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SYNOPSIS: Supplemental geotechnical investigations were conducted on two Detroit area projects with the purpose of optimizing design criteria for proposed drilled pier foundations. For both projects, the major effort involved the formulation, implementation and interpretation of a load test using an Osterberg load cell rather than conventional dead weight or reaction piers. Load and settlement trends were monitored with a series of strain gauges and telltales. Field data are presented in graphical form to illustrate the results of the load tests.

INTRODUCTION

The Osterberg load cell is a viable alternative to dead weight or reaction piers for performing load tests on drilled piers to loads as high as 3,000 tons (26,700 kN). Furthermore, it has the advantage of separating the end bearing and side friction components of a drilled pier. Definition of these components can be useful in the design of production piers which may be of varying sizes due to the magnitude of building loads being supported. A majority of the references given for this paper describe the development and past use of this device for load testing of drilled piers.

The Osterberg load cell is embedded within the test pier, in close proximity to the tip of the pier. It consists of a 14-inch (356 mm) high by 34-inch (864 mm) diameter expandable cell which is used for applying vertical loads both downward (on the bottom end of the test pier) and upward (along the sides of the pier). The loads are applied through a 4-inch (102 mm) diameter pressure pipe which is connected to a standard control panel and load jack pump.

As a means of installing the load cell, a steel beam was welded to the top of the load cell and the pressure pipe was mounted along the web of the beam. A series of strain gages and telltales were used to instrument the drilled test piers at both projects. A generic layout of the instrumentation used at both projects is presented in Figure 1.

Cost savings for performing a load test with this device can be realized by minimizing construction to only the test pier itself along with the nominal investment for the load cell. This eliminates costs associated with construction of an above-ground dead weight or the drilling of additional piers for reaction. Other costs include instrumentation for the load test such as telltales, strain gages and other devices.

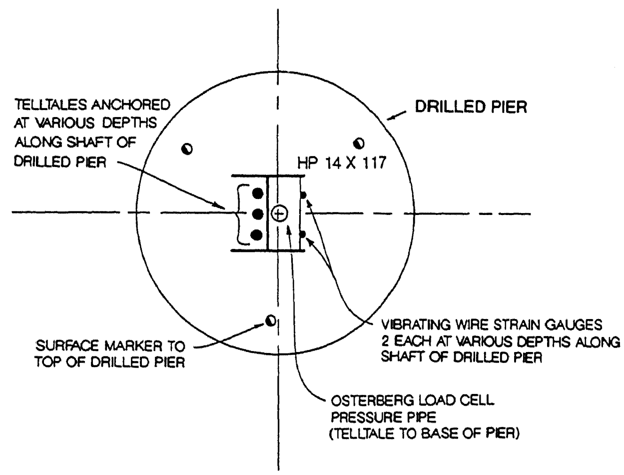

FIGURE 1
GENERIC LAYOUT OF INSTRUMENTATION AT TOP OF DRILLED PIER (TYPICAL BOTH PROJECTS)

CASE HISTORY I
OFFICE BUILDING
SOUTHFIELD, MICHIGAN

INTRODUCTION

The office building is part of a mixed used building complex in Southfield, Michigan. The office building is a steel framed structure with a full basement. Column loads ranged from about 500 to 2300 kips (2,200 to 10,200 kN).

The foundations for the office building consists of drilled piers with most piers having an enlarged base or bell. The bearing soils for the drilled piers consisted of a hard sandy clay till, commonly called hardpan. A one to three foot thick sand layer was encountered above the hardpan. Therefore, bells for drilled piers had to formed in the hardpan, rather than on top of the hardpan as is typically done is this area.

INVESTIGATIVE WORK AT THE SITE

A geotechnical investigation of the site was performed during the initial site planning and design phase. The general soil profile is shown on Figure 2.

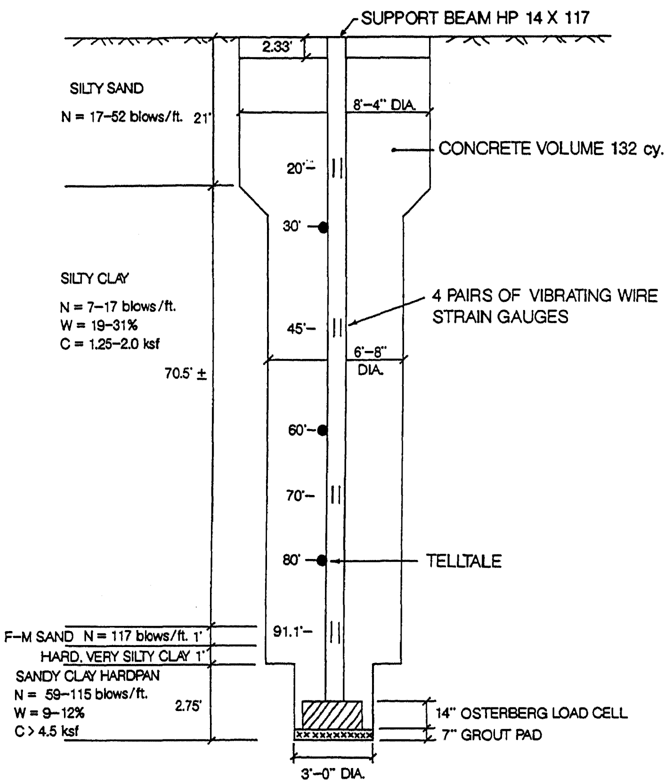

FIGURE 2

SCHEMATIC OF OFFICE BUILDING
DRILLED TEST PIER

The sand encountered above the hardpan was anticipated to cause some difficulties during the construction of the drilled shafts. Even though the sand layer was thin, groundwater seepage from the sand layer would likely necessitate the use of a full length temporary steel casing seated into the hardpan to seal off the groundwater. Therefore, straight shaft drilled piers with a bearing pressure of 30,000 psf (1,440 kPa) were recommended for the foundations in the original design. The drilled piers could then bear on top of the hardpan just below the sand layer.

Since straight shaft piers were recommend, the volume excavation and concrete required was relatively large. A supplemental investigation was then performed to increase the allowable bearing area so that drilled shafts with bells could be used. Using bells would allow smaller shafts and therefore provide significant savings in excavation and concrete.

The recommendation from the supplemental investigation was a higher bearing pressure of 60,000 psf (2,870 kPa) for the hardpan if settlements of 1 to 1.5 inches (25 to 38 mm) could be tolerated. In addition, a skin friction value of 500 psf (24 kPa) for the upper stiff to very stiff silty could also be used to resist the vertical loads.

The estimated settlement was within the tolerable range for the structure and the new recommended values for end bearing and skin friction were used for the design of the drilled shafts. With the higher allowable bearing pressure, smaller bearing areas could be used. Therefore, drilled shafts with bells formed in the hardpan were feasible. This change resulted in a significant cost savings for the project.

CONSTRUCTION OF THE DRILLED PIERS

Approximately 47 drilled piers were constructed with shafts ranging from 3 to 4.5 feet (0.9 to 1.4 m) in diameter and with bells up to 6.5 feet (2.0 m) in diameter. As anticipated, a temporary steel casing was required to seal off groundwater from the sand layer above the hardpan. The bells were formed within the hardpan below the steel casing. In some cases it was necessary to extend the piers deeper than designed due the difficulty in sealing off the temporary casing.

LOAD TEST BACKGROUND

After review of the soil information and the actual depths of the drilled piers there was some concern the design bearing pressure was too high for the production piers. Even though all of the drilled shafts were already constructed, it was decided a load test should be performed to verify the design bearing pressure and skin friction. In addition, the load test would provide additional information for future buildings in the complex.

INVESTIGATIVE PROCEDURES FOR THE LOAD TEST

Soil Boring

A soil boring was performed in the general area of the office building and confirmed the previous soil conditions at the site. Medium dense silty sand was encountered to depth of about feet (6.4 m). Stiff to very stiff silty clay was observed to a depth of about 93 feet (28.4 m). The driller reported a 1 foot thick layer of sand below the silty clay. Below the sand layer, a hardpan stratum was encountered to a depth of about 117 feet (35.7 m) which was underlain by a hard silty clay. Along the left side of

Figure 2, these strata along with pertinent geotechnical properties are shown.

Pressuremeter Tests

In addition to the field and laboratory testing, pressuremeter tests were performed at five levels along the length of the boring. One test was performed in the upper silty clay soils, with the remaining four tests performed in the hardpan. The pressuremeter tests indicated the hardpan became softer with depth at this location. This was not observed at the soil boring locations previously drilled within the building area.

CONSTRUCTION OF DRILLED TEST PIER

The drilled test pier was located about 250 feet (76.2 m) southeast of the office building. The configuration of the drilled test pier is presented in Figure 2. This figure is a cross-sectional view of the test pier along with the soil profile. The upper portion of the drilled shaft was 8'-4" (2.54 m) in diameter and extended to a depth of 21 feet (6.4 m). The main shaft of the test shaft was 6'-8" (2.03 m) in diameter and extended about 1 foot into the hardpan. A 3'-0" (0.9 m) diameter socket was then drilled about 2'-7" (0.79 m) into the hardpan.

Once the excavation was mechanically cleaned, 7 inches (178 mm) of grout was placed at the bottom of the socket, prior to installing the Osterberg load cell. The 4 inch (102 mm) diameter pressure pipe for the load cell was mounted along the web of a 94 foot (28.7 m) long HP14 x 117 beam welded to the top of the load cell. This beam was used for installation of the load cell into the drilled pier excavation and for orienting the tell tales and strain gages at pre-determined locations within the test pier. The specifics of the instrumentation are shown on Figures 1 and 2.

The concrete mix design consisted of a target strength of 4,000 psi (27,600 kPa). Based on the concrete cylinder test data, the seven-day test strengths exceeded the target strength.

LOAD TEST PROCEDURE

The load test procedure involved applying a seating load of 40 tons (356 kN) and then loading in 40 ton (356 kN) increments up to 200 tons (1780 kN). The load was reduced to the seating load of 40 tons (356 kN) and then reloaded up to 200 tons (1780 kN). Loads were then applied in 50 ton (445 kN) increments up to a maximum of 900 tons (8,000 kN). Each load increment was held for 15 minutes, except for the 200 ton (1,780 kN) and 400 ton (3,560 kN) load increments which were held for one hour. Deflection and strain gage readings were obtained as close to 1 minute, 5 minutes, and every 15 minutes after the full load was applied.

A maximum load of about 900 tons (8,000 kN) was held for about 4 minutes, and a maximum settlement of about 4.8 inches (121.9 mm) was measured at the bottom of the test caisson. At that time, the cell was unloaded in increments of 200 tons (1,780 kN) and the test was terminated.

LOAD TEST ANALYSIS/RECOMMENDATIONS

The results of the load test are presented in Figures 3 and 4.

End Bearing in Hardpan

Figure 3 shows the load/settlement response at the bottom of the pier. Hirany (1989) defines the bearing capacity for a drilled shafts as the bearing pressure which occurs at a settlement of 4% of the pier width. For the test pier, this would equal 1.4 inches (36 mm). For the load test, the observed load at a settlement of 1.4 inches (36 mm) was about 500 tons (4,450 kN). This corresponds to a bearing pressure of 141 ksf (6,750 kPa) for a 3 foot (0.9 m) diameter base. Using a factor of safety of 2, the allowable bearing pressure would be 70 ksf. At this is bearing pressure, the measured settlement was about 0.4 inches (10 mm).

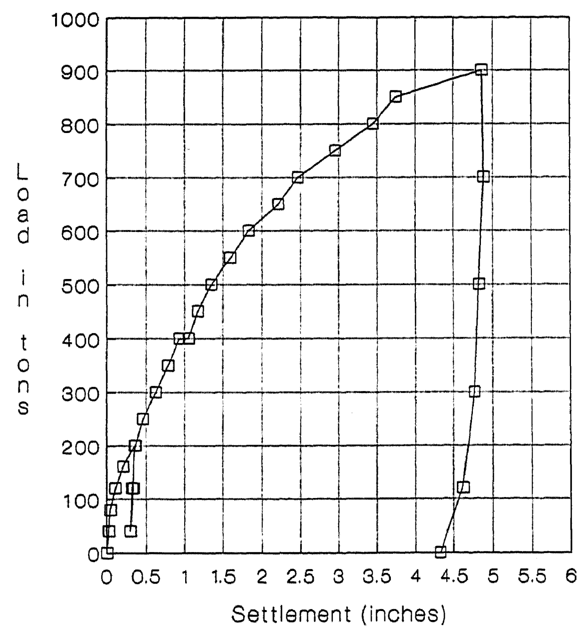

FIGURE 3
END BEARING LOAD-SETTLEMENT CURVE
OFFICE BUILDING

Skin Friction in Silty Clay

The load reduction curve in Figure 4 was obtained from the strain gauge data, based on the assumed values for the modulus of elasticity for concrete and steel. Approximately 10% of the load reached the 20 foot (6.1

m) depth level. Therefore, approximately 810 tons was carried in friction on the shaft below a depth of 20 feet (6.1 m). This corresponds to a unit friction value of about 700 psf (33,500 Pa). However, the top of the shaft moved less than 0.2 inches (5 mm). Ultimate skin friction is normally developed between movements of 0.25 and 0.5 inch (6.4 and 12.7 mm). Therefore, the ultimate skin friction for the silty clay is conservatively estimated at 800 psf (38,300 Pa).

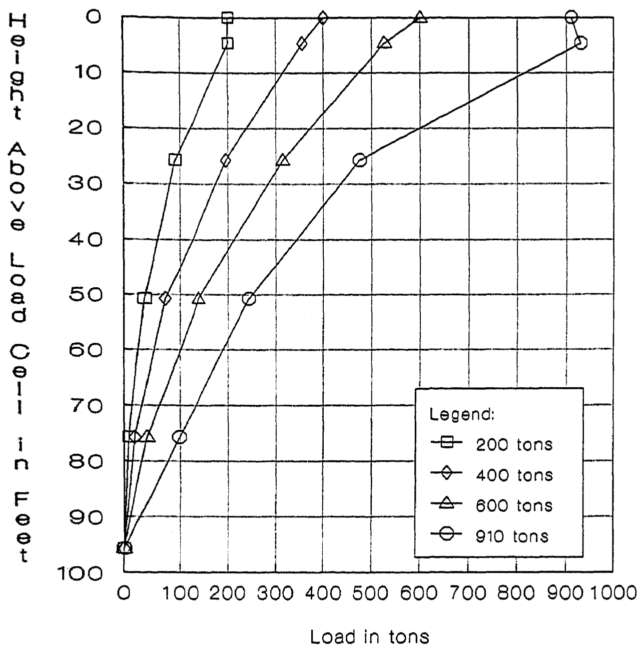

FIGURE 4
STRAIN GAGE DATA
OFFICE BUILDING

Comments on Existing Drilled Piers

In general, the load test verified the higher bearing pressure and skin friction of the upper silty clay for the drilled shaft design. However, some of the shafts were deeper than designed and could bear on the softer hardpan soils encountered at the test shaft location.

The settlement measured by the load test compared favorably with the predicted settlements using the pressuremeter data. Therefore, the pressuremeter information was also used evaluate the conditions for larger base diameters and deeper piers. Under the worse case conditions, settlements of up to 1.8 inches (46 mm) could be expected. However, the 1.8 inches (46 mm) of settlement assumes the full dead load plus live load will be applied to the shaft. Since at least some of the live loads are transitory, it is likely the maximum settlement under the worst case soil conditions will be less than 1.5 inches (38 mm).

CASE HISTORY II
VETERANS ADMINISTRATION MEDICAL CENTER
DETROIT, MICHIGAN

INTRODUCTION

The Veterans Administration site is located adjacent to the Detroit Medical Center, covering about 24 acres (97,100 square meters). The project involved the construction of a four story Diagnostic Unit, a Patient Resident Building consisting of five and seven story wings, two four-story parking decks and an Energy Center building. In general, these structures were constructed below the final exterior grades. Column loads varied from 600 to 6,000 kips (2,670 to 26,690 kN) with some uplift loads as high as 300 kips (1,335 kN).

A total of about 550 drilled piers were installed at the site during the summer of 1992. These piers were generally straight-shaft type, bearing directly on the hardpan, either at the top surface or socketed into the hardpan. At some locations, belled piers were constructed on top of the hardpan.

PREVIOUS INVESTIGATIVE WORK AT THE SITE

Prior to conducting the load test at the site, a preliminary and supplemental geotechnical investigation were conducted to ascertain the soil and groundwater conditions. The general soil properties of each layer are shown on the left side of Figure 5. The respective layer thickness shown in that figure represent those from the test boring drilled for the load test evaluation. In the boring drilled for the load test, the sand layer immediately above the hardpan layer, typically observed in the deep borings drilled for the preliminary and supplemental investigations, was not encountered.

The most notable groundwater conditions were those from the sand layer above the hardpan which stabilized between elevations 113 (34.4 m) and 132 feet (34.4 and 40.2 m)> This indicated the groundwater was under about 107 to 127 feet (32.6 to 38.7 m) of static head.

LOAD TEST BACKGROUND DATA

A supplemental geotechnical investigation, including a load test, was conducted on a test pier constructed at the site. The purpose of this investigation was to obtain further geotechnical information at the site as a means of optimizing the design criteria for the proposed drilled pier foundations. At the time the load test was performed, belled drilled piers, extending into the hardpan material, were being planned for providing support of the new structure. The drilled shafts were designed based on the City of Detroit presumptive end bearing pressure of 50 ksf (2,390 kPa). No contribution for side friction was included in the design at that time.

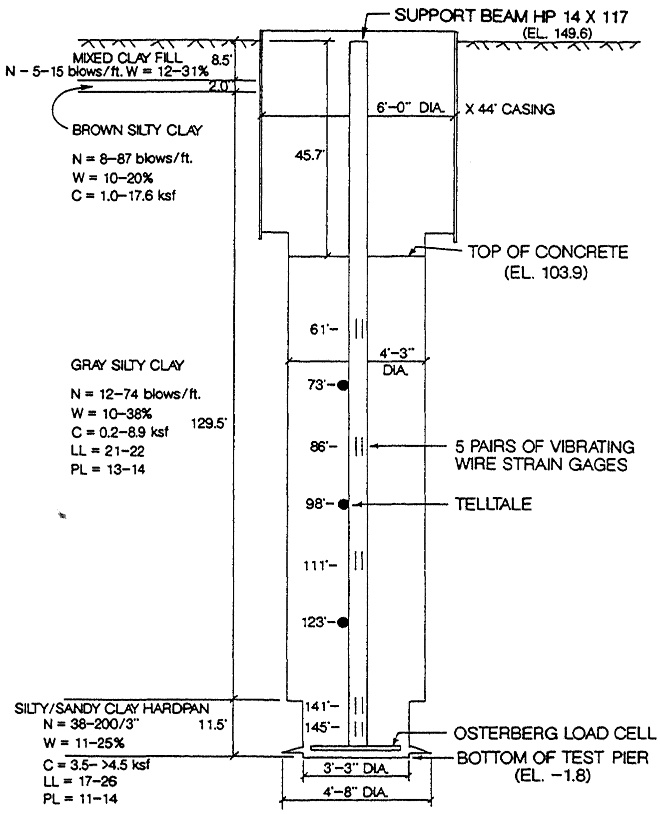

FIGURE 5
SCHEMATIC OF VETERANS ADMINISTRATION DRILLED TEST PIER

We recommended belling in the hardpan soil be avoided due to the higher risks and costs associated with this type of construction. As a consequence, either larger straight shaft piers would be required or the design criteria must be adjusted upward to accommodate the high column loads. Since the larger straight shaft alternative would also result in substantial additional costs to the project, we recommended the Veterans Administration invest in a load test to develop site specific design criteria. It was postulated the end bearing pressure could be increased in addition to utilizing side friction in the design to downsize the piers and reduce construction costs.

INVESTIGATIVE PROCEDURES FOR THE LOAD TEST

Soil Boring

Prior to constructing the test pier, a soil boring was drilled about 10 feet (3.1 m) from the test pier. The test boring confirmed previously established soil conditions at the site. The boring exhibited 8.5 feet (2.6 m) of mixed fill material overlying 3.5 feet (1.1 m) of brown silty clay and 130 feet of gray silty clay. Natural clay hardpan was encountered below the clays and it was penetrated 33 feet (10.1 m) to the explored depth of the boring. The sand layer above the hardpan was not encountered in this boring.

Pressuremeter Tests

In addition to the field drilling and laboratory testing, field pressuremeter tests were performed in the test boring. Four pressuremeter tests were conducted in the hardpan layer at different depths below the ground surface. The specific depth of each of the pressuremeter test location was selected based on the results of the soil boring which was representative of the hardpan material encountered. By determining the characteristics of the hardpan, it was believed a better estimate of the load-settlement characteristics could be determined prior to conducting the load test.

CONSTRUCTION OF DRILLED TEST PIER

The drilled test pier was constructed along the east side of the site. The configuration of the test pier is presented in Figures 1 and 5. Figure 5 is a cross-sectional view of the test pier along with the soil profile on the left side. Since the top of the proposed piers were to be established near elevation 111 (about 40 feet or 12.2 m below the ground surface), a 44'-3" foot long by 6 foot (13.5 m by 1.8 m) diameter temporary casing was installed near that level to isolate the pier from the skin friction in that zone. The main shaft of the test pier was extended just into the top of the hardpan and a diameter of 4'-3" feet (1.3 m) was measured in the field. A 3 foot (1.0 m) diameter socket was then drilled about 11 feet (3.4 m) into the hardpan. A 4.67 foot (1.4 m) diameter bell was then excavated.

Once the excavation was mechanically cleaned, a concrete pad was placed in the bell section, prior to installing the Osterberg load cell. The 4-inch (102 mm) diameter pressure pipe, for the load cell, was mounted along the web of a 150-foot (45.7 m) long HP14 x 117 beam welded to the load cell. Telltales and strain gages were oriented at pre-determined locations within the test pier. The specifics of the instrumentation are shown on Figure 5.

Once the load cell was placed, we observed the location of the bottom of the cell was about 0.7 to 0.8 feet (0.2 m) above the bottom of the bell. Details of the bottom of the pier are shown in Figure 6. The excavation was then filled with concrete to complete the construction. Three reinforcing bars, each 52 feet (15.9 m) long, were installed at the top of the test pier in a triangular pattern around the HP beam.

The concrete mix design consisted of a target strength of 4,000 psi (27,580 kPa). Based on the concrete cylinder test data, the three-day test strengths exceeded the target strength.

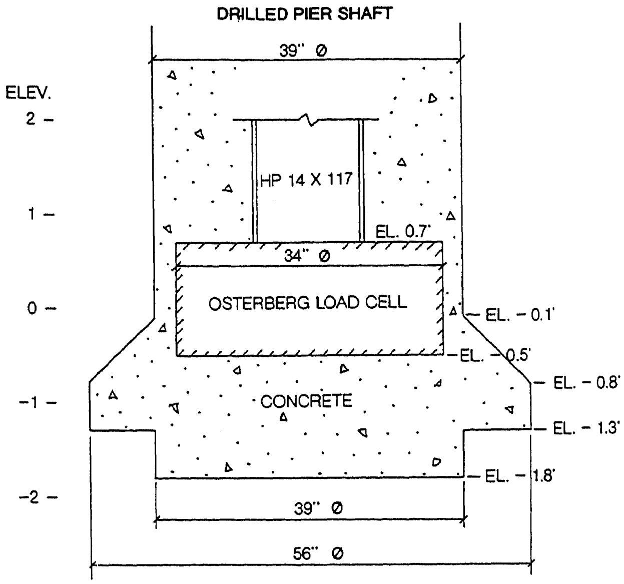

FIGURE 6

SCHEMATIC OF BOTTOM SECTION OF VETERANS ADMINISTRATION DRILLED TEST PIER

LOAD TEST PROCEDURE

The load test procedure involved applying seating loads of 25, 75 and 125 tons (222, 667 and 1,112.1 kN), backing off to 25 tons (222 kN) and then applying loads in 50 ton (445 kN) increments up to a maximum of 650 tons (5,783 kN). Each load increment was held for 15 minutes, except for the 350 to 500 ton (3,110 to 4,450 kN) loads, which were held for one hour, to obtain extended creep readings. Deflection and strain gage readings were obtained as close to 1 minute, 5 minutes, 10 minutes and 15 minutes after the full load was applied.

When the load was increased to about 650 tons (5,780 kN), the settlement was more than twice the "failure level" defined by Hirany (1989) as 4 percent of the end bearing diameter, or 1.56 inches (40 mm). For this particular test, we assumed the effective bearing area is based on the 3.25 foot (1.0 m) diameter pilot hole in the hardpan and not the 4.67 foot (1.42 m) diameter bell. This is due to the small 0.7 to 0.8 foot (0.2 m) distance between the bottom of the load cell and bottom of the bell. Under these high loads, we believed the concrete in the bell cracked and that the outer edges of the bell did not effectively transmit load to the hardpan. See Figure 6 for details.

Since the upward shaft movement was relatively small, the maximum friction capacity had not been obtained and we continued applying load until either the load could not be developed or the piston travel capacity was reached for the load cell (maximum between 5 and 6 inches, or 127 mm and 152 mm)). A maximum load of about 689 tons (6,130 kN) was held for about 7 minutes, and a maximum settlement of 5.06 inches (128.5 mm) was measured at the bottom of the test pier. At that time, the cell was unloaded in increments of 175 tons (1,560 kN) and the test was terminated.

LOAD TEST ANALYSIS/RECOMMENDATIONS

Our review of the drilled pier load test data was confirmed by the pressuremeter results for the hardpan material. However, to simplify the presentation of our findings, the following discussions will focus on the load test results. Based on the maximum applied load of 689 tons (6,130 kN), the maximum pressure at the tip of the test pier is computed to be about 166 ksf. Furthermore, design values for skin friction in the hardpan and adhesion for the upper clays were incorporated into the design. The results of the load test are presented in Figures 7 and 8 for the end bearing load vs. settlement curve and strain gage data, respectively.

End Bearing in Hardpan

Based on the load-settlement data for end bearing in Figure 7, it appears an ultimate failure load of about 600 tons (5,340 kN) is appropriate for the test pier. This results in a back-figured applied pressure of 145 ksf. For design purposes, a safety factor of 2 is applied to these values. Therefore, we recommended a design pressure of 70 ksf (3,350 kPa) for end bearing in the hardpan be used. Based on the load-settlement data, at the 70 ksf (3,350 kPa) pressure or 300 ton (2,670 kN) load in the test pier, the settlement was about 0.5 inches (13 mm).

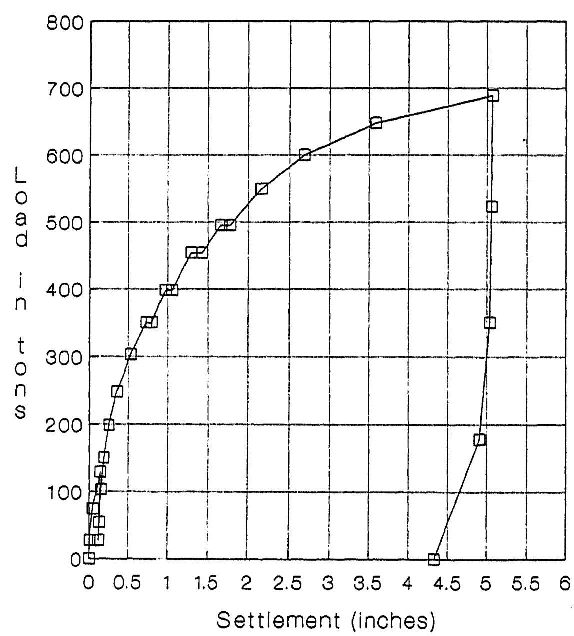

FIGURE 7
END BEARING LOAD-SETTLEMENT CURVE
VETERANS ADMINISTRATION MEDICAL CENTER

Skin Friction in Hardpan and Adhesion in Upper Clays

Concerning the skin friction in the hardpan and adhesion for the upper clays, two pieces of information were analyzed. First, the strain gage data (Figure 8), was produced on the basis of applied loads from the load cell as noted by the respective curves. The computed loads from the strain gage data, are based on assumed values of the modulus of elasticity for concrete and steel.

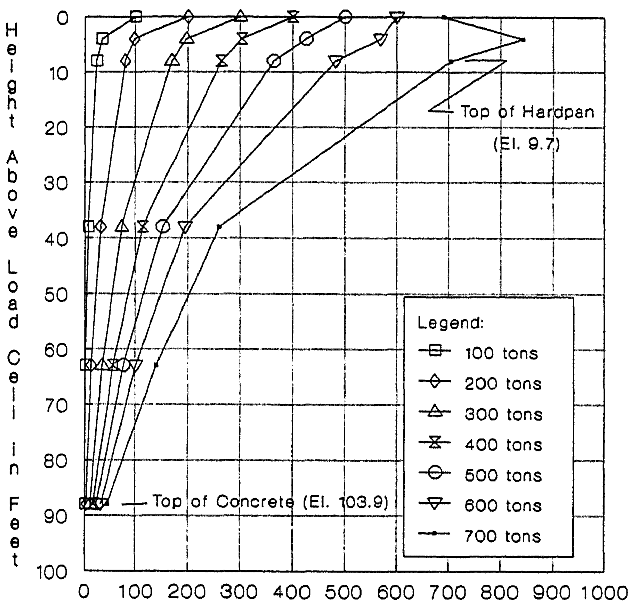

FIGURE 8
STRAIN GAGE DATA
VETERANS ADMINISTRATION MEDICAL CENTER

Initially, one would think the upper clays in addition to the hardpan contributed to the upward resistance of the test pier movement, since the strain gages at greater heights above the load cell indicated loads as high as about 38 percent of the applied load. However, the tell tale data suggests the hardpan resisted all of the upward loading since a maximum movement of about 0.063 inches (1.60 mm) of movement occurred at the tell tale located closest to the load cell. Normally, ultimate skin friction is developed with 0.25 to 0.5 inches (6.4 to 12.7 mm) of movement. Therefore, we judged the hardpan resisted the maximum loading of 700 tons with little upward movement. On this basis, for the 10-foot (3.1 m) socket in the pier, the back-figured skin friction is 13.5 ksf (646 kPa) and we recommended 15 ksf (718 kPa) be used in the design.

For the resistance of the upper clays in the profile, it appears viable to assign a nominal design adhesion value. As indicated in the previous paragraph, the ultimate skin friction in clays is developed with about 0.25 to 0.5 inch (6.4 to 12.7 mm) of movement. Since the end bearing value of 70 ksf (3,351.6 kPa) is predicated on about 0.5 inch (12.7 mm) of movement at the tip of the pier, we believe full ultimate skin friction in the upper clays can be developed. Based on the accumulated shear strength data for the project and information developed from a previous load test in similar materials, we recommended a design adhesion value of 0.5 ksf (24 kPa) be used in the clays below elevation 111 feet (33.8 m).

Other Design Considerations For Construction

For design purposes, it was possible to use drilled piers socketed into the hardpan for the project. However, the top of the hardpan surface varied with location and the presence of the lower strength hardpan at this site required changes in the drilled pier configuration to be made during construction.

CONCLUSIONS

The versatility of the Osterberg load cell allows the professional to separate the end bearing and side friction components of drilled piers for design. At both of the Detroit area projects, described in this paper, the authors attempted to configure the test piers in such a way to permit failure in both end bearing and side friction. Unfortunately, both test piers failed in end bearing and the ultimate frictional resistance in the hardpan and overlying silty clay layers were not fully mobilized. Furthermore, the nature of the hardpan below the test piers at both sites indicated zones of lower strength and higher moisture content. Based on the load test results, it appears a conservative end bearing pressure of 70 ksf may be used with the possibility of much higher pressures if these lower strength-higher moisture content zones are not present.

REFERENCES

Hirany, A. and Kulhawy, F.H. (1989), "Interpretation of Load Tests on Drilled Shafts Part 1: Axial Compression", Foundation Engineering: Current Principles and Practices, Proceedings of the 1989 Foundation Engineering Congress, Evanston, Illinois, June 25-29, pp. 1132-1149.

Osterberg, J. (1989), "New Device for Load Testing Driven Piles and Drilled Shafts Separates Friction and End Bearing", Proceedings of the International Conference on Piling and Deep Foundations, London, May 15-18, pp. 421-427.

Osterberg, J. (1992), "Rock Socket Friction in Drilled Shafts: It's Greater Than You Think", Foundation Drilling, Association of Drilled Shaft Contractors, January, pp. 15-21.

Thompson, D.E., Erikson, C.M. and Smith, J.D. (undated), "Load Testing of Deep Foundations Using the Osterberg Load Cell", Paper prepared by Haley & Aldrich, Inc., 18 pages.