Pile Bearing in Burlington Limestone

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Three field load tests of drilled shafts socketed in Burlington limestone were conducted using the Osterberg load cell. The objective of these tests was to compare the shaft capacities obtained from the field load tests with capacities predicted using analytical methods and with typical presumptive design capacities. It was believed that the actual capacities of the drilled shafts would be considerably greater than the capacities predicted from presumptive bearing capacity values. Based on the results of this testing, the following conclusions were drawn. Observed values of side resistance are comparable to the predicted values obtained from empirical relationships. Observed values of end bearing pressure greatly exceed the presumptive values of allowable bearing capacity commonly used for the design of shafts bearing on Burlington limestone. The test shafts were not failed in end bearing and it is believed that the ultimate end bearing pressures would significantly exceed the observed end bearing pressures. The actual factors of safety of shafts in Burlington limestone that are designed for end bearing only, using typical presumptive end bearing capacities, will exceed 6. Side resistance will carry a large portion of the load and for service loads, the entire load may be carried by side friction. Key words: foundations, drilled shafts, Osterberg cell, load test.

INTRODUCTION

It is common engineering practice to design rock-socketed drilled shafts for end bearing only, based on conservative presumptive values of allowable bearing capacity. For example, for the Burlington limestone studied in this paper, a typical allowable bearing capacity is 1914 kPa (40,000 psf). The use of conservative values is due in part to the lack of full scale field load test data that would allow for the validation of less conservative design procedures. Often, site investigations terminate at auger refusal, in which case only the location of the rock is known and very little is known about rock strength. Further, the difficulty and cost of performing full scale load tests of drilled shafts in rock, limits the amount of data available for design procedure validation. Recently, the development of the Osterberg load cell provided a more economical means for conducting load tests. To date, the Osterberg load cell has not been used extensively in Mid-America and particularly it has not been used extensively in limestone.

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OSTERBERG LOAD CELL

The Osterberg load cell was developed and patented by Dr. Jorj Osterberg (*3*). The Osterberg load cell is a static load testing device for shafts and piles. An Osterberg cell load test uses an especially designed hydraulic to create pressures in excess of 55 MPa (8,000 psi) at the bottom of the shaft, loading the pile or shaft in end bearing and upward side resistance.

The Osterberg load cell is lowered into the shaft via the reinforcing cage or if no reinforcement cage is used, a small I-beam or channel can be used to place the load cell. The hydraulic lines and telltale rod casings are also attached to the reinforcement cage. The telltale rods allow for the measurement of the movement of the bottom and the top of the cell. These movements and the movement of the top of the shaft or pile are measured using dial gages supported by an independent reference beam.

The Osterberg cell is pressurized using a compressed air driven pump with diluted automotive antifreeze as the hydraulic fluid. The soil and/or rock surrounding the shaft or pile provides the reaction for the load test. As the cell is pressurized, the bottom of the cell moves downward, testing end bearing, while the top of the cell moves upward, testing side resistance. Schmertmann (The Bottom-Up, Osterberg Cell Method for Static Testing of Shafts and Piles, *unpublished data*) fully discusses the advantages and disadvantages of the use of the use of the Osterberg load cell.

TEST METHODS AND PROCEDURES

Shaft Excavation

Hayes Drilling Inc. of Kansas City, Missouri began shaft construction on December 9, 1996. Three shafts were excavated using a truck-mounted rotary drill. A 45.72 cm (18 in.) diameter auger bit with carbide cutting teeth was used to excavate the overburden as well as the rock socket. Water was used as lubrication during the drilling process and to facilitate the removal of the rock cuttings. The base of the socket was cleaned by rapidly spinning the auger bit after the addition of water and then lifting out the rock cuttings.

Osterberg Cell Assembly and Placement

The Osterberg cells used in the base of the three shafts were 33 cm (13 in.) in diameter and approximately 31.75 cm (12.5 in.) high. The cells had a maximum load producing capability of 4000 kN (450 tons). Figure 1 illustrates the Osterberg cell assembly.

After the completion of drilling, a small seating layer of concrete was placed by free fall into the base of the shaft. The Osterberg

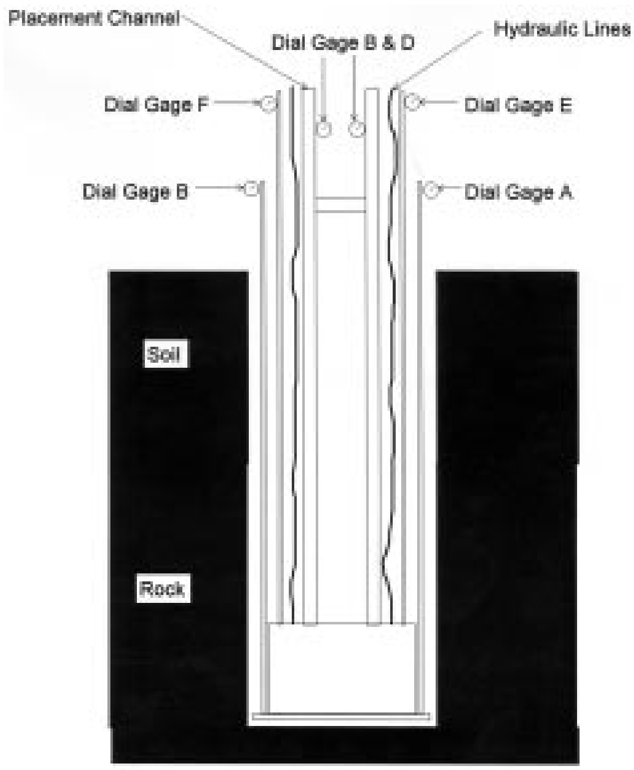

FIGURE 1 Osterberg load cell and movement measurement schematic.

cell base plate was greased to ensure no concrete adhesion. The cell was then lowered into the shaft and seated onto the base layer of concrete. The remaining concrete was then placed by free fall into the shaft. Three concrete test cylinders were made for each shaft so that the strength of the concrete could be measured. The concrete was allowed to cure for 6 days before the load test was performed. The average concrete strength at the time of shaft testing was 47.2 Mpa (5300 psi).

Load Test Procedure

A steel channel reference beam was placed near the drilled shaft assembly. Six digital dial gages were attached to the reference beam or steel channel by magnetic stands. The dial gages were designated A through F (see Figure 1). Machined steel telltale rods were inserted into the telltale casings. Dial gages A and B measured the downward displacement of the base plate telltale rods and dial gages C and D measured the upward displacement of the top of shaft. Dial gages E and F were attached to the channel frame and measured the displacement between the top of cell telltale rods and the top of the shaft or otherwise stated they measured the compression of the shaft.

The Osterberg cell was pressurized in increments of approximately 3445 kPa (500 psi). The pressure was held at each loading increment for a total of 4 minutes. The load increments were increased until side friction shear failure occurred.

RESULTS AND DISCUSSION

Site Investigation

The initial site investigation consisted of collecting eight previous subsurface investigations that were performed in the general vicinity of the three research shafts. These investigations were performed from 1988 to 1995 by Engineering Surveys and Services of Columbia, Missouri for the purpose of new construction.

The overburden consisted of mostly glacial drift. This ranged in depth from zero to over 6 m (20 ft.). The drift consisted of sandy clay, sandy silty clay, gravelly clay and is sometimes underlain by Pennsylvanian shales. These materials are underlain by massive Mississippian limestone bedrock.

Burlington limestone bedrock depths in the area range between 1.8 and 12.8 m (6 and 42 feet). The surface of the limestone is irregular and weathered in some areas. The weathered layer varied in thickness from a few centimeters to over a meter.

Three unconfined compression strength tests of Burlington limestone core samples show a 43.6 MPa (6,336 psi), 73.8 MPa (10,718 psi), and 64.7 MPa (9,395 psi) rock strength. Four core samples provided rock quality designations (RQD) and percent recoveries These include a 90 percent recovery with a 78 RQD, a 100 percent recovery with an 80 RQD, a 100 percent recovery with a 100 RQD, and a 100 percent recovery with an 85 RQD.

During the drilling of the shafts glacial till was found at the surface. It was predominantly clay with some silt, sand and gravel. No shale was found during the drilling process. However, a thin layer of weathered limestone was encountered on top of the limestone bedrock.

After completion of the shafts, a feeler gage was used to scrape the sides of the socket in order to find seams or fractures. Small seams were found in shafts 1 and 2 but no seams were detected in shaft 3. No ground water was encountered in any of the shafts. The depth to limestone was 4.12 m (13.7 ft), 4.02 m (13.2 ft), and 3.77 m (12.4 ft) for shafts #1, #2 and #3, respectively.

Downward End Bearing and Upward Side Resistance Load Movement Curves

The downward end bearing load movement curves were obtained directly from dial gages A and B, which measured the difference between the displacement of the reference beam and the telltale rods extending to the base of the cell. The upward side resistance movement was obtained directly from dial gages C and D, which measured the difference between the displacement of the reference beam and the top of the shaft. The side resistance load is the net load, calculated by subtracting the weight of the shaft from the cell load. The loads for the downward end bearing movement are the cell loads.

Shafts 1 and 3 were loaded until side resistance failure occurred. Shaft 2 was initially loaded to about 1000 kN (120 tons) and then unloaded due to a an equipment malfunction in the hydraulic pump. Shaft 2 was subsequently reloaded until side resistance failure occurred. Figure 2 shows the Osterberg cell load movement curves for shaft 3. The upward shear movement curve is typical of side resistance failure. Side resistance failure occurred at 3500, 1500

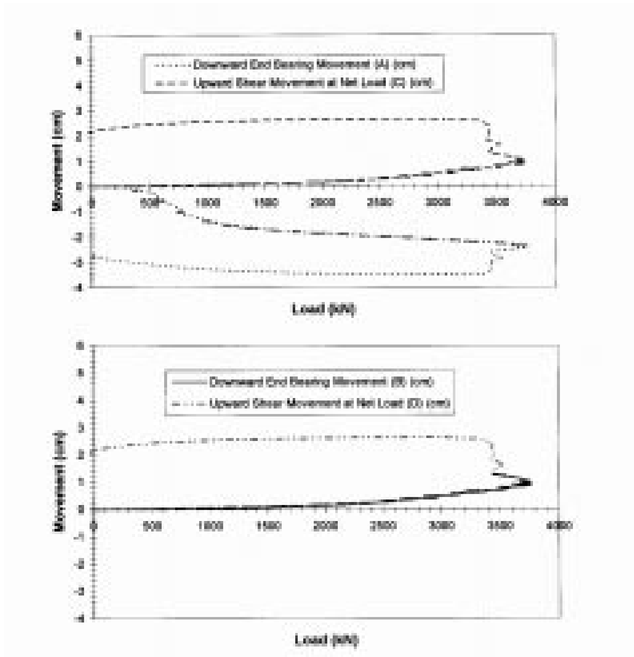

FIGURE 2 Osterberg cell load movement curves for shaft #3.

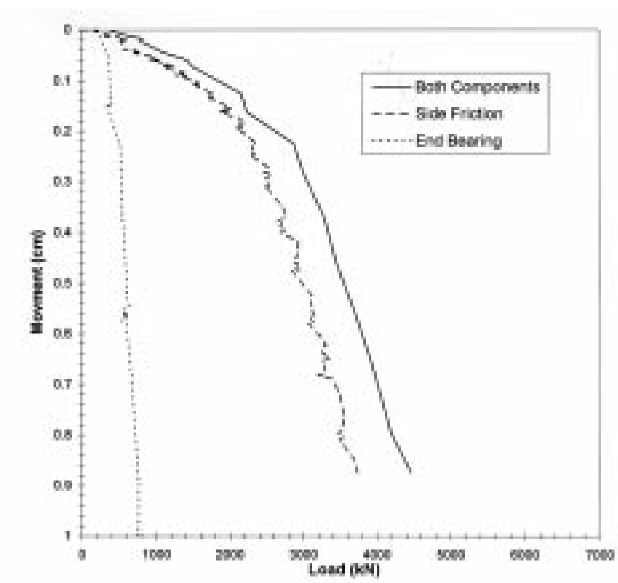

FIGURE 3 Equivalent load movement curve for shaft #3.

and 3800 kN for shafts 1, 2, and 3 respectively. The downward end bearing movement curve show some interesting anomalies. For shaft 3, it appears that the dial gauge B telltale casing became plugged and as a result the telltale rod did not move down with the bottom of the load cell, but rather up with the shaft. Finally, the downward end bearing movements curve shows continuing downward displacement of the load cell after side resistance failure has occurred. This is possible only if simultaneous end bearing and side resistance failures occur; which seems highly unlikely. It most likely indicates that after side resistance failure, ground movement at the surface raised the elevation of the reference beam.

Reconstructed, Equivalent Top Load Movement Curve

Reconstructed, equivalent top load movement curves can be developed by adding side resistance movement data and end bearing movement data. Goodwin (Bi-Directional Load Testing of Shafts to 6000 Tons, *unpublished data*) indicates that the reconstructed curves will represent the load movement of a shaft loaded in the conventional field load test manner if 1) the end bearing load movement in a conventionally loaded shaft is the same as the load movement curve developed by the bottom of the Osterberg cell, 2) the upward side resistance movement curve for the Osterberg cell test is the same as the downward side resistance movement in a conventionally top loaded test and 3) the compression of the shaft is considered negligible and the shaft is assumed rigid.

Equivalent load movement curves were reconstructed up to the maximum test load of 6524 kN (733 tons) for shaft 3. The reconstructed equivalent top load movement curve for shaft 3 is presented in Figure 3.

Observed End Bearing Pressures and Side Resistance

The maximum side resistance of the three shafts was reached and therefore can be compared directly with predicted side resistance values. Due to the limitations of the bi-directional loading of the Osterberg cell the maximum capacity in end bearing was not reached.

Side resistance is typically predicted using empirical relationships between side resistance and either concrete or rock strength. Williams et al. (*7*) and Rowe and Armitage (*8*) provide relationships developed for used with limestone rock. The predicted side resistance capacities were calculated using a concrete strength of 47.2 Mpa (5300 psi) rather than the higher unconfined compressive strength of the rock. The lower value should be used when calculating predicted side resistance because side resistance failure will occur in the lower strength material. The predicted side resistance using the Williams relationship is 1550 kPa (225 psi) and using the Rowe and Armitage relationship it is 1252 kPa (181 psi). The observed side resistance values for shafts 1, 2 and 3 respectively are 2343 kPa (340 psi), 916 kPa (133 psi), and 2278 kPa (330 psi).

The predicted values of side resistance are significantly lower than the values of side resistance observed from shafts 1 and 3. The side resistance value observed from shaft 2 is lower than predicted values. Based on this data, the authors conclude that the empirical relationships are adequate if typical design factors of safety are used.

Due to the limitations of the Osterberg cell it was not possible to reach the maximum end bearing capacity. Since the Osterberg cell loads the shaft from the bottom, the applied load can only be as large as the load bearing mechanism with the lowest capacity. In the case of shafts 1, 2, and 3 it was side resistance. The observed end bearing pressures at termination of testing were 21.4 MPa (3112 psi), 9.1 MPa (1320 psi) and 22.9 MPa (3325 psi) for shafts 1, 2, and 3 respectively.

CONCLUSIONS

It is common engineering practice to design rock-socketed drilled shafts for end bearing only, based on conservative presumptive values of allowable bearing capacity. For the Burlington limestone studied in this paper, a typical allowable bearing capacity is 1914 kPa (40,000 psf).

It is also typical to specify that shafts be socketed 0.61 m (2 ft) into sound rock. The conservatism of this approach to design can be illustrated with the following example.

Given a design shaft load of 2670 kN (300 tons) and an allowable end bearing pressure of 1914 kPa, the shaft would have a design diameter of 1.37 m (54 in.). Using the lowest observed value of side resistance, 916 kPa (133 psi), the side resistance capacity of the shaft would be 2409 kN (270 tons). Based on the lowest observed value of end bearing pressure, 9.1 MPa (1320 psi) the end bearing capacity would be 13,448 kN (1511 tons) and probably much larger. Therefore, particularly at service loads, the shaft load would be carried almost entirely by side resistance and the actual factor of safety would be greater than 6, possibly much greater.

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REFERENCES

1. Kiehne, C. Full Scale Load Tests of Rock Socketed Drilled Shafts. Thesis completed in partial fulfillment of the degree Master of Science at the University of Missouri-Columbia, Missouri, 1997, 129 pages.
2. Rosenberg, P., and N. L. Journeaux. Friction and End Bearing Tests on Bedrock for High Capacity Socket Design. *Canadian Geotechnical Journal*, Vol.13, 1976, pp. 324-333.
3. Osterberg, J. O., and S. F. Pepper. A New Simplified Method for Load Testing Drilled Shafts. *Foundation Drilling*, Association of Drilled Shaft Contractors, August 1984, pp. 9-11.
4. Unklesbay, A. G. *Geology of Boone County Missouri*, Vol. 33, State of Missouri, Jefferson City, Missouri, 1952.
5. Williams, A. F., Johnston, I. W., and I.B. Donald. The Design of Socketed Piles in Weak Rock, *Proceedings of the International Conference on Structural Foundations on Rock*, Sydney, Australia, 1980, pp. 327-347, A. A. Balkema, Rotterdam, Netherlands.
6. Rowe, R. K., and H. H. Armitage. Theoretical Solutions for Axial Deformation of Drilled Shafts in Rock. *Canadian Geotechnical Journal*, Vol. 24, pp. 114-125.