Design, Construction and Performance of a Deep Excavation in Soft Clay

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SYNOPSIS: A deep internally braced excavation in soft clay was performed for a pump station at a sewage treatment plant in Milwaukee, Wisconsin. The design was influenced by the limited site area; potential for bearing capacity failure and/or hydrostatic blowout in the bottom of the excavation; and the necessity to limit ground deformation outside the excavation to protect existing structures and utilities. A performance specification and design was prepared by the owner's engineer. The design included minimum earth and hydrostatic lateral loading conditions to be used by the contractor, a minimum depth of penetration for the earth support system, and a maximum allowable horizontal deflection. The final earth support system design was prepared by the contractor and reviewed by the owner's engineer. Construction monitoring included slope inclinometers (to measure horizontal deflection of the earth support system) and piezometers to measure hydrostatic pressure in a confined aquifer. Measured horizontal deformation of the excavation support system exceeded the predicted deformations. The influence of the contractor's methods and sequence of excavation and earth support system installation on the actual-versus-predicted deformations are also discussed.

INTRODUCTION

The new influent pump station and preliminary treatment facility for the Jones Island Wastewater Treatment Plant is part of the Milwaukee Metropolitan Sewerage District's (MMSD) $1.6-billion rehabilitation and expansion program. The new pump station and preliminary treatment facility is on Jones Island, next to existing treatment plant facilities and new plant facilities under construction, and directly west of a major bridge over the Milwaukee Harbor entrance. The new treatment facility, which receives the main plant influent, is a pile-supported structure. The deepest part of the excavation for the facility was directly adjacent and parallel to the only road entering the plant site. The main plant water, gas, sewage, and storm sewers run adjacent to the excavation, parallel to the main road.

Figure 1 shows the location of the pump station/preliminary treatment facility relative to the existing plant facilities, existing bridge, and new structures under construction. An incineration structure, which had occupied part of the site, was demolished as part of MMSD's program. As part of the demolition, the incinerator building's pile foundation (consisting of 40- to 50-foot-long timber piles) was extracted.

GEOTECHNICAL PROFILE

A field investigation was conducted before design to determine subsurface conditions. The field investigation consisted of soil borings, in situ vane shear testing and laboratory testing of samples obtained from the borings. Additional field investigations were performed during construction. The field investigation revealed a general soil profile consisting of 35 feet of loose to dense granular fill, underlain by 40 feet of estuarine deposited soils consisting of 7 feet of medium stiff organic silty clay, underlain by 15 feet of soft to medium silty clay, underlain by 18 feet of stiff, organic, silty clay. Below the estuarine deposits are 95 to 105 feet of glacial drift, consisting of clay, silt, sand, and gravel. The glacial drift overlies bedrock. Laboratory consolidation tests show that the organic silty clay layers in the estuarine deposits are normally consolidated and highly compressible. The general soil profile of the site and the associated field and laboratory strength tests are shown in Figure 2.
Figure 2 also shows the general groundwater level determined from observation wells installed in select borings on the site prior to and during construction. A surface aquifer and a confined aquifer in the glacial drift, isolated by silty clay layers, were also identified in the field investigation.

The construction of the pump station required an excavation approximately 160 feet by 60 feet in plan dimension and approximately 42 feet deep. The design identified the following concerns:

- The existing sewage treatment plant facilities are founded on short timber piles that terminate in the compressible silty clay layer. Lowering the groundwater table could cause settlement of the timber piles and supported structures. Also, lowering the groundwater table could expose the tops of the timber piles and cause drying and deterioration of the tops of the untreated timber piles.

- The confined aquifer in the glacial drift has a hydrostatic pressure equal to Elevation -1.4 feet (MMSD datum). With the excavation 42 feet deep (approximate Elevation -35), the magnitude of the hydrostatic boundary force at the bottom of the silty clay layer (confining layer) exceeds the total weight of the silty clay layers below the bottom of the excavation. The uplift pressure was calculated as 4.2 kips per square foot; the pressure from the weight of the silty clay soil within the excavation above the glacial soil was calculated as 3.4 kips per square foot.

In addition, the demolition of the incinerator building and the extraction of the timber piles left zones that may have been loosened and weak-

- The soft- to medium-stiff clay layer at the excavation bottom appeared susceptible to failure due to bottom heave. Estimates of the factor of safety (F.S.) against bottom heave ranged from slightly less than 1.0 to 1.2. Using Terzaghi's model (Terzaghi and Peck, 1967), and assuming that the average shear strength below the excavation bottom is 850 pounds per square foot, gives the following:

\[
F.S. = \frac{N_c S}{\gamma D + q}
\]

Where:

- \(N_c = 6.3\) (bearing capacity factor, from Bjerrum and Eide, 1956)
- \(S_{AVG} = 0.85\) ksf
- \(\gamma D = 4.9\) kips per square foot (ksf)
- \(q = 0.6\) ksf (assumed surcharge)
- \(F.S. = (6.3) \frac{0.85\text{ksf}}{4.9\text{ksf} + 0.6\text{ksf}}\)
- \(F.S. = 0.97\)

- The immediately adjacent treatment plant utilities had to remain in service during construction, and thus the design had to accommodate that need.

DESIGN APPROACH

Primary factors in the approach to the design of the excavation were:

- The location of the excavation and the critical nature of the adjacent utilities, transportation facilities, and existing plant facilities
- The tight schedule of the construction project and the impact that any delay would have on this and subsequent projects
- Use of standard construction methods and techniques

It was determined that delay in construction or interruption of service due to a construction failure was an unacceptable risk. The excavation design, therefore, intruded into the contractor's traditional area of design of temporary earthwork facilities. The excavation design approach that was adopted had the following objectives:

- Reduce the risk of temporary excavation failure through performance criteria
Use prescriptive criteria to remove some of the contractor's judgement and risk-taking ability

Adopt a construction monitoring program to measure compliance with the contract documents and assess the effectiveness of the contractor's methods.

To achieve these objectives, the contract documents contained the following provisions:

- A 2-inch lateral-movement limit on the excavation support system. The limit was based on Peck (1969) and Hansen and Clough (1978). Two inches was intended to be a "warning" limit and was estimated as approximately one-half to one-third of the maximum movement, based on empirical data presented by Hansen and Clough (1978) for braced excavation with prestressed bracing.

- A requirement for depressurizing the lower aquifer within the excavation to a level within 10 feet of the bottom of the excavation when the excavation was below Elevation -27. This requirement was set to prevent bottom blowout or boiling due to hydrostatic pressure.

- A requirement that the contractor extend the temporary earth support system to the glacial drift layer (Elevation -68) to address the bottom heave problem.

The contract documents also contained a geotechnical report that addressed the issues of constructability and design of the excavation for the pump station.

The geotechnical report contained lateral earth pressures to be used by the contractor during design of the excavation. The lateral earth pressures above the excavation bottom were based on apparent earth pressure theories developed by Terzaghi and Peck (1967). For the embedded portion of the excavation support, it was recognized that an unbalanced earth pressure may develop. Earth pressures were calculated with Rankine earth pressure theories, using undrained strength values. For design, it was assumed that the excavation wall was pinned at the bottom support, and fixed at the bottom of the steel sheets (Elevation -68).

The contractor was also required to preload each brace to between 35 and 50 percent of the estimated design loads.

EARTH SUPPORT SYSTEM

The layout of the earth support system and the location of slope inclinometers are shown on Figure 3. Final locations for the inclinometers were chosen after the contractor's final excavation design was approved. Two slope inclinometers were installed on the west side of the excavation. Seven piezometers were also installed next to the excavation to monitor hydrostatic pressures in the lower aquifer. From Elevation +7 (ground surface) to Elevation -5, the contractor chose to open excavate the east side, and install soldier piles (HP 10 42 at 7-foot centers) and wood lagging on the north, south, and west sides of the excavation. Foundation piling were driven from Elevation -5. Sheetpile were used to enclose the excavation on all sides below Elevation -5. The sheetpile used was Hoesch 175 (∅ = 48.4 inches/1f) on the west side, and PZ 32 (∅ = 38.3 inches/1f) on the east side. A transition piece was installed consisting of a PZ 32 interlock split lengthwise and welded to an H-175 sheet.

The schedule for the walers and bracing is shown in Table 1. Braces on the north end of the excavation were larger members because the span was longer. Sheetings on the west side extended to Elevation -65, and on the east side to Elevation -38.

Depressurizing in the lower aquifer was accomplished with two 3-inch submersible pumps. The flowrate during construction was steady and approximately 200 gpm. Table 2 shows piezometer levels prior to and during construction dewatering. The tips of the piezometers are in a sand layer in the glacial drift at approximately Elevation -70. Surface dewatering for the remainder of the site was accomplished by pumping from shallow wells inside a slurry wall. The slurry wall was constructed around the site and intersected the sheeting.
Table 1

<table>
<thead>
<tr>
<th>Level</th>
<th>Walers</th>
<th>Struts</th>
<th>K = (AE)/L</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>W33x118</td>
<td>N = 22&quot; x 312</td>
<td>58.5</td>
</tr>
<tr>
<td></td>
<td>S = 10&quot; x 312</td>
<td>44.0</td>
<td>1,020</td>
</tr>
<tr>
<td>2</td>
<td>W30x108</td>
<td>N = 22&quot; x 312</td>
<td>61.0</td>
</tr>
<tr>
<td></td>
<td>S = 17&quot; x 312</td>
<td>47.0</td>
<td>955</td>
</tr>
<tr>
<td>3</td>
<td>W36x150</td>
<td>N = 22&quot; x 312</td>
<td>58.5</td>
</tr>
<tr>
<td></td>
<td>S = 20&quot; x 312</td>
<td>44.0</td>
<td>1,130</td>
</tr>
<tr>
<td>4</td>
<td>W33x118</td>
<td>N = 22&quot; x 375</td>
<td>58.5</td>
</tr>
<tr>
<td></td>
<td>S = 20&quot; x 375</td>
<td>44.0</td>
<td>1,130</td>
</tr>
<tr>
<td>5</td>
<td>W30x108</td>
<td>N = 22&quot; x 375</td>
<td>58.5</td>
</tr>
<tr>
<td></td>
<td>S = 20&quot; x 375</td>
<td>44.0</td>
<td>1,365</td>
</tr>
</tbody>
</table>

Notes:
- aInclined.
- N = North side.
- S = South side.

Results

Depressurizing the Lower Aquifer

Table 2 shows the drawdown in the piezometer installed in the lower aquifer.

Table 2

<table>
<thead>
<tr>
<th>Piezometer</th>
<th>Level</th>
<th>Before Pumping</th>
<th>15 Days After Pumping</th>
<th>30 Days After Pumping</th>
<th>60 Days After Pumping</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-1.4</td>
<td>-16.4</td>
<td>-13.0</td>
<td>-19.9</td>
<td>-21.6</td>
</tr>
<tr>
<td>2</td>
<td>-1.3</td>
<td>-15.8</td>
<td>-11.1</td>
<td>-16.8</td>
<td>-18.1</td>
</tr>
<tr>
<td>10</td>
<td>-1.4</td>
<td>-18.2</td>
<td>-13.0</td>
<td>-19.9</td>
<td>-21.6</td>
</tr>
<tr>
<td>11</td>
<td>-1.4</td>
<td>-15.8</td>
<td>-11.1</td>
<td>-16.8</td>
<td>-18.1</td>
</tr>
<tr>
<td>12</td>
<td>-9.2</td>
<td>-20.9</td>
<td>-17.3</td>
<td>-19.5</td>
<td>-21.6</td>
</tr>
<tr>
<td>13</td>
<td>-9.2</td>
<td>-20.9</td>
<td>-17.3</td>
<td>-19.5</td>
<td>-21.6</td>
</tr>
</tbody>
</table>

Notes:
- aNot read.
- bNot installed.

NOTE: Groundwater levels are elevations (MMSD datum).

Measured Deflections

Figures 4 and 5 show measurements from slope inclinometers SI-1 and SI-2, which were on the west side of the excavation. Also shown on Figures 4 and 5 are the excavation sequence and brace installation sequence, noted in days after the excavation started.

The slope inclinometer data shows that a great deal of movement occurred at the soldier pile and lagging support while it was cantilevered, and again before Brace No. 2 (the inclined brace) was installed. Additional movement continued at each brace level after the braces were installed.

Although not shown on Figures 4 and 5, there was additional deflection between the end of September and the end of December, several months after all braces had been...
DISCUSSION

Sheet Pile Wall Deflections

The most significant features of deflection behavior as shown on Figures 4 and 5 are the movement before installation of the upper level braces, and the continued movement during, and even after, all levels of bracing were installed.

As shown on Figures 4 and 5, the first level of excavation was reached on Day 8 and the first level bracing was not installed until Day 30. As shown on Figure 5, the deflection at Day 12 (2 days after the excavation to Level 1 was completed) was less than 1/2 inch. This was the case at both the top of the earth support system and at Elevation -11, the elevation at which the first level bracing was to be installed.

Figure 4 shows that by Day 27 significant deflection (approximately 2-1/2 inches) had occurred at the top of the sheet piling. Further, Figure 5 shows that between Day 12 and Day 47, 2 days after the Level 2 bracing was installed, the deflection at the top of the sheeting was approaching 6 inches. The deflection at Elevation -11 exceeded 2 inches at both inclinometer locations before the installation for the first level bracing. The upper 18 feet of earth support system (soldier pile and lagging and upper sheet piling) were allowed to cantilever for approximately 30 days. If the first level of bracing had been installed within just a few days after the completion of the first level excavation and the cantilever condition had existed only for a very short duration, the deflections of the wall would have been reduced significantly. The duration and sequence of critical construction phases (in this case, installation of the upper level braces) had a significant influence on the behavior and performance of the earth support system.

Approximately 1 inch of movement also occurred at each brace level after the braces were installed. Each brace was preloaded by calibrated jack as it was installed, and shimmed with the jack in place. Upon investigation, however, it was found that some of the walers had separated from the sheeting (in other words, that there was a gap between the walers and the sheeting in some brace locations). Some of the walers used in the construction were cambered wide-flange sections salvaged from a bridge demolition project. Some of the gaps between walers and sheeting were 6 inches wide. Once found, the gaps were shimmed (starting on Day 65), and subsequent movement was relatively less. Table 3 indicates the estimated maximum load at each strut (based on the loads given in the geotechnical report), the relative stiffness of each strut, the ideal elastic deflection at each strut, and the actual deflection at each strut level after strut installation through the time the base slab was cast.

It can be seen that the actual deflections exceed the theoretical deflections by a factor of 3 to 8. The imposed preload in the struts was 100 kips, which should also have reduced the theoretical deflections. O'Rourke (1981) discussed preloading practices and the effect that preloading has on the effective stiffness of the excavation support system. It appears that the preloading was ineffective and that most of the preload may have been taken in the camber of the walers. It further seems that the excavation support system was actually much more flexible than its theoretical stiffness.

The continued movement of the earth support system, even after all levels of bracing were installed, is not well understood. It is believed that this continued movement occurred in part as a result of thermal contraction (the temperature was approximately 120°). Another factor may have been a general trend of the entire system to

<table>
<thead>
<tr>
<th>Level</th>
<th>Estimated Maximum Strut Loads (kip)</th>
<th>Strut Stiffness k = A/E/L (kip/in)</th>
<th>Ideal Elastic Deflection 6 = P/k (in)</th>
<th>Actual Deflection (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>252</td>
<td>N = 935</td>
<td>.27</td>
<td>1.6</td>
</tr>
<tr>
<td>2</td>
<td>200</td>
<td>N = 885</td>
<td>.25</td>
<td>1.1</td>
</tr>
<tr>
<td>3</td>
<td>302</td>
<td>N = 935</td>
<td>.22</td>
<td>1.7</td>
</tr>
<tr>
<td>4</td>
<td>612</td>
<td>N = 1,125</td>
<td>.54</td>
<td>1.5</td>
</tr>
<tr>
<td>5</td>
<td>503</td>
<td>N = 1,125</td>
<td>.45</td>
<td>1.4</td>
</tr>
</tbody>
</table>

N = North side.
S = South side.
move eastward as the theoretical earth pressures on the west side of the excavation exceeded those on the east side. The entire east side for several hundred feet was excavated to Elevation -5, approximately 12 feet lower than the west side.

Base Heave

No instrumentation was installed to monitor or measure base heave, but it did not appear that base heave was a problem. In fact, it appears that extending the sheet pile wall into the dense and/or stiff glacial soils provided an effective means of controlling base heave.

As discussed previously, the potential for base heave was identified early in the design phase. The analysis of the base heave problem indicated a marginal to unacceptable factor of safety, ranging from 1.2 to less than 1.0. Although base heave did not appear to be a problem, the potential for base heave was evidenced during construction by significant measured movement of the buried portions of the sheet pile wall at locations where heaving potential was suspect (approximate depth of 50 feet). The deflection at depth 50 feet approached 4 inches. This movement is shown in Figures 4 and 5.

The movement measured in the buried portion of the sheets also suggests that the earth pressures on the buried portion of sheeting should be considered in the design of the earth support system. In this case, the buried section was considered as fixed at the top of the glacial soils and pinned at the lower level brace. The earth pressure was based on Rankine active earth pressure theory. This approach provided adequate design of the earth support system.

Hydrostatic Blowout

Maintaining the hydrostatic pressure in the lower aquifer to less than 10 feet above the bottom of the excavation provided adequate protection against blow out and/or boiling.

Design Approach

The design approach for this project was unusual in that areas of traditional contractor design (temporary earthwork and excavation support facilities) were specifically addressed in the design and construction contract documents in terms of prescriptive and performance criteria. Specifically, the contractor was given:

- The minimum earth pressures to be used in design of the earth support system, along with maximum deflection criteria
- The minimum depth of the bottom of the earth support system (in this case, the minimum tip elevation of the steel sheet piles)
- Criteria regarding groundwater and hydrostatic pressure levels

This design approach was adopted because it allowed the contractor to design the details of the earth support system and incorporate these details with his chosen method of construction. At the same time, this approach removed some of the contractor's judgement and risk-taking ability in the design of temporary construction works. That removal was deemed necessary to minimize the potential for construction delays or interruption of service of existing utilities and facilities.

The criteria for the final design of the temporary support works was modified from the criteria in the contract documents as a result of geotechnical investigation performed during construction by the contractor. The design approach in this case prompted the following positive results:

- The contractor retained the services of a geotechnical engineer to further investigate appropriate criteria for the design of the earth support system.
- The contractor participated in instrumentation beyond that required by the contract documents.
- The project was completed on time without serious interruptions or inconveniences to the owner. The only failure experienced was a broken 10-inch water line during backfilling and removal of the sheeting.

CONCLUSIONS AND SUMMARY

A deep excavation was made in soft- to medium-stiff clays for an influent pump station and preliminary treatment facility for the Jones Island Wastewater Treatment Plant in Milwaukee, Wisconsin. The excavation used an unusual design approach in that the contract documents contained performance and prescriptive criteria to address traditional contractor design areas. Lateral deflections were monitored as the excavation proceeded and the earth support system bracings were installed.

Significant deflections in the earth support system occurred early in the project and continued movement occurred after all levels of bracing were installed. The duration and sequence of critical construction phases, particularly the timing of the bracing installation, have a significant influence on the deflection behavior of earth support systems. Careful preloading and ensuring preload transfer also have a significant influence on deflections.

Conventional methods of controlling base heave and hydrostatic blowout provided adequate performance. Extending the sheet piles below the layer with heaving potential into denser materials provides adequate performance, provided the earth pressures on buried portions of the sheeting are considered in the design of the earth support system.
The unusual approach to the design of the excavation support system encouraged positive input from the contractor and resulted in a successful project.

ACKNOWLEDGEMENTS

The geotechnical investigation for this project was one element of the overall design of the Jones Island Wastewater Treatment Plant expansion conducted for the Milwaukee Metropolitan Sewerage District (MMSD). Permission of the MMSD is gratefully acknowledged.

REFERENCES


