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Existing Pile Load Capacity Evaluation

Alan Kropp
Alan Kropp and Associates, Berkeley, California

SYNOPSIS:

An evaluation of the capacity of piles supporting an existing five-story building became necessary when heavy new shear walls were proposed for the structure. Data regarding these piles was obtained from the soil investigation report for the project, from two pile load tests at the site, and from driving records for 229 production piles. Additional information was derived from soil borings and pile load tests for two adjacent buildings. Calculations were made of the anticipated pile capacities of piles at load test locations, and empirical correction factors developed to modify the calculated values to match the load test results. The same calculation methods and empirical correction factors were then used to develop ultimate capacities at each production pile location, and appropriate safety factors applied to estimate allowable pile capacities.

INTRODUCTION

A five-story parking garage was constructed in 1972. In 1982, some cracking in the structure was observed and a detailed evaluation of the original garage design was performed. The evaluation concluded that five shear walls should be added to the facility to provide adequate lateral stability during seismic shaking. A geotechnical evaluation of the existing pile foundation was performed to determine if the piles could carry the additional weight of the new shear walls.

EXISTING DATA

A soil investigation for the parking garage was originally performed in 1970 which provided recommendations for the proposed pile foundations. This study included the drilling of five exploratory borings on the site. Two pile load tests were performed in March 1972, and 229 production piles were driven in March and April of 1972.

Two 13-story office buildings were constructed on piles adjacent to the parking facility. The foundation investigations for these two sites were performed in 1969 and 1972 and included four and three exploratory borings, respectively. One pile load test was performed for the second office building.

The approximate locations of the parking facility, the office buildings, the exploratory borings and the pile load tests are shown on the Site Plan, Figure 1.

LEGEND

- - - - - - - Approximate Location of Property Line
- - - - - - - Approximate Building Limits
P-1 △ Approximate Location of Exploratory Boring for Parking Facility
I-1 △ Approximate Location of Exploratory Boring for Office Building No. 1
II-1 △ Approximate Location of Exploratory Boring for Office Building No. 2
PT-1 △ Approximate Location of Test Pile for Parking Facility
II T-1 △ Approximate Location of Test Pile for Office Building No. 2

Figure 1. Site Plan

APPROXIMATE SCALE
(Feet)
SITE CONDITIONS

A. Exploratory Boring Data

The subsurface materials encountered in the borings below the parking facility generally consisted of soft to stiff, silts and clays with sand lenses extending to a depth of about 45 feet. Below this depth, dense sand layers with occasional clay layers were encountered extending to the maximum depth explored of about 95 feet. Gravel layers were encountered in several of the borings, particularly near the bottom of the borings. The borings below the first office building site generally encountered sand layers with silt lenses in the upper 45 feet, and then sand and gravel layers below that depth. At the second office building site, the borings typically encountered clay layers with interbedded sands extending to a depth of about 80 feet, and then sand and gravel layers to the bottom of the borings at a depth of about 100 feet. Thus, the subsurface conditions were somewhat different at each of the three sites.

Although some variability was encountered in the borings, the groundwater was generally present a depth of about 15 to 25 feet at the time of the investigations.

B. Test Pile Data

Two pile load tests on 10-inch square, prestressed concrete piles were performed on the parking garage site. As shown on the Site Plan, one test pile was located adjacent to Boring 1 (in the western corner) and the other was located adjacent to Boring 3 (in the eastern corner). Test Pile 1 (adjacent to Boring 1) was driven to a depth of about 63 feet, while Test Pile 2 (adjacent to Boring 3) was driven to a depth of about 63 feet. Load testing indicated an ultimate capacity of 117 tons for Test Pile 1, and an ultimate capacity of 195 tons for Test Pile 2. Based on this information, the soil engineer said that the proposed heavily loaded production piles (carrying loads of 83 to 97 tons) should be extended to a depth of 77 feet and should have blow counts of at least 30 blows per foot for the last 3 feet.

One pile load test was performed adjacent to Boring 3 on a 12-inch square prestressed concrete pile for the second office building (see the Site Plan). This pile was driven to a depth of about 84 feet (after the upper 40 feet of soil had been predrilled). The pile load test indicated an ultimate capacity of about 320 tons. For the design capacity of 150 tons, the soil engineer recommended all production piles be predrilled to a depth of 40 feet and then driven 10 feet into the dense sand and gravel layer encountered at a depth of about 75 feet. 

C. Production Pile Driving

The pile driving records for the parking garage indicated that 229 blows were required to depths ranging from about 66 to 90 feet (although most tips were at depth between 77 and 80 feet). All piles were 10-inch square, prestressed piles.

EVALUATION AND CONCLUSIONS

A. Soil Layers

On the basis of the pile driving blow counts, and the soil boring logs, rough classifications were made between types of soil (clay, sand, or gravel) and subdivisions were established between individual soil types (i.e. soft, firm, or stiff clay). These soil categories are presented on Tables 1, 2, and 3. These soil categories are presented on Tables 1, 2, and 3. Three types of clay, three types of sand and two types of gravel were identified, and each soil type was given a different layer designation. Differentiation between clay and sand layers was partially based on the blowcount, while distinctions between sand and gravel layers was primarily based on the blowcount.

B. Soil Properties

Very limited soil testing was performed by the soil engineers on samples obtained from the five borings at the parking garage site. Water content and dry density tests were performed on most samples, while five direct shear tests and one consolidation test were performed on other samples. Because of erratic test results, these results were not used to a significant degree in our analyses. Also, very few standard penetration resistance values were recorded, so little information regarding soil properties could be obtained using correlations between blow counts and soil properties.

The soil layers encountered in the borings at the second office building site were somewhat similar to those at the garage site, except the upper sand layers below the parking facility were not encountered in the office building borings. General laboratory tests performed by the soil engineer on samples recovered from the borings included moisture content, dry density, and Atterberg limits tests. In addition, one direct shear test and two consolidation tests were performed. Most samples were obtained with a 3.25-inch diameter sampler driven by a 342-pound weight dropping 18 inches. These values were converted to rough standard penetration resistance (N) values using general correlation factors which considered driving energy and sampler size differences.

The evaluation of the soil property data that existed from previous laboratory testing and rough correlations with standard penetration resistance values indicated that very sparse data on soil properties was available. Therefore, it was concluded that a more reliable procedure to determine load capacity was to estimate soil strength properties based on the pile driving blowcounts, and then modify the strength characteristics of the soil layers using a ratio of the load capacity of the test piles computed using these properties to the actual load capacity determined by the pile load tests.

The initial soil properties selected for the various soil sublayers are presented on Tables 1, 2, and 3. Wet density values were generally obtained by evaluating wet density values recorded on the soil samples tested by the soil engineer at the garage.

### TABLE 2. CLAY PROPERTIES

<table>
<thead>
<tr>
<th>Clay Type</th>
<th>Pile Driving Blowcount</th>
<th>Wet Density (pcf)</th>
<th>Undrained Cohesion (psf)</th>
<th>CK</th>
<th>Undrained Soil – Pile Adhesion (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>5 – 2</td>
<td>120</td>
<td>500</td>
<td>0.90</td>
<td>450</td>
</tr>
<tr>
<td>B</td>
<td>5 – 6</td>
<td>120</td>
<td>750</td>
<td>0.84</td>
<td>630</td>
</tr>
<tr>
<td>C</td>
<td>7 – 10</td>
<td>125</td>
<td>1000</td>
<td>0.71</td>
<td>710</td>
</tr>
</tbody>
</table>

### TABLE 2. SAND PROPERTIES

<table>
<thead>
<tr>
<th>Gravel Type</th>
<th>Pile Driving Blowcount</th>
<th>Wet Density (pcf)</th>
<th>Undrained Soil – Pile Friction Angle - θ (%)</th>
<th>Vesic (1967)</th>
<th>Corrected Value</th>
<th>f/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>G</td>
<td>25 – 65</td>
<td>125</td>
<td>38</td>
<td>1.7</td>
<td>0.90</td>
<td>90</td>
</tr>
<tr>
<td>H</td>
<td>55 – 1000</td>
<td>120</td>
<td>41</td>
<td>2.4</td>
<td>1.27</td>
<td>120</td>
</tr>
</tbody>
</table>

### TABLE 3. GRAVEL PROPERTIES

<table>
<thead>
<tr>
<th>Sand Type</th>
<th>Pile Driving Blowcount</th>
<th>Wet Density (pcf)</th>
<th>Undrained Soil – Pile Friction Angle - θ (%)</th>
<th>Vesic (1967)</th>
<th>Corrected Value</th>
<th>f/A</th>
</tr>
</thead>
<tbody>
<tr>
<td>D</td>
<td>1 – 4</td>
<td>125</td>
<td>30</td>
<td>1.05</td>
<td>0.56</td>
<td>-</td>
</tr>
<tr>
<td>E</td>
<td>5 – 14</td>
<td>130</td>
<td>33</td>
<td>1.20</td>
<td>0.64</td>
<td>51</td>
</tr>
<tr>
<td>F</td>
<td>15 – 25</td>
<td>130</td>
<td>35</td>
<td>1.36</td>
<td>0.72</td>
<td>65</td>
</tr>
</tbody>
</table>
site. The undrained cohesion values for clay layers, as well as the drained friction angle for sand and gravel, were estimated on our experience with similar materials in the area.

C. Shaft Resistance

From Poulos (1980), the following formulas for shaft resistance were obtained:

For clay:
\[ P_{su} = A_b c_a \]

For sand and gravel:
\[ P_{su} = A_b K_s \tan \theta'_{a} \sigma'_{v} \]

where \( P_{su} \) = ultimate shaft resistance
\( A_b \) = shaft area
\( c_a \) = soil-pile adhesion
\( K_s \) = coefficient of lateral pressure
\( \theta'_{a} \) = drained friction angle between soil and pile
\( \sigma'_{v} \) = effective stress along shaft

The factors used in our initial capacity calculations for clay layers were determined by interpolation between the factors proposed by Tomlinson (1937) and Kerisel (1965). When sand or gravel layers were present, the relationships between the friction angle for sand and gravel layers, were obtained:

\[ f_{b}/f_{s} = \text{ratio of base resistance to shaft resistance} \]

In clay layers, the commonly accepted value of \( N_c = 9 \) proposed by Skempton (1959) was used. Where sand or gravel layers were present, the relationship of \( \theta \) to \( f_{b}/f_{s} \) proposed by Vesic (1967) was adopted for the study.

D. Tip Resistance

The equations for tip resistance of the pile were also obtained from Poulos (1980). The equations used in this study were:

For clay:
\[ P_{bu} = A_b c_u N_c \]

For sand and gravel:
\[ P_{bu} = A_b f_{b}/f_{s} \]

where \( P_{bu} \) = ultimate base resistance
\( A_b \) = base area
\( N_c \) = bearing capacity factor
\( c_u \) = undrained soil cohesion

This comparison indicates that calculated values were generally about twice the recorded values. It appears that the overestimation of the load capacities resulted primarily from overestimation of the soil properties because of the lack of quality laboratory testing of soils at the site. To a lesser degree, the overestimation might be attributed to the use of the Vesic relationship of \( \theta \) vs \( K_s \tan \theta'_{a} \) rather than the relationship proposed by Meyerhof (see Figure 2). However, the similarity of the ratios between the recorded and calculated values for the three piles is quite good, considering that varying soil profiles were present at the three locations and that PT-1 bears on clay, PT-2 bears on sand, and IIT-1 bears on gravel.

The majority of the calculated pile loads on all of the three test piles were carried by the portions of the pile shaft in sand or gravel layers, while the clay layers contributed relatively minor amounts. Therefore, it was decided to apply an empirical correction factor to the sand and gravel, and not correct the clay values. By applying a correction factor of 0.53 to the \( K_s \) tan \( \theta'_{a} \) values originally obtained from Figure 2, the shaft and tip resistance in sand and gravel layers was modified so that the calculated and recorded capacities were nearly the same. The corrected values of \( K_s \) tan \( \theta'_{a} \) are presented on Figure 2. The \( K_s \) tan \( \theta'_{a} \) values used are generally between the values proposed by Vesic (1967) and Meyerhof (1976) for sand layers (\( \theta = 30^\circ - 33^\circ \)), but lower than both values for gravel layers (\( \theta = 38^\circ - 41^\circ \)). This may mean that the sand properties were accurately estimated originally, but the gravel properties were overestimated.

E. Correction Factors

Using the equations, soil profiles, and soil properties previously discussed, the ultimate capacity was calculated of the two test piles on the site, as well as the test pile on the second office building site. The calculated capacities were then compared to the values recorded in the pile load tests. The results of these comparisons are presented below.

<table>
<thead>
<tr>
<th>TP</th>
<th>CUC (Tons)</th>
<th>RUC (Tons)</th>
<th>RC/CC</th>
</tr>
</thead>
<tbody>
<tr>
<td>PT-1</td>
<td>193</td>
<td>117</td>
<td>0.61</td>
</tr>
<tr>
<td>PT-2</td>
<td>369</td>
<td>195</td>
<td>0.56</td>
</tr>
<tr>
<td>IIT-1</td>
<td>623</td>
<td>320</td>
<td>0.51</td>
</tr>
</tbody>
</table>

TP = Test Pile Designation
CUC = Calculated Ultimate Capacity
RUC = Recorded Ultimate Capacity
RC/CC = Recorded Capacity/Calculated Capacity

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F. Ultimate Capacity

Using the techniques discussed above, a procedure was developed to estimate the present ultimate load capacity for the existing piles under the five proposed shear walls. The results indicate the piles studied had ultimate capacities which vary from about 160 tons to 384 tons (320 to 768 kips). Soil conditions, pile driving records and load capacity data for the two extreme piers are summarized on Figures 3 and 4.
H. Factors of Safety

Typically, a factor of safety of 2.0 is used to convert ultimate pile capacity to an allowable value for dead plus live loads. However, Terzaghi (1967) indicates that "if the ultimate bearing capacity is determined by means of a load test, the customary factors of safety range between 1.5 and 2." It is our opinion that a factor of safety of 2 is a reasonable value to apply to ultimate capacities determined after soil borings have been drilled and pile load tests performed for a project because large uncertainties still exist about soil profile variations between boring locations. At the garage site, the pile driving records for 229 piles were studied to provide a much more detailed evaluation of soil conditions across the site. Because the level of uncertainty concerning subsurface conditions was substantially reduced for this project, it was recommended that a factor of safety of 1.5 be applied to the ultimate capacities to obtain an allowable capacity for dead plus live loads.

There is little published information about the factor of safety to be applied to ultimate pile capacities to determine the allowable capacity for earthquake loading. Standard practice is often to apply a one-third increase to the allowable capacity determined for dead plus live loads. We recommended that a factor of safety of 1.1 be used to convert ultimate pile capacity to allowable capacity during earthquake loading. We should note that if a one-third increase is used on the allowable dead plus live load capacity obtained using a safety factor of 1.5, then the resulting factor of safety for earthquake loading is about 1.13.

REFERENCES


