CASE HISTORY: MULTIPLE AXIAL STATNAMIC TESTS ON A DRILLED SHAFT EMBEDDED IN SHALE

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Two new railroad bridges in Kansas City, Missouri are founded on drilled shafts embedded into the soil and underlying shale. The uncertainty arising from the non-homogeneous, anisotropic nature of most shales, and the variability of shales across the U.S., makes it a difficult material to assign appropriate design properties. As was the case on this project, the uncertainty of foundation performance in shale can lead to overly conservative designs, particularly with drilled shaft foundations. An extensive Statnamic axial testing program was performed to verify that the load-settlement behavior of the drilled shafts would be acceptable for the intended railroad traffic. Axial Statnamic testing was applied in three increasing cycles of 927 kips, 1007 kips, and 3177 kips, respectively. The maximum movements recorded at the shaft head during each of the loadings were 0.052 inches, 0.062 inches, and 0.257 inches, respectively. The loads applied in each of the respective Statnamic tests were not sufficient to reach the capacity of the shaft; so direct comparison of measured and calculated capacities is not possible. However, evaluation of the estimated design capacity can be accomplished based on results from the Statnamic testing program by utilizing normalized load transfer relations presented by O'Neill and Reese (1999). Such comparisons necessarily require *extrapolation* of the data measured in the Statnamic tests following typical load-displacement response. Results indicate the Kulhawy and Phoon (1993) method for estimating side resistance in Pleasanton shale rock sockets, assuming a smooth socket, appears to agree well with observed behavior during Statnamic testing. Accordingly, the O'Neill and Reese (1999) method for estimating the toe resistance in intermediate geomaterials, assuming cohesive rock with Rock Quality Designation (RQD) between 70 and 100, appears to agree well with observed behavior during Statnamic testing. Finally, based on Statnamic testing at varying load magnitudes and normalized curves developed by O'Neill and Reese (1999), the contribution of side resistance to the capacity of a deep foundation can be determined assuming no load is transferred to the base of the foundation during lower magnitude testing. Furthermore, higher magnitude testing can provide a means to evaluate the accuracy of design end bearing calculations.

Introduction

The presence of shale at relatively shallow depths (less than 100 ft) exists extensively across the United States and elsewhere. As a result, many of the foundation schemes used today either bear on, or in, shale. However, the non-homogeneous, anisotropic nature of most shales, and the variability of shales across the U.S., makes it a difficult material to assign appropriate design properties. This uncertainty can lead to overly conservative designs, particularly with drilled shaft foundations. It is important for the geotechnical community to collect and analyze as much data as possible on actual performance of drilled shafts constructed in shale in order to properly design shale-embedded drilled shafts in the future.

A small portion of a flood control project in Kansas City, Missouri requires construction of two new railroad bridges across the Blue River. The new bridge bents are to be founded on drilled shafts. The shafts will be embedded into the soil and underlying shale at varying lengths, depending on the structural requirements and stratigraphy at each bent location. An extensive Statnamic axial testing program was performed to verify that the load-settlement behavior of the drilled shafts will be acceptable for the intended railroad traffic. This case history documents the behavior of a single drilled shaft constructed in shale and exposed to three independent Statnamic load tests of varying magnitude. However, the loads applied in each of the respective Statnamic tests were not sufficient to reach the capacity of the shaft. Direct comparison of measured and calculated capacities is therefore not possible. However, evaluation of the estimated design capacity can be accomplished based on results from the Statnamic testing program by utilizing normalized load transfer relations presented by the O'Neill and Reese (1999). Such comparisons necessarily require *extrapolation* of the data measured in the Statnamic tests following typical load-displacement response. Presented in this case history is a description of the design and construction process, a description and characterization of the subsurface materials that were encountered during both the site exploration and the drilled shaft excavation, a description of the Statnamic testing procedure, and the results and conclusions of the testing program.

Drilled Shaft Construction

The tested foundation, shown graphically in Figure 1, had a total embedded length of 61 feet. The upper 48 feet of the shaft was 72 inches in diameter and had a permanent steel casing that allowed drilling to be performed dry and provides considerable structural capacity to the finished shaft. The steel casing was advanced via vibration through the overburden soil prior to excavating for the drilled shaft. Upon attaining the depth of the shale with the drilled shaft excavation, the steel casing was rotated using the auger Kelly bar such that it penetrated approximately 4.5 feet into the weathered portion of the shale. This was considered necessary to prevent groundwater from entering the excavation at the soil/shale interface. It was successful as no water was noticed in the drilled shaft upon excavating the shale with the auger, despite being well below the measured groundwater depth. The lower 13 feet of the shaft below the casing was a 66-inch diameter socket into the underlying competent shale bedrock. The reinforcing cage consisted of 34 No. 11 longitudinal bars oriented vertically and No. 5 shear hoops spaced 12 inches on center. The concrete had a design strength of 5000 lb/in^2. The permanent steel casing had a wall thickness of 0.5 inches.

Subsurface Description

The general stratigraphy of the site consists of five different soil strata. Beginning at the existing ground surface, Stratum I is an approximately 11-ft thick, soft to stiff, low-plasticity clay. Stratum II is an approximately 19-ft thick layer of very soft to medium low-plasticity clay. Beneath Stratum II is an approximately 12-ft thick layer of soft to stiff low-plasticity clay that makes up Stratum III. While all soils in Strata I, II and III classify as CL because of their similar Atterberg limits, their behavior is quite different as a result of different geologic age and environmental exposure. Stratum II is assumed to be normally consolidated, as can be inferred based on the high moisture content (very near the liquid limit) and low Standard Penetration test blow counts. In the absence of environmental conditions, Strata I and II would likely be considered as one stratum, however, Stratum I exists and is different than Stratum II as a result of dessication. Stratum III, although similar to Stratum II with respect to index properties, is assumed to be overconsolidated. Stratum IV consists of an approximately 1.5-ft thick layer of dense gravel-sand-silt mixtures. Stratum V is composed of shale and is divided into two sub-strata. The shale is from the Pennsylvanian System, Missourian Series, Pleasanton Group. The upper shale sub-stratum is weathered, whereas the lower sub-stratum is intact. Measured and assumed properties of the soil and rock are shown graphically in Figure 1.

The ground water level was measured in a borehole left open overnight and was at a depth of approximately 8 feet below the ground surface. The water level in the area fluctuates with the water level in the Blue River. During the excavation for the drilled shaft, the shale spoils collected from the auger flights were observed to be dry.

Computed Drilled Shaft Axial Capacity

The soil and rock properties used to estimate the axial compressive capacity of the drilled shaft were determined by a number of means. The undrained shear strength in the three clay strata were assumed by the authors based on SPT blow counts, Atterberg limits, in-situ moisture content, and extensive experience in the project area. Note that no laboratory shear strength or consolidation testing was performed

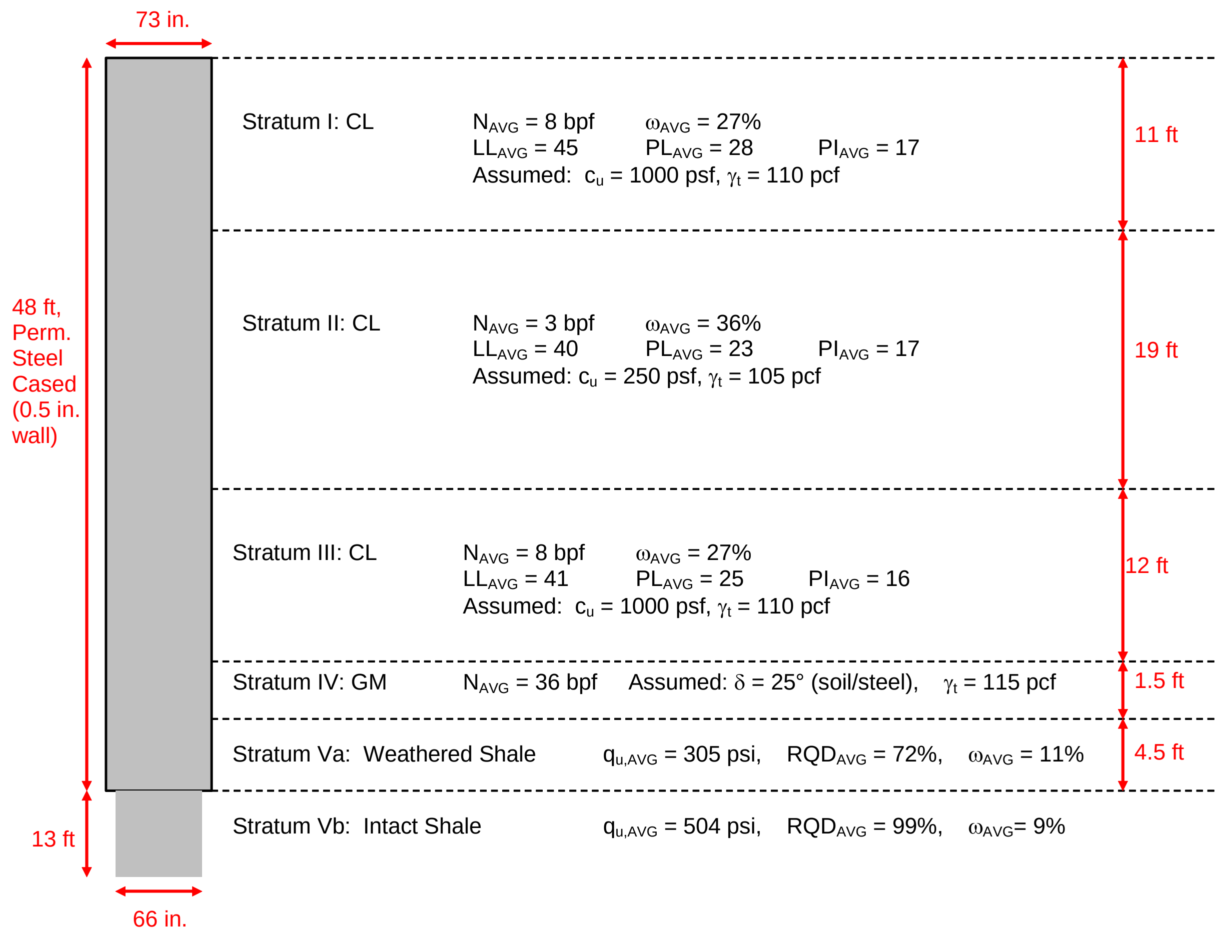

Figure 1: Conceptual cross-section showing various soil and rock strata and associated properties.

on soil samples for this project. However, this work is part of an on-going flood control project that has been under construction for nearly 20 years. Over that time, many such tests have been conducted on similar soil of the same deposit. Likewise, the drained friction angle of the soil in Stratum IV was assumed based on recorded SPT blow counts. Values of unconfined compressive strength for the weathered and intact shale were nominal averages of two tests from the weathered shale and three tests from the intact shale.

Using an alpha factor of 0.55 (O'Neill, 2001), the computed skin friction for Strata I, II, and III are 116 kips, 50 kips, and 126 kips, respectively. Assuming a lateral earth pressure coefficient (K) of 0.8 for Stratum IV, the computed skin friction is 27 kips. The skin friction developed in Strata V-a and V-b was computed using equation (1) (O'Neill, 2001 after Kulhawy and Phoon, 1993). The computed skin friction for Stratum V-a is 581 kips (73 inch outside shaft diameter). The computed skin friction for Stratum V-b is 1960 kips (66 inch outside shaft diameter). The combined skin friction for the shaft is 2860 kips.

$$f_{max}/p_a = \Omega(q_u/2p_a)^{0.5} \qquad (1)$$

where: f_{max} = maximum skin friction (lb/ft^2),
p_a = atmospheric pressure (2116 lb/ft^2),
Ω = 1 for smooth rock sockets, and
q_u = unconfined compressive strength (lb/ft^2).

The toe capacity was computed using the O'Neill and Reese (1999) method for intermediate geomaterials, assuming cohesive rock with RQD between 70 and 100. The equation (2) is given below (note the units in equation (2) are maintained as produced by O'Neill and Reese due to the exponent, but all reported values in this document have been converted back to English units). From Equation

(2), q_{max} equals 190 kips/ft^2. The resulting toe capacity calculated at the base of the 66-inch diameter is therefore 4505 kips. Potential bearing arising from the change in shaft diameter was neglected in the estimation of total capacity.

$$q_{max} = 4.83(q_u)^{0.51} \qquad (2)$$

where, q_{max} = toe resistance (MPa), and
q_u = unconfined compressive strength of competent shale (3.45 MPa, equivalent to 504 lb/in^2).

By adding the skin friction and toe resistance terms together, the estimated axial compressive capacity of the drilled shaft is 7365 kips.

Testing Procedure

Axial compressive Statnamic testing was applied in three increasing cycles. The magnitudes of the loadings were 927 kips, 1007 kips, and 3177 kips, respectively (approximately 12 percent, 14 percent, and 43 percent of the estimated capacity, respectively). The design criterion was based on an allowable deflection at the anticipated maximum load. At the time of design, the variability in the shale and the owner's required maximum allowable vertical deflection of 1-inch dictated an overly-conservative design. Accordingly, the initial intent of the load tests was to "proof-test" the drilled shaft. Hence, the loading apparatus was sized more for allowable and/or expected capacities rather than for loads that would induce plunging failure.

The first two loading cycles were applied 13 days after the drilled shaft concrete was placed. The third cycle was applied 21 days after the first two (34 days after the shaft was poured). The drilled shaft that was tested will also serve as a production shaft.

The equipment used in this testing program produced a time-dependent load that lasted approximately 0.5 seconds. Of course, the rapid loading of the soil and rock may result in a higher observed shear strength than would be measured by conventional laboratory shear tests at lower strain rates (Casagrande and Wilson, 1951), but the effects of this on the results of this study are thought to be minor. Note that the three clay layers (Strata I, II and III) contribute only 10 percent of the estimated side friction and only 4 percent of the estimated capacity. In addition, the nature of the Statnamic tests is thought to more accurately model the loading of the foundation by the intended rail traffic. The load applied to the drilled shaft was measured with a calibrated ring type electronic resistance load cell, located between the shaft head and the Statnamic piston. Acceleration was measured with four accelerometers arranged approximately 90 degrees apart and attached to the head of the shaft. From the measured accelerations, the displacement was calculated by double integration. Elevation measurements were surveyed prior to and at the completion of each load cycle. The surveyed measurements agreed well with the displacements determined from the testing device's accelerometers.

Test Results

Very little shaft movement was observed during the first two loading cycles. During the first load cycle, which applied a load of 927 kips, the head of the shaft moved downward 0.052 inches. The permanent set at the conclusion of the first test was 0.017 inches. During the second load cycle, which applied a load of 1007 kips, the head of the shaft moved downward 0.062 inches. The permanent set at the conclusion of the second test was 0.023 inches. During the third load cycle, which applied a load of 3177 kips, the head of the shaft moved downward 0.257 inches during loading. The permanent set at the conclusion of the third test was 0.128 inches. The third test, which applied an axial compressive load approximately 3 times larger than the previous two tests, resulted in approximately 5 times more movement.

The elastic compression of the shaft was computed using Equation 3. The respective areas (A_1 and A_2 defined below) were computed after converting the area of steel to an equivalent area of concrete based on a Young's modulus of Elasticity of steel equal to 29,000 k/in^2. Note that both the reinforcing steel and the permanent steel casing were considered when computing the area of the equivalent concrete sections.

$$\Delta L = P_{HEAD}L_1/E_{concrete}A_1 + P_{HEAD}L_2/E_{concrete}A_2 \qquad (3)$$

where: ΔL = change in member length (in.),
P_{HEAD} = load imposed on the shaft head (kips),

L_1 = 72-inch diameter shaft length (576 in.),
L_2 = 66-inch diameter shaft length (156 in.),
$E_{concrete}$ = Young' modulus of elasticity (assumed to be 3500 k/in^2 for concrete),
A_1 = equivalent cross-sectional area of the 72-inch diameter portion of the shaft (5383 in^2.), and
A_2 = equivalent cross-sectional area of the 66-inch diameter portion of the shaft (3371 in^2.).

The computed elastic compression of the drilled shaft including the reinforcing steel and the permanent steel casing for the three loading cycles is provided in Table 1 along with other pertinent data collected during the testing program. The vertical displacements measured at the head of the shaft during the testing cycles are also provided graphically in Figure 2.

The final set after each load cycle was added to the computed elastic compression for that respective load, and that sum was compared to the maximum movement measured during the test. In all instances, the difference between measured and computed movements is less than 8 percent, indicating that for all load cycles, the rebound of the soil/rock upon unloading was negligible. Furthermore, it appears evident that the difference between measured and computed movements is inversely related to applied load; i.e., as the load increases, the differences decrease.

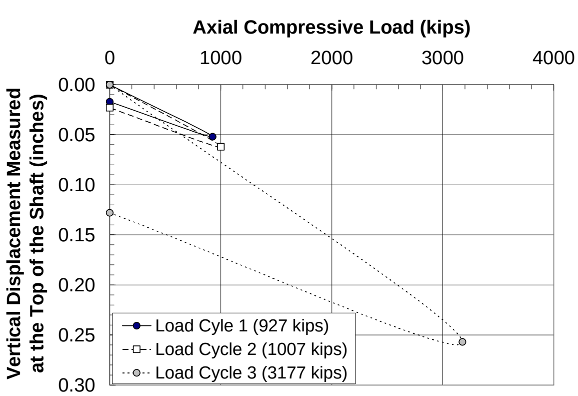

Figure 2: Load-settlement curves for the three load cycles.

Comparison of Design Capacity and Observed Behavior

The Statnamic test was originally intended to verify the required capacity of the shaft and the static analysis method used to estimate the capacity. While at the capacity of the equipment mobilized to this job site, the inability of the tests to exceed the capacity of the shaft and initiate plunging failure was a result of the contract specifications and should not be considered a constraint of the Statnamic test method. As a result, direct comparison of measured and calculated capacities is therefore not possible. However, evaluation of the estimated design capacity can be accomplished based on results from the Statnamic testing program by utilizing normalized load transfer relations presented by O'Neill and Reese (1999). Such comparisons necessarily require *extrapolation* of the data measured in the Statnamic tests following typical load-displacement response.

To use the normalized curves presented in Appendix C of O'Neill and Reese (1999), the following information is required (note the same notation is used herein as in the referenced publication):

Δ_L = elastic change in member length, calculated in Equation (3),

k = 0.5 (all load transferred in side resistance), 0.67 (portion of the load transferred in base resistance)

$\delta_s = k\Delta_L$ (δ_s is the compression within the drilled shaft due to column action)

w_T = maximum movement measured during each test

$w_s = w_T - 0.5\delta_s$ (w_s is the movement at the center of the shaft assuming uniform side load transfer rate)

$w_b = w_T - \delta_s$ (w_b is the settlement at the base)

Table 1: Comparison of measured data and computed data during Statnamic Testing.

Test No.	Axial Compressive Load Applied (1)	Computed Elastic Compression (2)	Maximum Measured Movement during Test (3)	Measured Set at Conclusion of Test (4)	Computed Elastic Compression plus Measured Set (5)	Difference between Column (3) and Column (5)	
	(kips)	(inches)	(inches)	(inches)	(inches)	(inches)	(%)
1	927	0.039	0.052	0.017	0.056	0.004	7.7
2	1007	0.042	0.062	0.023	0.065	0.003	4.8
3	3177	0.134	0.257	0.128	0.262	0.005	1.9

Use of the O'Neill and Reese (1999) relations requires the diameter of the shaft. However, the shaft herein changed in diameter from 73 inches for the upper 48 feet to 66 inches for the lower 13 feet. The average diameter over the 61-foot length of the shaft, and that used in this analysis, was 71.5 inches. However, the base diameter was still taken as 66 inches for toe resistance calculations. All information required for use of the O'Neill and Reese (1999) normalized curves is shown in Table 2

If we assume that none of the load applied in Tests 1 and 2 reaches the base of the shaft, the ultimate side load transfer (USLT) can be estimated using the normalized curves assuming the entire load is carried by side resistance. By entering the plot shown in Figure 3 with the settlement-diameter ratio (0.059 shown in Table 2), the USLT computed from Test 1 data is 2915 kips (927/0.318). Similarly, the USLT computed from Test 2 data is 2868 kips (1007/0.351). The average USLT from Tests 1 and 2 is 2892 kips. This agrees well with the design skin friction computed previously, which was 2860 kips (a difference of about 1 percent).

Assuming the USLT determined in Tests 1 and 2 are accurate, the normalized curves can then be used to evaluate the base resistance using results from Test 3. Based on Test 3 results and the normalized curves, approximately 86 percent of the USLT is mobilized in side shear in Test 3, or 2490 kips (2892x0.861). This leaves 687 kips (3177-2490) mobilized in end bearing. Figure 3 indicates approximately 18 percent of the ultimate end bearing load is mobilized in base resistance in Test 3. Assuming that 687 kips are transferred in end bearing and following the same logic as before, it then follows that the ultimate end bearing (UEB) based on the normalized load-settlement response in end bearing (Figure 4) is 3817 kips (687/0.18). This is approximately 15 percent lower than the ultimate end bearing capacity estimated using Equation 2 (4505 kips), but probably within the range of variability expected of the extrapolation procedure. Such agreement not only reconfirms the accuracy of computing USLT from Tests 1 and 2, but also confirms the accuracy of the design end bearing calculation. The sum of the extrapolated USLT and UEB is 6709 kips (2892+3817). This underestimates the calculated design capacity of 7365 kips by 9 percent.

Table 2: Required information for use of O'Neill and Reese (1999) normalized curves.

Test	ΔL (inches)	k	δ_s (inches)	w_T (inches)	w_s (inches)	w_b (inches)	w_s/71.5 in. (percent)	w_b/66 in. (percent)
1	0.039	0.5	0.020	0.052	0.043	0	0.059	0
2	0.042	0.5	0.021	0.062	0.052	0	0.072	0
3	0.134	0.67	0.090	0.257	0.212	0.167	0.297	0.253

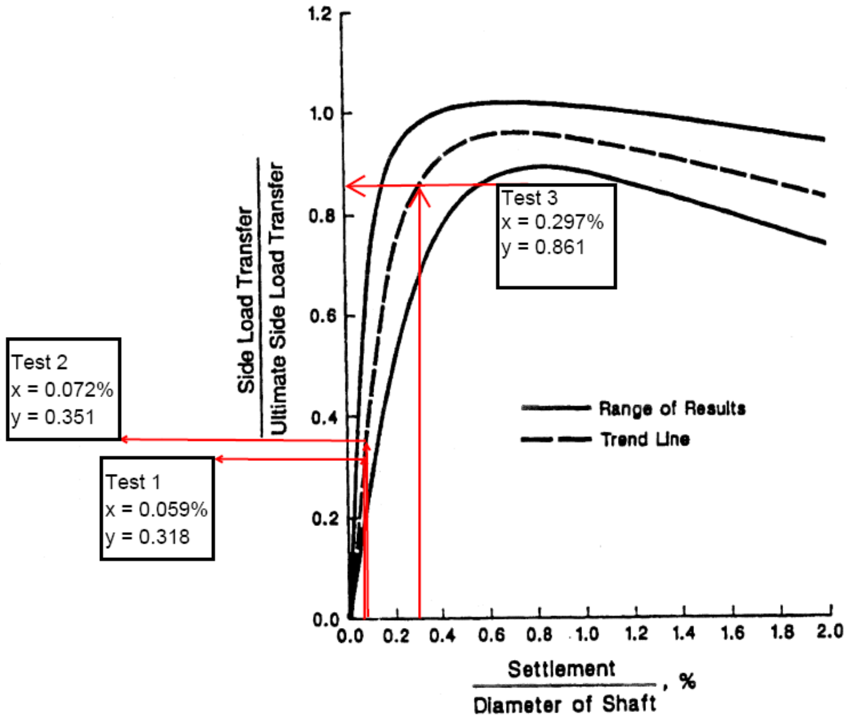

Figure 3: Normalized load transfer relations for side resistance in cohesive soil (after O'Neill and Reese, 1999).

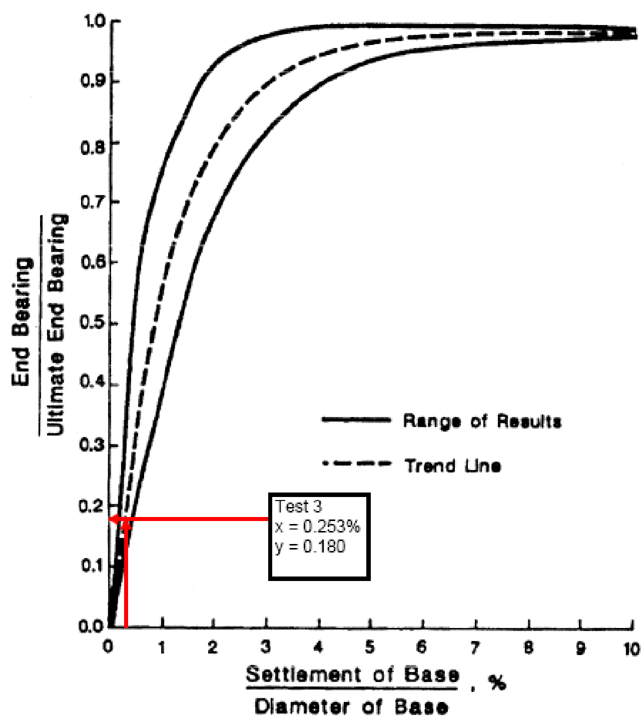

Figure 4: Normalized load transfer relations for base resistance in cohesive soil (after O'Neill and Reese, 1999).

Conclusion

The tested drilled shaft near the Blue River in eastern Kansas City had a total embedded length of 61 feet. The upper 48 feet was 72 inches in diameter and protruded into weathered Pleasanton shale approximately 4.5 feet. The remaining 13 feet was 66 inches in diameter and was embedded in competent Pleasanton shale. A permanent steel casing encased the entire 72-inch diameter portion of the drilled shaft. Axial Statnamic testing was applied in three increasing cycles. The magnitudes of the loadings were 927 kips, 1007 kips, and 3177 kips, respectively. The maximum movements recorded during each of the loadings were 0.052 inches, 0.062 inches, and 0.257 inches, respectively.

The method for estimating side resistance of drilled shafts presented by Kulhway and Phoon (1993) in Pleasanton shale rock sockets, assuming a smooth socket, appears to agree well with observed behavior during Statnamic testing. Accordingly, the method for estimating the end bearing of drilled shafts presented by O'Neill and Reese (1999) for intermediate geomaterials, assuming cohesive rock with RQD between 70 and 100, appears to agree well with observed behavior during Statnamic testing.

Based on Statnamic testing at varying load magnitudes and normalized curves developed by O'Neill and Reese (1999), the contribution of side resistance to the capacity of a deep foundation can be determined assuming no load is transferred to the base of the foundation during lower magnitude testing. Furthermore, higher magnitude testing can provide a means to evaluate the accuracy of design end bearing calculations.

To the knowledge of the authors, no other publications or data exist utilizing similar staged Statnamic testing and associated back-analysis as presented herein. Accordingly, the practicing engineer should be aware of the limitations resulting from drawing conclusions based on the testing of one foundation element, particularly when no site-specific laboratory shear strength or consolidation testing was performed.

No measurable rebound was observed upon unloading the shaft in any of the tests. This may be a result of the relatively low loads applied to the shaft with respect to the computed capacity.

References

KULHAWY, F.H. and PHOON, K.K., 1993. Drilled shaft side resistance in clay soil to rock. American Society of Civil Engineers, ASCE Geotechnical Special Publication No. 38, pp. 172-183.

O'NEILL, M.W., 2001. Side resistance in piles and drilled shafts. American Society of Civil Engineers, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 127, Issue 1, pp. 3-16.

O'NEILL, M.W. and REESE, L.C., 1999. Drilled shafts: construction procedures and design methods. Federal Highway Administration Publication No. FHWA-IF-99-025.

CASAGRANDE, A. and WILSON, S.D., 1951. Effect of rate of loading on the strength of clays and shales at constant water content. Geotechnique, Vol.2, No.3, pp. 251-263.