.

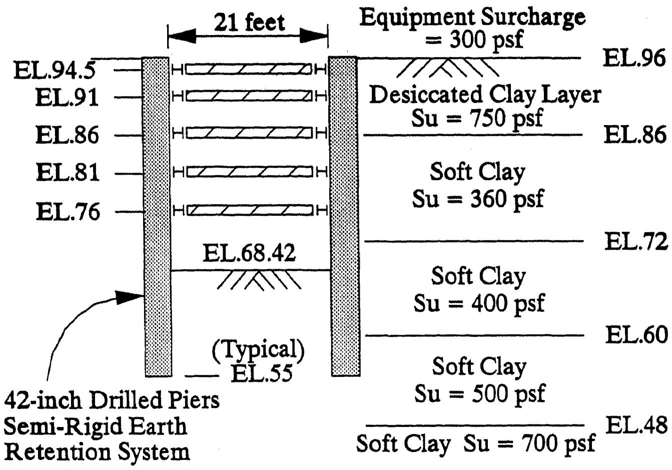

Figure 2 Oakwood Junction Chamber earth retention system and site soil profile

Conclusions and recommendations for deep excavation in soft clay, using a semi-rigid braced excavation system, are also presented.

SUBSURFACE CONDITIONS

The site is covered by 10 feet of fill and naturally deposited sands. This upper stratum is underlain by a thick layer of low plasticity clay from elevation 98 to about elevation 25. As shown in Figure 2, the upper 10 feet of the clay is desiccated and has an undrained shear strength of 750 psf. Below elevation 86, the clay is saturated, and is normally consolidated to slightly overconsolidated. The clay has a soft to very soft consistency, with an undrained shear strength is as low as 360 psf at elevation 86, and gradually increases to about 700 psf below elevation 48.

The clay layer is underlain by a glacial till, locally called hardpan, which consists of a very dense mixture of silty clay, sand and gravel. Underlying the glacial till, there is a competent limestone bedrock. The bedrock contains an artesian ground water with a head of approximately 90 feet. For further details of the subsurface conditions refer to Abedi et al. (1992).

DESIGN CONSIDERATIONS

Earth Pressure

To minimize soil movements and to prevent basal heave, Metcalf & Eddy designed a semi-rigid earth retention system for the OJC construction. Several design approaches were evaluated to determine the earth pressure envelope used for the internal bracing structural design. The design pressure diagram for a braced excavation was primarily developed based on flexible retention systems, such as sheet pile walls. The calculated active earth pressure coefficient, K_A was as high as 1.1 when calculated by the method proposed by Henkel (1971). Based on the local experience with the Detroit clay, however, an active earth pressure coefficient greater than 1.0 was considered to be excessive. The method proposed by Peck (1969), which yielded a K_A equal to 0.8, was used in the design.

Soil Movement Criteria

During the design phase, it was determined that the ONI should not move more than 0.75 inches. Based on the soil movement influence zone proposed by Goldberg et al. (1976), if the tip of the drilled pier moved more than 1.0 inch, the ONI would move more than 0.75 inches and possibly compromise the structural integrity of the ONI.

The importance of predicting the maximum soil movements due to a braced excavation was addressed by Clough et al. (1989). In an attempt to more precisely predict soil lateral and vertical movements due to the OJC excavations, additional detailed analyses were carried out before and during the initial construction phase. Analytical tools used for the prediction included a semi-empirical model proposed by Clough et al. (1989), and a finite element code, SOILSTRUCT, developed by Clough.

The semi-empirical model proposed by Clough requires the following index parameters: the factor of safety against basal heave, depth of excavation, and bracing system stiffness. To be consistent with the parameter used to establish the model, calculation of the factor of safety against basal heave was determined using the method proposed by Terzaghi (1967), which does not consider penetration effect of the buried length of the retention wall.

The SOILSTRUCT is a soil-structure interaction finite element program that can simulate the incremental excavations and installation of internal braces. Behavior of soils follows a nonlinear, stress-dependent hyperbolic stress-strain relationship during primary loading, and a stress dependent, linear response during unloading and reloading. Behavior of structural elements, such as concrete and steel, is assumed to be linear elastic. Interface elements with different properties can be assigned between the soil and the structure elements. Based on the models mentioned above, the predictions of the maximum soil lateral movements are presented in Figure 3. Comparisons between the predictions and actual measurements are discussed in subsequent sections.

Basal Heave

The original design specified a 30.5-foot deep by 21-foot wide excavation. Based on basal heave analyses, it was determined that the drilled piers should be embedded a depth of $(2/3)(B/\sqrt{2})$, where B is the width of the excavation. Accordingly, the drilled piers were extended 10 feet below the bottom of the OJC excavation. Several methods are available to calculate the factor of safety against basal heave including Terzaghi (1967), U. S. Navy's Design Manual 7.02 (1986), and the Pile Buck Steel Sheet Piling Design Manual (1987). With a 30.5-foot excavation depth, the calculated factors of safety varied from 0.80 to 1.20. This range depended primarily on the method of

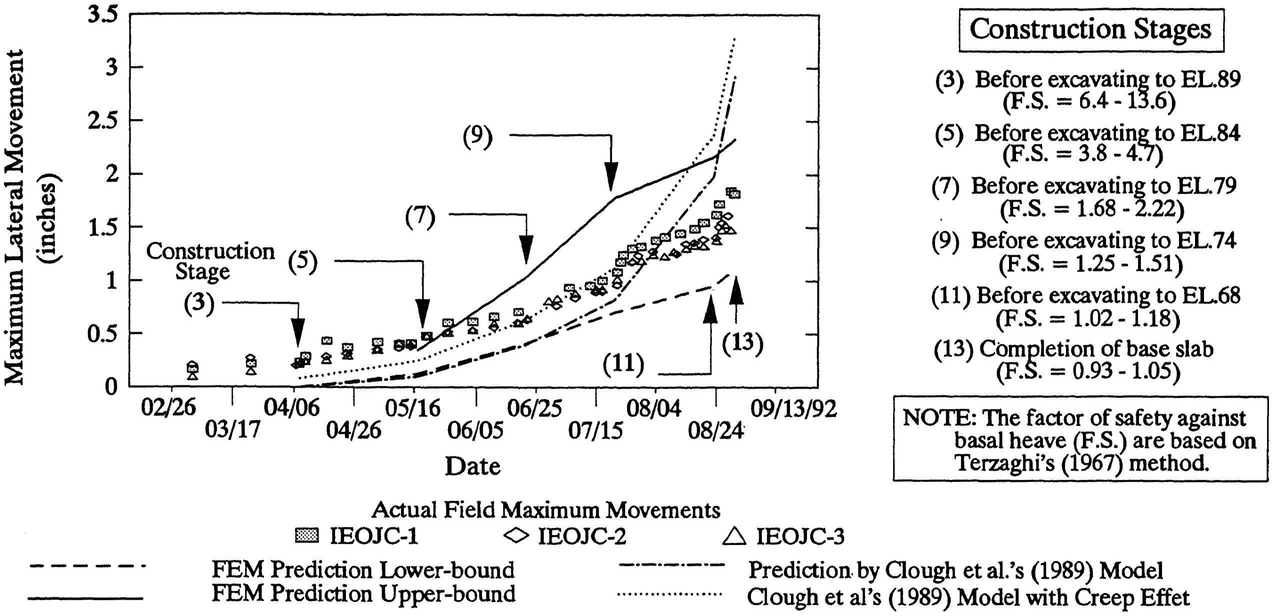

Figure 3 Comparisons of actual maximum lateral soil movements versus predictions

modelling the embedment depth of drilled piers. Other parameters affecting the calculation of the factor of safety include: the width and depth of the excavation, soil undrained shear strength, buried length of the drilled pier, and the contribution of the soil below the bottom of excavation and between the embedded portion of the drilled piers.

In the Terzaghi's method, the basal heave factor of safety is calculated by ignoring the effect of the drilled pier buried length below the excavation. In this case, the depth of the excavation is measured from the ground surface to bottom of the excavation, and the factor of safety against basal heave is equal to 0.8. The full buried length of the drilled piers is accounted for by taking the excavation depth from the ground surface to the tip of the pier, and treating the soil inside the excavation as a surcharge. Due to the penetration of the drilled piers, the undrained shear strength of the soil and the bearing capacity factor were both higher than the corresponding values for the previous method, and results in a higher factor of safety. Furthermore, this latter method allows for the consideration of the adhesion that develops between the soil inside the excavation and the piers. By accounting for the full embedment and adhesion, the factor of safety increases of 1.20 was calculated. For a semi-rigid earth retention system of the OJC, where the potential failure plane is forced to develop below the tip of the piers, the latter approach appeared to be reasonable.

THE OBSERVATIONAL APPROACH

Given that the construction of the OJC was critical to the success of the project, and the excavation was next to the existing ONI, an observational approach was required. As a major component of this approach, Metcalf & Eddy specified an extensive geotechnical instrumentation program. A variety of instruments were installed to monitor the movements of soil mass beneath and adjacent to the excavation. The instruments included inclinometers, inclinometer/extensometers, heave gages, deformation rods, and strain gages. Each instrument was selected to answer one or more of the following questions:

- Is bottom heave occurring?
- Is the existing ONI moving?
- Are the piers settling?
- Are the lateral and vertical soil movements excessive?
- Are the slopes stable?
- Is the structural bracing over-stressed?

The locations of the geotechnical instruments for the OJC are shown on Figure 1. There were three lines of instruments between the excavation and the ONI. The instruments closest to the drilled piers, labelled IEOJC's, were located to provide an early response of the soil as the excavation progressed. The other lines of instruments, labelled IEONI's, and the deformation rods, were used to evaluate the effect of the excavation on the ONI. Three heave gages were installed inside the excavation to monitor the bottom heave. The inclinometers, labelled IOJC's, were installed to monitor the slope south of the excavation.

The actual field measurements at each construction stage were compared to those predicted by different models. Figure 3 shows that the actual movements fell within the upper and lower bound FEM predictions, and conformed well with the predictions using Clough's semi-empirical model. Throughout the excavation, the predictions were used to identify potential soil movements that would be detrimental to the ONI.

CONSTRUCTION OF THE OJC

The OJC site was pre-excavated to provide a 30-foot wide bench at elevation 96 surrounding the OJC excavation area. This bench was intended to provide a balanced earth pressure acting on the drilled piers during the excavation. Beyond the bench, there were approximate 2 horizontal to 1 vertical slopes along the south and west sides of the OJC site.

The first phase of the construction was the installation of the 118 drilled piers along the perimeter of the OJC. The drilled pier installation started by vibrating a 42-inch diameter steel casing into the ground for the full length of the pier. After auguring the soil out of the casing, the W36x230 member was set inside the casing and the annulus was then filled with concrete. The position and alignment of the W36x230 member was adjusted immediately after vibrating the casing out of the ground before the concrete gained initial set. During the removal of the casing, it was discovered that adjacent piers were settling. The effect of the vibration was found to affect piers as far as 6 to 7 pier diameters away. It was concluded that the newly installed piers had not yet gained sufficient shaft frictional resistance to prevent them from sinking. To solve the problem, the W36x230 structural member of the pier was temporarily supported by cross struts seated on wooden mats that were placed on the ground surface.

After the installation of the 42-inch piers, a barrier was to be installed behind the piers to prevent the soil from squeezing through the structural piers and into the excavation. The original design included two types of barriers: a sheet pile wall and concrete-filled backup piers. The sheet pile wall was installed along the south side of the OJC, and concrete backup piers were specified to be along the remaining perimeter of the OJC. The sheet pile wall was required on the south side of the OJC to provide a flat face for future tunnel construction to the ONI. The concrete-filled backup piers were 41-foot long and 24 inches in diameter, and were designed to be tangent to two 42-inch piers. However, the contractor encountered difficulties when attempting to install the backup pier. When vibrating the backup pier casing into the ground, concrete protrusions on the wall of the 42-inch piers forced the casing away from its intended location. After several attempts to install the casing, it was decided to attempt to install the backup piers uncased. As the hole was drilled, the soil squeezed into the open hole, and dragged the adjacent 42-inch piers down. Consequently, it was decided to delete the backup piers. During the installation of the 42-inch piers, the concrete had bulged out. In almost all instances, adjacent piers were in contact with each other for their full length. As the excavation proceeded, this contact was verified and no soil squeezed between the piers.

CONSTRUCTION PERFORMANCE

The excavation and bracing of the OJC commenced in early 1992. Because the bracing design involved large sectional steel members and heavily welded connections, each level of the excavation and bracing construction took about one month. The original design called for a 30.5-foot deep excavation with six levels of internal braces. After evaluation of the base slab elevation, it was decided that the base slab elevation could be raised. Accordingly, a final excavation depth of 28 feet rather than 30.5 feet was specified, and only five levels of bracing were required.

Soil Movements Adjacent to the Excavation

In order to compare actual and predicted measurements, thirteen construction stages of the OJC braced excavation are defined as presented in Table 1. Figure 4 shows the maximum lateral and vertical soil movements at different construction stages. The data are from the inclinometer/extensometers labelled IEOJC's, which were located 5 to 7 feet away from the OJC excavation. At completion of the OJC excavation, the maximum lateral and vertical movements had similar magnitudes of about 1.5 to 2.0 inches, which was about 0.45% to 0.60% of the excavation depth. The maximum lateral movements occurred at elevation 65 to 70, approximately the level of the final excavation. Based on the deformation rod data, the ONI, with a center line elevation of 74, settled about 0.4 inches due to the OJC excavation. Based on the soil movement patterns shown in Figure 4, the abrupt incremental movements that occurred soon after each excavation were followed by a creep type of soil movement during the installation of the corresponding bracing. This creep behavior suggested that the longer the excavation remained open, the greater the soil movements that would occur.

As shown in Figure 3, four prediction curves were made by using a finite element method (FEM) and a semi-empirical approach. The FEM upper-bound and lower-bound predictions were based on different assignments of the interface element characteristics. The third prediction curve was derived from the Clough et al. (1989) semi-empirical model. The fourth prediction was also made using the Clough's semi-empirical model but considering additional soil creep effect based on the field observation of soil movements occurring during the lengthy bracing and welding periods. Construction of each level of the OJC bracing took almost one month, which is longer than the normal case histories used by Clough et al. (1989). Referring to the inclinometer data shown in the Figure 4, soil creep movements during the early construction stages were significant and not anticipated or included in the predictions made prior to the construction. Therefore, based on the field observation, creep movements, ranging from 0.10 to 0.23 inches, were added to the semi-empirical model, and the predictions were revised accordingly.

Reviewing the results shown in Figure 3 suggests that the actual field measurements were within those predicted by the finite element analyses. The comparison also suggests that the

Table 1 Construction stages of the Oakwood Junction Chamber excavation

Construction Stages	Date
(1) Before OJC excavation	02/26/92
(2) Right after excavating from EL.96 to EL.93.5	03/04/92
(3) Completed bracing at EL.94.5; Before excavating to EL.89	04/08/92
(4) Right after excavating from EL.93.5 to EL.89	04/15/92
(5) Completed bracing at EL.91; Before excavating to EL.84	05/17/92
(6) Right after excavating from EL.89 to EL.84	05/22/92
(7) Completed bracing at EL.86; Before excavating to EL.79	06/19/92
(8) Right after excavating from EL.84 to EL.79	06/27/92
(9) Completed bracing at EL.81; Before excavating to EL.74	07/17/92
(10) Right after excavating from EL.79 to EL.74	07/24/92
(11) Completed bracing at EL.76; Before excavating to EL.68.42	08/20/92
(12) Right after excavating from EL.74 to EL.68.42	08/25/92
(13) Completion of base slab	08/31/92

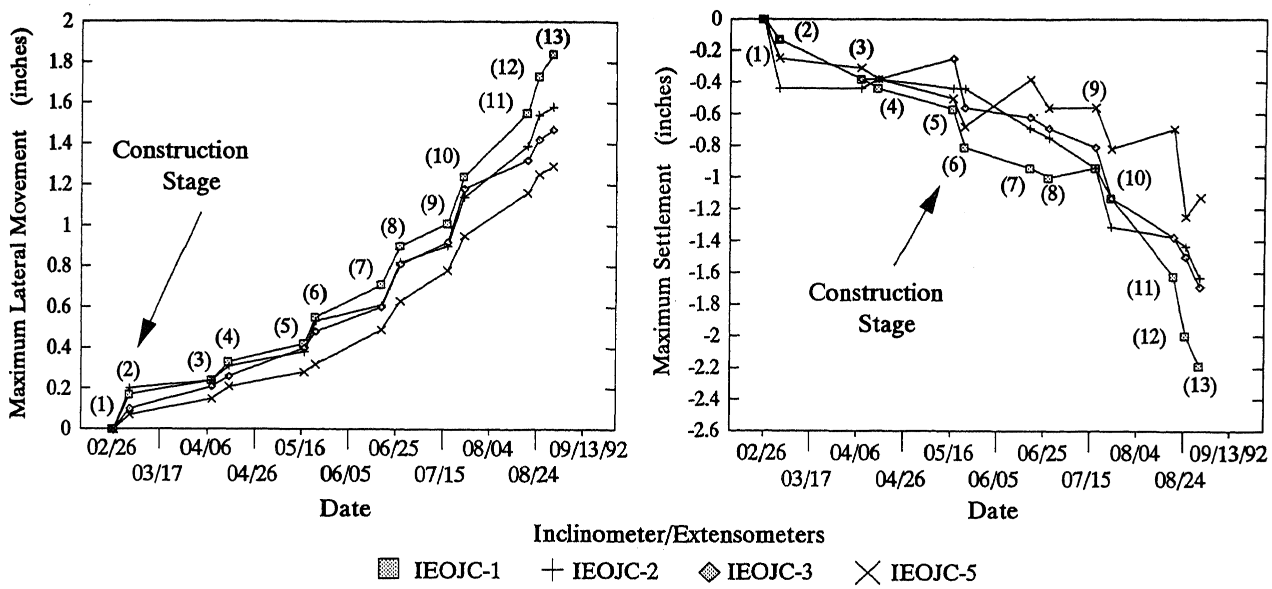

Figure 4 Maximum lateral and vertical soil movements adjacent to the Oakwood Junction Chamber excavation

predictions from Clough's semi-empirical model conformed well with the field measurements except overestimation of the maximum soil movement for the final construction stages. In general, the models used for predicting maximum soil lateral movements were promising. However, there is a discrepancy between the models that remains unclear. The Clough et al. (1989) approach predicted that the rate of movement would increase significantly after excavating to elevation 74, construction stage No.9. However, this trend was not shown in the FEM predictions. In addition, daily monitoring of the actual field measurements did not clearly confirm that the rate of soil movements increases with the depth of excavation.

Basal Heave

As discussed in the previous section, the factor of safety against basal heave of the OJC final excavation was close to 1.0. Heave inside the excavation area was closely monitored by three sondex heave gages at the locations shown in Figure 1. Figure 5 shows the heave measurements at different construction stages. The data show that although the rate of heave increased significantly after excavating to elevation 74. Although the data suggested a tendency of an increasing rate of heave, the total heave measured at the completion of the base slabs was 3.0 inches. The actual heave was less than that predicted by the SOILSTRUCT finite element analysis, which ranged from 4.5 to 9.0 inches.

Performance of the Drilled Piers

The top of the drilled piers was surveyed as a part of Metcalf & Eddy's construction monitor program. Figure 6 shows the settlement history of selected drilled piers. The piers settled at a constant rate before excavating the OJC to elevation 74. At that stage, it seemed that the excavations had no influence on the pier settlements. After excavating to elevation 74, the rate of settlement increased significantly. This was similar to the increase in the basal heave rate that was observed after the same construction stage. However, the actual soil movements did not clearly show the same trend. The pier settlement finally stopped after the base slab was constructed.

CONCLUSIONS AND RECOMMENDATIONS

A 28-foot deep excavation in soft clays using a semi-rigid earth retention system was successfully completed in the Detroit Wastewater Treatment Plant expansion project. Design and construction of the OJC retention system and measured performance during construction provided valuable information for future applications of similar braced excavation systems in soft clays. Based on the performance of the OJC construction, the following conclusions and recommendations are made:

1. The earth pressure envelope for the OJC semi-rigid earth retention system in soft clays was based on the method suggested by Peck (1969). The performance of the bracing system was satisfactory.

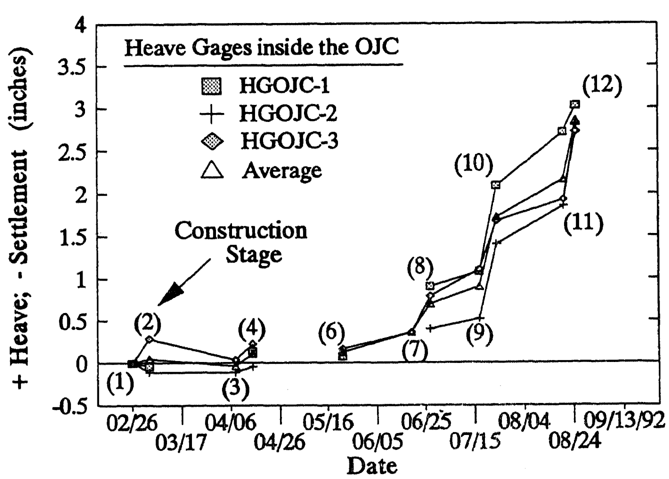

Figure 5 Maximum soil heaves inside the OJC excavation area

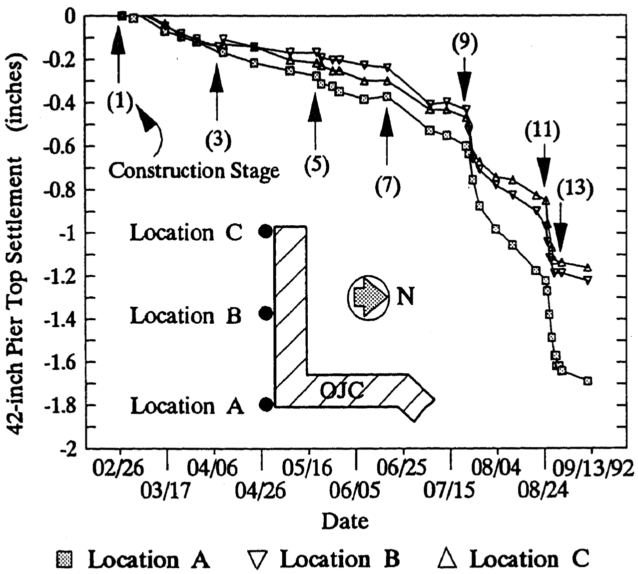

Figure 6 42-inch drilled pier settlements due to the Oakwood Junction Chamber excavation

2. The factor of safety against basal heave of the OJC excavation was in a range close to 1.0. Based on the satisfactory performance of the construction, the semi-rigid tangent wall extended below bottom of the excavation appears to have effectively increased the factor of safety against basal heave. The increase may be attributed to gain of the wall penetration effect, and adhesion developed between the soil inside the excavation and the tangent wall.

3. The semi-rigid earth retention system was effective in minimizing adjacent soil movements. The magnitudes of the maximum soil lateral and vertical movements next to the wall due to the OJC excavation were approximately 1.5 to 2.0 inches, or about 0.45% to 0.60% of the excavation depth.

4. To predict soil maximum lateral movements, a soil-structure interaction finite element analysis code, SOILSTRUCT, and a semi-empirical method proposed by Clough et al. (1989) were used. The actual field measurements fell within the upper and lower bound predictions of the finite element analyses. The semi-empirical model also conformed well to the field measurement except the final stages of the excavation where the model overestimated the maximum soil movements.

5. It is essential to implement an observational approach using a well-defined geotechnical instrumentation program for deep excavations near existing critical structures. When choosing appropriate instruments to answer definite geotechnical questions, the program provides valuable data for evaluation of the design and construction performances. It also provides insight that may not be available during the design phase. Accordingly, modifications during construction can be cost effective and lead to the success of the project.

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