Overburden Effect on the Axial Resistance of Instrumented CFA Piles

Timothy C. Siegel[1], M. ASCE, P.E., G.E., D.GE and Peter Faust[2]

[1]Principal Engineer, Dan Brown and Associates, PC, 1808 Northshore Hills Blvd., Knoxville, TN 37922, ph. 865-809-4883, tsiegel@danbrownandassociates.com
[2]Project Manager, Malcolm Drilling Company, Inc., 3503 Breakwater Court, Hayward, CA 94545, ph. 510-780-9181, pfaust@malcolmdrilling.com

ABSTRACT: Static pile loading tests and high strain dynamic tests were performed to confirm the foundation design for a major expansion to a hospital on the Stanford University campus in Palo Alto, California. The foundations were 610 mm (24 in) diameter continuous flight auger (CFA) piles to be installed at the bottom of a staged excavation. Advantages to the construction schedule made it desirable to execute this test program on piles installed approximately 6.1 m (20 ft) above the final excavation level; however, such an approach was recognized to require accounting for the overburden effect prior to application of the results to design. Therefore, the test piles were instrumented with strain gages and subsequently monitored throughout testing in an effort to allow meaningful interpretation of the overburden effect. Dynamic testing of one of the test piles after excavation to grade suggests that the implemented methodology resulted in a conservative geotechnical design.

INTRODUCTION

A load test program was performed to confirm the design of 610 mm (24 in) diameter continuous flight auger (CFA) piles for a major expansion to the Lucile Packard Children's Hospital on the Stanford University campus in Palo Alto, California. The expansion included a seven-story hospital building and a three-story parking garage. To optimize the construction schedule, the test piles were installed above the final excavation level. The design team recognized that such an approach would require accounting for the overburden effect in the data interpretation and design confirmation. The overburden effect may be divided into two components:

(1) Side resistance that existed during the test loading from soil that was subsequently removed prior to installation of the production piles, and;

(2) Changes in side and toe resistances as a result of the excavation of overburden soil and associated reduction in the vertical stress and increase in the over-consolidation ratio (OCR). The vertical stress and OCR are parameters expected to influence the geotechnical axial resistance of coarse-grained soil.

SITE CONDITIONS

The site is located within the Coast Ranges geomorphic province which is characterized by generally northwest-trending, elongated mountain ranges with peak elevations of 610 to 1220 m (2,000 to 4,000 ft) above sea level separated by narrow valleys (Rutherford & Chekene, 2009). Locally, the site is underlain by an alluvial plain situated between the foothills of the Santa Cruz Mountains and the San Francisco Bay. The regional slope is gently inclined toward the bay. Ground surface elevations in the vicinity of the site are estimated to be approximately 24.4 to 29 m (80 to 95 ft). The project location is shown in the Site Vicinity Map in Figure 1.

FIG 1. Site vicinity map

The subsurface conditions are characterized by a 60 to 215-m (200 to 700-ft) thick layer of well consolidated Pleistocene-age alluvium overlying Jurassic- to Tertiary-age bedrock. The upper alluvium can be divided into two general conditions – (1) coarse-grained soils that may be described as clayey sand/well graded sand and locally silty sand, sandy silt, gravel lenses, and cobbles, and (2) fine-grained soils consisting mostly of clay. The results of a typical test boring performed at the site are illustrated in Figure 2. The ground water was maintained at a depth of approximately 1.2 m (4 ft) below the ground surface.

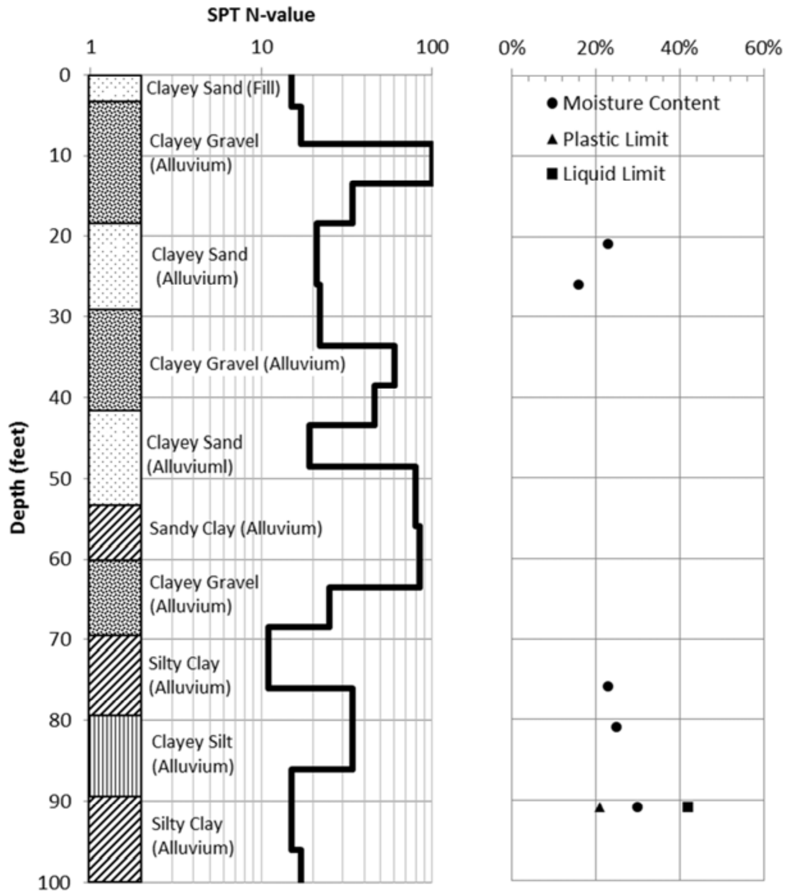

FIG 2. Typical test boring (1ft = 0.305m)

PILE TEST PROGRAM

The test program included eight (8) 610 mm (24 in) diameter CFA piles (designated TP1 thru TP8) installed using a continuous flight auger to excavate a cylindrical volume of soil and then pumping fluid cement grout through the hollow stem of the auger into the excavated volume as the auger is extracted. The drilling platform was a Bauer BG-28 which is a fixed-mast rig capable of generating significant downward force or crowd. While the cement grout was still fluid, a full length No. 18 centerbar and partial length cage of eight (8) No. 8 (415 MPa or 60 ksi) longitudinal bars were inserted into each of the test piles. The test pile lengths are summarized in Table 1.

Table 1. Test pile lengths (1ft = 0.305m)

Test Pile	Length m (ft)	Test Pile	Length m (ft)
TP1	25.1 (82.4)	TP5	28.4 (93.0)
TP2	27.8 (91.1)	TP6	27.9 (91.5)
TP3	28.2 (92.4)	TP7	25.2 (82.8)
TP4	25.0 (82.1)	TP8	23.0 (80.6)

Each test pile was instrumented with multiple levels of resistance type strain gages (supplied by Applied Foundation Testing) attached to the steel reinforcement to allow monitoring of the axial strain along the pile length during static and dynamic testing. The static loading tests were performed in accordance with the Quick Method described in *ASTM D1143 Standard Test Method for Piles Under Static Axial Compressive Load* prior to dynamic testing performed by GRL Engineers, Inc. using the APPLE® device. The maximum applied static compressive load was limited to the safe load of the test frame of slightly greater than 5.3 MN (1200 kips). A photograph of the static test frame setup on one of the test piles is Figure 3. The APPLE® device (66.7 kN or 15 tons) used for this project is shown in Figure 4. The dynamic impact to the top of the pile was typically applied by dropping the weight from a height of 1829 mm (72 in). The high strain dynamic pile measurements were collected and processed with a Pile Driving Analyzer® (PDA) in accordance with *ASTM D4945 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations.*

FIG 3. Static test setup (note the excavation slope in the background)

SUMMARY OF RESULTS

The results of the pile loading tests are summarized in Table 2. The distributions in the mobilized resistances developed during the static tests were determined from interpretation of the strain gage data as described by Siegel (2010). The CAPWAP capacities were determined using an iterative procedure which uses measured force and velocity data to determine a matching soil model. The solution includes equivalent static resistances.

FIG 4. APPLE® device

Table 2. Results of Pile Loading Tests kips (1kip = 4.45kN)

Test Pile	Static Test Results (kips)			CAPWAP Capacities (kips)		
	Side	Toe	Total	Side	Toe	Total
TP1	1202	25	1227	1870	430	2300
TP2	1191	36	1227	1930	250	2180
TP3	1193	36	1229	1750	400	2150
TP4	1138	90	1228	1635	365	2000
TP5	1169	53	1222	1865	375	2240
TP6	1095	137	1232	1800	260	2060
TP7	1121	48	1169	1390	585	1975
TP8	1194	25	1219	2150	460	2610

Notes:
1. The static test results are mobilized resistances at relatively small top movements.
2. The CAPWAP capacities are ultimate static resistances.

INTERPRETATION AND APPLICATION

It was readily apparent from the results that the static tests did not fully mobilize the geotechnical resistance of the piles. The maximum pile head movements during static testing were small [on the order of 8 to 10 mm (or 0.3 to 0.4 in)]. A compilation plot of the top load versus the head movement is presented in Figure 5. Also, the CAPWAP capacities were well above the corresponding resistances that were mobilized during static testing. This precluded any verification of the CAPWAP capacities with the static results. To incorporate a substantial degree of conservatism, the methodology applied to the interpretation and application focused on the static test results as follows:

- Develop a compressive load distribution by interpretation of the strain gage data;

- Determine the mobilized side resistance corresponding to approximately 6.1m (20ft) of soil that will be excavated prior to installation of the production piles;

- Assign parameters of the β-method for sand and sandy soil where: $\beta=(1-\sin\phi')\tan\phi' OCR^{\sin\phi'}$ (ϕ' is the effective friction angle and OCR is the overconsolidation ratio) and the α-method for clays by matching the compressive load distribution interpreted using the strain gage data;

- Calculate the design side resistance for sands and gravels using the anticipated post-excavation effective stress and OCR and;

- Estimate the design toe resistance using conventional geotechnical calculations based on the conditions encountered in the test boring nearest the respective test pile. This was necessary because none of the test pile fully mobilized the toe resistance.

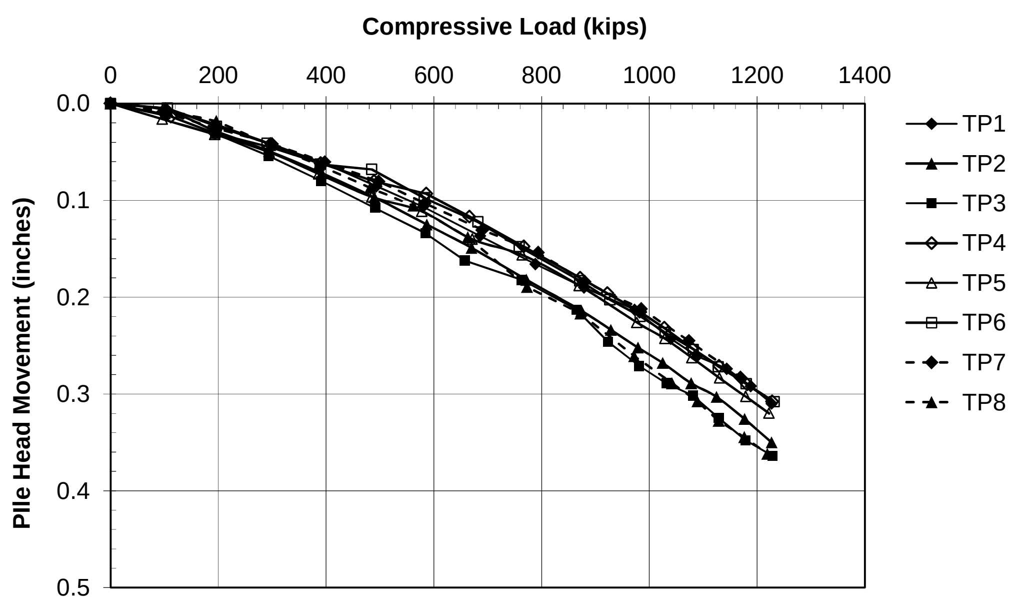

FIG 5. Pile Head Movement Versus Top Load (1 in = 25.4 mm, 1 kip = 4.45 kN)

Interpretation and Application of Test Pile TP2

In this section the proposed methodology for accounting for the soil overburden effect is illustrated for test pile TP2. The distribution of internal compressive load as interpreted by the strain gage data is graphically presented in Figure 6. The strain gage data interpretation assumed that the residual load was negligible at the time of testing. A non-linear secant modulus (varying with strain) was determined for each pile using the data from the strain gages that were embedded two feet below the pile top. The modulus was adjusted for the reduction in reinforcing steel below the bottom of the reinforcing cage in proportion to the steel in the cross-sections.

The resistance mobilized in the upper 6.1 m (20 ft) of soil (which will be excavated to achieve the design subgrade) was determined by subtracting the internal compressive load of 3715 kN (835 kips) at 6.1 m (20 ft) below ground from the applied load of 5460 kN (1227 kips). The resulting difference of 1745 kN (392 kips) is the portion of the applied top (compressive) load that is resisted by the pile-soil interface along the upper 6.1 m (20 ft) of pile. The dashed lines representing the internal compressive load distribution in Figure 6 appear to be nearly parallel between the upper two strain gage levels for the final few loading increments. This suggests that the side resistance along this portion of pile was fully mobilized during testing.

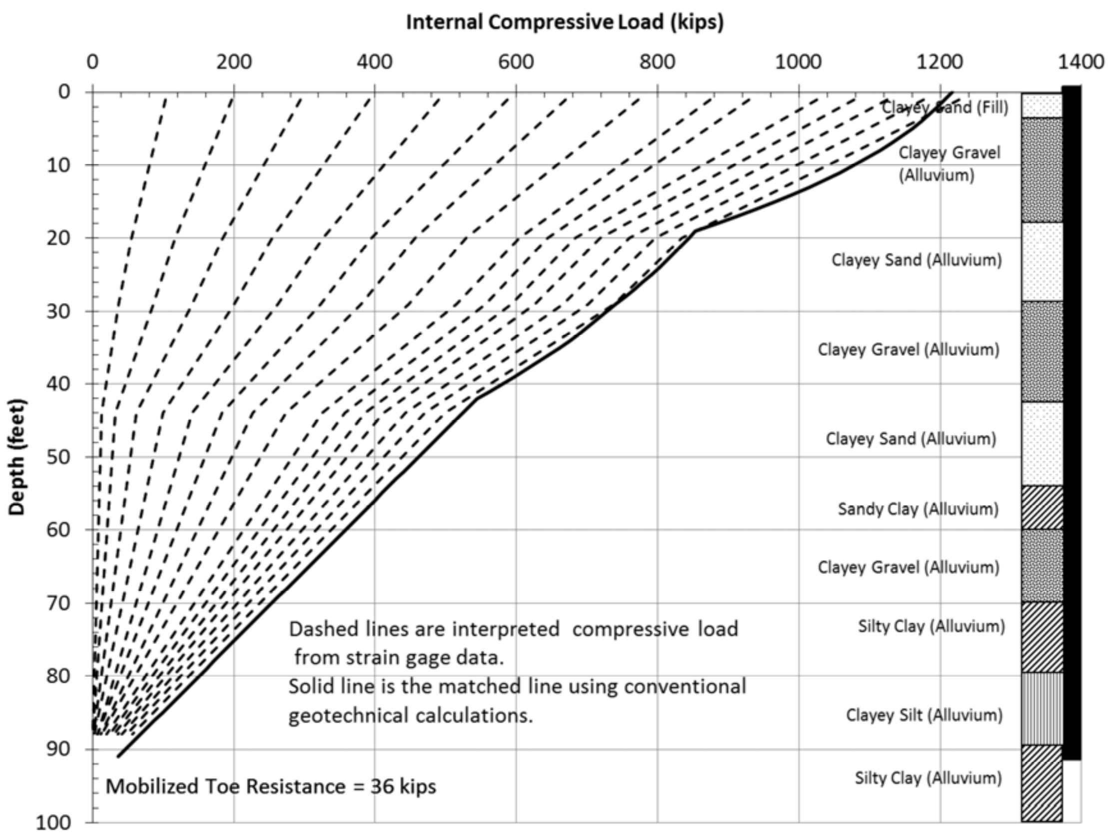

FIG 6. Axial Compressive Load Distribution for Test Pile TP2 (1 ft = 0.305 m, 1 kip = 4.45 kN)

The solid line in Figure 6 represents axial compressive load distribution backfitted using the β-method and α-method for coarse-grained and fine-grained soils, respectively. Judgment was necessary during backfitting and particularly selection of ϕ' and OCR. The side resistance for design was then calculated using the backfitted parameters and the anticipated post-excavation effective vertical stress and OCR. The toe resistance for design was calculated using conventional geotechnical calculations for the post-excavation condition based on data from the nearest test boring.

Results for All Test Piles

The results for all the static pile loading tests are summarized in Table 3. The adjustments for stress change are significant as their values range from 360 kN to 1001 kN (81 kips to 225 kips) approximately 7% to 17% of the measured side resistance. It is hypothesized that the reason for this is that the horizontal stresses are greater within the partial excavation that existed at the time of testing than would have been present in the free-field. Overall, the results illustrate the importance for accounting for overburden effect in load tests performed prior to planned excavations or prior to potential scour.

Table 3. Results for All Test Piles kips (1kip = 4.45kN)

Test Pile	Measured Side Resistance	Side Resistance in Upper 6.1m (20ft)	Adjustment for Stress Change	Adjusted Side Resistance	Toe Resistance	Design Resistance
TP1	1202	327	152	723	393	1116
TP2	1191	383	92	716	85	801
TP3	1193	453	81	659	393	1052
TP4	1138	150	225	763	85	848
TP5	1169	225	200	744	393	1137
TP6	1095	451	142	502	393	895
TP7	1121	439	165	517	93	610
TP8	1194	134	129	931	480	1411

Notes:

1. Adjusted Side Resistance = Measured Side Resistance – (Side Resistance in Upper 6.1m + Adjustment for Stress Change)
2. Toe resistance is estimated based on test boring data using conventional geotechnical correlations.

Test pile TP-4 was dynamically tested a second time several weeks after the initial testing once the area had been excavated approximately 6.1 m (20 ft) to prepare the design grade. The results of the dynamic testing predict a side resistance of 4895 kN (1100 kips) and a toe resistance of 2626 kN (590kips) for a total compressive resistance of 7521 kN (1690 kips). The increase in toe resistance from 1624 kN (365 kips) to 2626 kN (590kips) as inferred from the dynamic testing of test pile TP-4 suggests that the soil at the pile toe was preloaded during downward pile movement that occurred during the static pile loading test and initial dynamic testing. Dynamic testing of production piles at final grade level resulted in predicted side resistances of 4139 kN and 4895 kN (930 kips and 1100 kips) while the predicted toe resistances were 579 kN and 712 kN (130 kips and 160 kips).

CONCLUSIONS

On the basis of the results of this study, the following conclusions are made:

(1) The overburden effect and specifically the change in side resistance due to the reduction in vertical stress can be significant for the pile loading tests performed prior to planned excavations or prior to potential scour.

(2) The observations that the side resistance was greater than expected was due, at least in part, to greater horizontal stresses within the partial excavation that existed at the time of testing than would have been present in the free-field.

(3) A rational method for accounting for overburden effects in the interpretation of pile loading tests is proposed. Specific to this project, the geotechnical resistance was not fully mobilized and, as a result, there is inherent conservatism included in its application.

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