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February 11, 2008

Alabama Department of Transportation
Bureau of Materials and Tests
1409 Coliseum Boulevard
Montgomery, AL 36130-3050

Attention: Ms. Lyndi Davis Blackburn, P.E.

Re: BRF-00008(536)
Drilled Shaft Load Test - Assessment
Sumter County

Dear Lyndi:

We are pleased to provide this report of our assessment of the drilled shaft load tests performed at the site of the SR 8 bridges in Sumter County. ALDOT requested that we review the load test results and the adjusted design values to determine if they can be extrapolated to other cohesive intermediate geomaterials (IGM) in similar geologic formations within the state. As outlined in your letter dated January 8, 2007, the formations of interest are:

Demopolis Chalk
Mooreville Chalk
Nanafalia Formation
Naheola Formation
Porters Creek Formation
Clayton Formation

The following documents were provided to us for the project:

- Bridges Foundation Report, Additional Lanes on US Hwy 80, ALDOT Project NHF-010(550), Sumter County, Alabama, dated April 10, 2006, by Southern Earth Sciences, Inc.
- Final Report of Axial Statnamic Load Testing, SR 8 from CR 71 to SR 28, ALDOT Project No.: BRF-0008(536), Sumter County, Alabama, dated October 20, 2006, by Applied Foundation Testing, Inc.
- Final Report of Lateral Statnamic Load Testing, SR 8 from CR 71 to SR 28, ALDOT Project No.: BRF-0008(536), Sumter County, Alabama, dated October 30, 2006, Applied Foundation Testing, Inc.

- Letter from Southern Earth Sciences, Inc. dated November 28, 2006 providing revised end bearing and side shear values for shaft design based on the load test results.

We were also provided the following reports for a nearby project to the east on U.S. 80 in the same chalk formation as the subject project:

- Foundation Report, Bridge over Tombigbee River on US 80 Main Channel, BIN 019105, ALDOT Project No. MGF-0101(519), Sumter and Marengo Counties, by ALDOT Bureau of Materials and Tests, dated November 30, 2004.
- Report of Additional Soil Boring and Unconfined Compression Tests, Bridge over Tombigbee River on US 80 Main Channel, BIN 019105, ALDOT Project No. MGF-0101(519), Sumter and Marengo Counties, by ALDOT Bureau of Materials and Tests, dated January 21, 2005.

Sumter County Load Test Summary

The bridge project included four bridges on SR 8 in Sumter County. These bridges span the Sucarnoochee River, Ponkabia Creek, and Mill Creek. The geotechnical exploration was performed by Southern Earth Sciences, Inc. (SES).

The subsurface conditions along the project alignment consisted of sandy and silty soils overlying the chalk of the Porters Creek and Clayton formations. The thickness of the overlying alluvial soils varied across the project alignment. The chalk encountered was described as hard gray silty clay (CH) or hard gray silt (MH). Mud rotary drilling techniques were used to advance the borings in the chalk with SPT samples obtained at typically 5-foot intervals. SPT N-values were typically 50 blows for 3 inches to 50 blows for 5 inches ($N = 50/3''$ to $50/5''$), though there were some isolated zones with N values in the 30 to 40 blows per foot range. No other sampling was attempted, so there was no unconfined compression or triaxial shear test data available.

SES recommended that the bridges be supported on driven piles or drilled shaft foundations bearing in the chalk. The design axial loads for the drilled shafts ranged from 300 tons to 825 tons. SES designed shafts with diameters of 3.5 to 7 feet and lengths ranging from 40 to 105 feet below grade to support the loads under the design scour conditions.

Static axial and lateral load tests were performed on an 84-inch (7-foot) diameter, 41 foot long shaft. The upper 10.5 feet of shaft was permanently cased with a 90 inch outside diameter steel casing and a wall thickness of 0.5 inch. The Mill Creek Bridge site was selected as the location of the load tests due to the relatively shallow alluvial layer present at this location.

The maximum derived static axial load of the test shaft was 8,760 kips with maximum and permanent displacements of 0.36 inches and 0.13 inches, respectively. The maximum unit side shear resistance measured in the uncased portion of the test shaft ranged from 5.8 ksf to 10.7 ksf, averaging 8.2 ksf over the length of the socket. The maximum unit base resistance measured at the shaft tip was approximately 41.6 ksf. AFT reported that the unit side shear resistance may not have developed according to normal mechanisms due to the step down in cross section from the cased section to the uncased section. Some base resistance load was

likely carried by the step. Since the gross settlement of the shaft head was less than one percent of the shaft diameter, the test did not fully mobilize the ultimate side shear and base resistances.

After reviewing the load test report, SES recommended revised design criteria for modifying the drilled shaft designs. Tables 1A and 1B below lists the original design parameters used by SES and the recommended parameters for redesign as outlined in their letter dated November 28, 2006.

Table 1A - Drilled Shaft Design Values - Initial Design

SPT N values	Friction reduction factor, α	Unit Side Shear (ksf)	End bearing coefficient, N_c	Unit End Bearing (ksf)
N > 100 (refusal)	0.5	5	8.0	40
N < 100	0.5	3 to 4	8.0	40

Table 1B - Drilled Shaft Design Values - Adjusted Design

SPT N values	Friction reduction factor, α	Unit Side Shear (ksf)	End bearing coefficient, N_c	Unit End Bearing (ksf)
N > 100 (refusal)	1	6	8.0	40
N < 100	1	4 to 5	8.0	40

Data Collection

To begin the data collection phase of our work, we reviewed a large database of public project drilled shaft load tests compiled by Loadtest, Inc and made available through their cooperative effort with the ongoing nationwide research effort funded by ADSC: The International Association of Foundation Drilling. The database was searched for projects in the southeastern U.S. with drilled shafts constructed in cohesive IGM similar to the chalk tested in Sumter County. Once potential load tests were identified in the database, we contacted the various state DOTs to obtain detailed geotechnical data from the projects. The data was used to evaluate the suitability of each particular test for inclusion in this study.

In addition to the Loadtest, Inc. database, we searched available literature and made inquiries within the foundation industry to find other relevant load test data. We also obtained existing load test data from ALDOT.

Table 2 summarizes the load tests that were selected for inclusion in this study based on our review of the data collected. A total of 15 tests were selected. A case history summary of each load test is included in the Appendix.

Table 2 - Summary of Load Test Data

State	Project	Test No.	Reference	Test Type	Material	SPT N values (bpf)	q _u (ksf)	Unit Side Shear (ksf)	Unit End Bearing (ksf)
AL	US 80 over Mill Creek	AFT-106058	1	Statnamic	Demopolis/Mooreville Chalk (hard gray, clayey silt)	N > 100	---	5.8 – 10.7	41.6
AL	SR 10 Blue Springs	LT-8571	2	O-Cell	Claystone	N > 100	---	0.9 - 1.7	27.4
AL	Hyundai Motor Manufacturing	LT-8904	2	O-Cell	Demopolis Chalk	42 - 62	18.4	3.4 – 9.4	90.9
AL	Andalusia, AL	WRT-1-1	3	Conventional	Claystone	---	17 - 127	7.0 - 9.6	---
MS	US 45 over Town Creek	LT-8194	2	O-cell	Mooreville Chalk (Hard grey, clayey, silt)	---	23.5	5.1	197.0
MS	SR 25 over Talking Warrior Creek	LT-8373	2	O-cell	Basal Formation (Hard Clayey Silt and Silty Clay)	---	6.1 - 27.9	2.2 - 5.9	67.8
MS	US 82 Oktibbeha County	LT-8461-1	2	O-cell	Demopolis Formation (Hard, Argillaceous Chalk)	---	10.9 - 38.6	3.1 - 7.3	214.0
MS	US 82 Oktibbeha County	LT-8461-2	2	O-cell	Prairie Bluff Formation (Hard, Silty, Clay) and Ripley Formation (Hard, Sandy, Silt)	---	27.1 - 28.8	2.1 - 3.9	108.0
MS	SR 42 over Thompson Creek	LT-8487	2	O-cell	Very stiff to hard, clayey silt and silty sand	---	7.1	1.9 - 5.0	24.8
MS	I-55 at Old Agency Rd.	LT-8788	2	O-cell	Yazoo Formation (Hard, tan, silty clay)	---	9.0 - 11.1	0.3 - 1.5	52.3
MS	SR 9 over SR 6	LT-8912-1	2	O-cell	Clayton Formation (Hard, clayey silt and silty clay)	N > 100	18.0	7.7 - 8.4	202.8
MS	SR 9 over SR 6	LT-8912-2	2	O-cell	Ripley Formation (Hard, very fossiliferous, sandy silt)	N > 100	18.0	8.8 - 12.9	221
MS	Leake County, MS	WRT-4	3	O-cell	Chalk		12.1	3.2	46.2
SC	Mt. Pleasant, SC	WRT-5-1	3	Conventional	Cooper Marl	9 - 100+	2.9	3.6	28.6
SC	Mt. Pleasant, SC	WRT-5-2	3	Conventional	Cooper Marl	9 - 100+	2.9	3.6	---
SC	Cooper River Bridge Charleston/Mt. Pleasant, SC	LT-86500	2	O-cell (10 tests)	Cooper Marl (Clayey sand, sandy clay, sandy silt)	15 - 100+	---	2.0 - 6.5	43.5 - 80
SC	Breach inlet Bridge	LT-8661	2	O-cell	Cooper Marl (Clayey sand, sandy clay, sandy silt)	15 - 28	4.2 - 5.9	0.2 - 2.8	49.4

Reference:

1. AFT-XXXXXX: Statnamic test report from AFT, Inc. with permission of owning state DOT
2. LT-XXXX: O-Cell test report from Loadtest, Inc. with permission of owning state DOT;
3. WRT-X: Test data from Thompson, W.R. III (1994) *Axial Capacity of Drilled Shafts Socketed into Soft Rock*, M.S. Thesis, Auburn University, AL

Analysis

Analyzing a set of load test data can be complicated by the lack of consistency of the types of data collected, particularly in strength data used to characterize the geomaterials. The unconfined compressive strength, or q_u , is typically used to describe cohesive IGM, as outlined in *Drilled Shafts: Construction Procedures and Design Methods*, FHWA Publication No. IF-99-025 (“FHWA manual”) and *Load Transfer for Drilled Shafts in Intermediate Geomaterials*, FHWA Publication No. RD-95-172. By definition, a cohesive IGM is a cohesive earth material with $10 \text{ ksf} < q_u < 100 \text{ ksf}$. Not all of the test sites in the set of data collected for this project have q_u data: some projects have unconfined compressive strength data, some have triaxial shear strength data, and some have only SPT data.

The collected load test data listed in Table 2 has values of q_u ranging from 2.9 to 127 ksf. The test sites in the Cooper Marl in South Carolina had q_u values on the low end of (or slightly below) the cohesive IGM range. Test site in claystone had a few q_u values at the upper end of (or slightly higher) the cohesive IGM range. Most of the Mississippi tests had no SPT data for the IGM strata. Except for the tests in Cooper Marl, SPTs (where performed) were usually recorded as 100+ blows per foot, or refusal. The sites in the Cooper Marl tended to have blow counts less than 50 blows per foot.

The only soil test data obtained at the US 80 project site were SPT data, making a direct comparison to most of the collected load test data difficult. Our experience is that q_u values for the chalk materials in Alabama typically range from 10 to 50 ksf, within the range of the load test data set in this report. We used an estimated $q_u = 45 \text{ ksf}$ for the US 80 site for our analysis. We also used unconfined compression tests data from the nearby bridge project to the east over the Tombigbee River for our analysis of the US 80 site.

Side Shear

Most state DOTs, including Alabama, use the 1999 FHWA manual for drilled shaft design. The basic formula for calculating ultimate unit side shear resistance is:

$$\text{Unit Side Resistance (ksf): } f_{\max} = \alpha \cdot \phi \cdot q_u,$$

where α and ϕ are functions of the IGM quality.

There are several other methods documented in the literature. The most recent evaluation of the various methods is contained in *NCHRP Synthesis 360: Rock Socketed Shafts for Highway Structure Foundations*, published in 2006 by Dr. John Turner. This synthesis will be included in the new edition of the FHWA Drilled Shaft Manual currently being written by a team led by Dr. Turner and Dr. Brown.

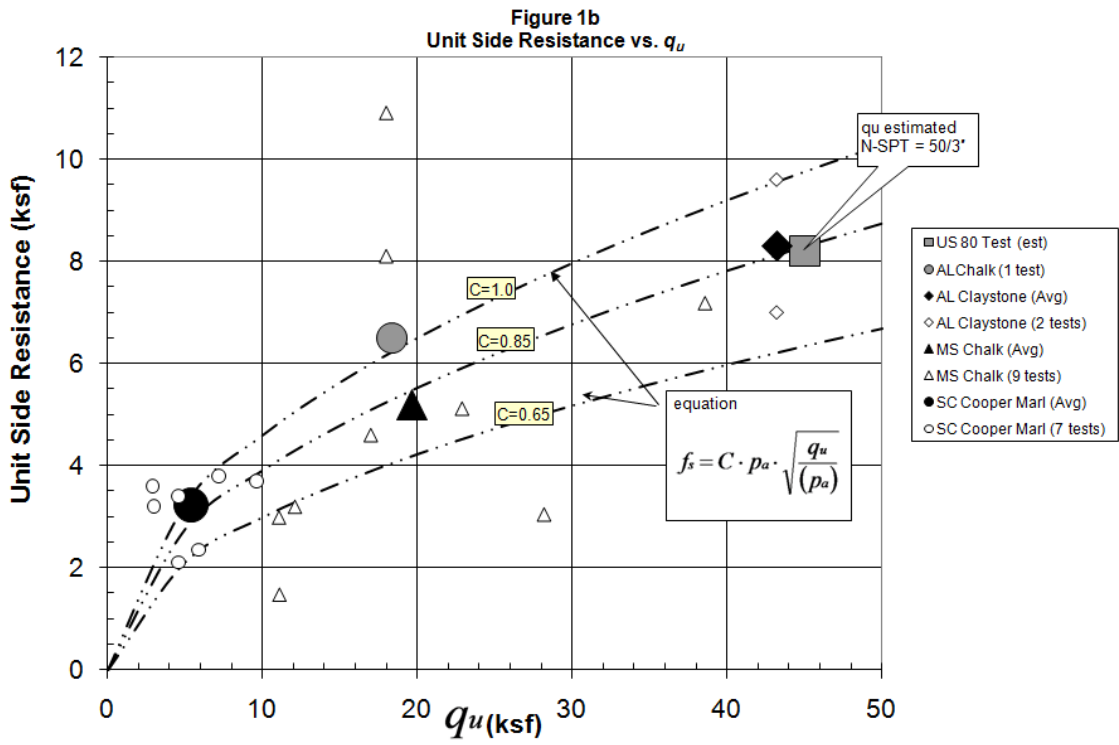
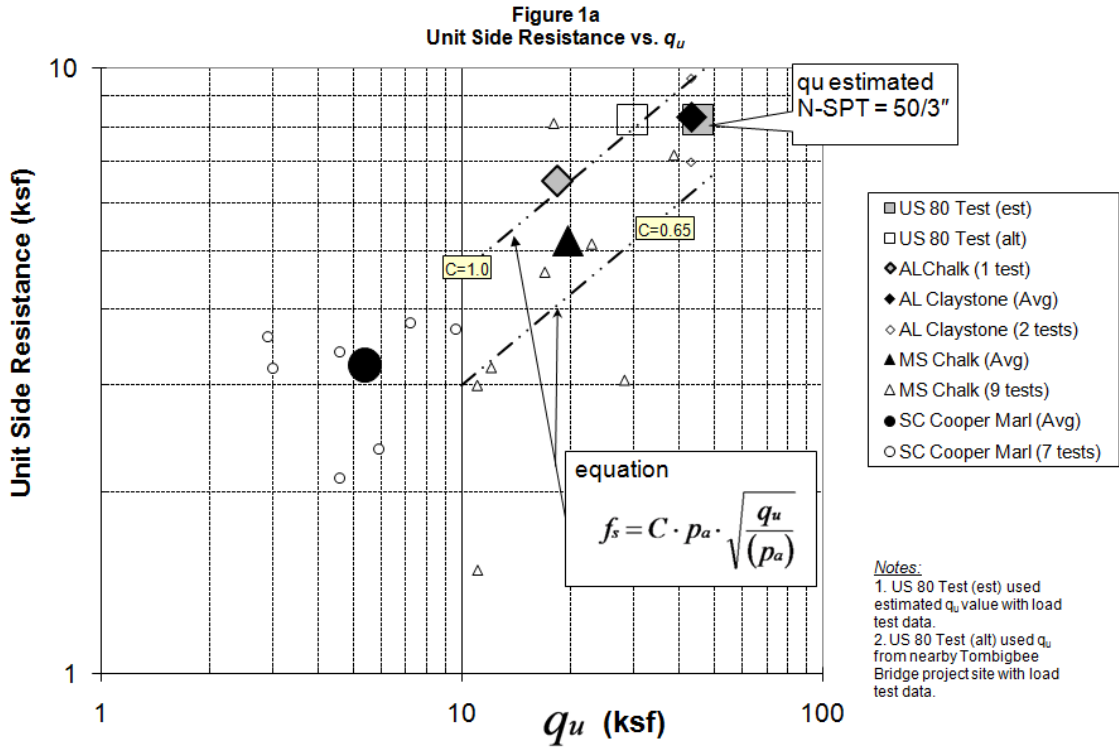
In the *NCHRP Synthesis 360*, several basic formulas for estimating ultimate unit side shear resistance utilize a correlation between ultimate unit side shear, f_s , and the square root of the unconfined compressive strength, q_u . These relationships may be expressed in a dimensionless form by normalizing compressive strength by atmospheric pressure as:

$$\text{Unit Side Resistance (ksf): } f_s = C \cdot p_a \cdot \sqrt{\frac{q_u}{p_a}}$$

where p_a is atmospheric pressure and C is an empirical constant.

Correlations of this type have been applied to rock sockets in shale, mudstone, claystone, limestone, and marl. Horvath and Kenny (1979) first proposed such a relationship, with an empirical constant ranging from 0.65 to 1 for smooth to rough sockets (with their original expression adjusted to the units normalized by p_a as indicated above). Rowe and Armitage (1987) proposed a similar expression with different constants, and Kulhawy and Phoon (1993) evaluated a larger database of rock sockets and proposed values of C ranging from 1 to 3 depending upon sidewall roughness. An updated evaluation of these test data by Kulhawy et al in 2005 suggests that a value of C = 1 would represent a conservative estimate of design ultimate side shear resistance, based on the most up-to-date analysis of the available data.

The above relationships are compared with measured unit side resistance from the data evaluated in this study, with the data plotted versus q_u on Figures 1a and 1b. These data are presented on both a log-log plot (Figure 1a on which the square root relationship plots on a straight line) and on a normal x-y plot (Figure 1b). Each test site with q_u data is shown as a small open symbol. The averages of the groups of tests (MS Chalk, SC Cooper Marl, and AL Claystone) are shown in large shaded symbols. The US 80 test is plotted both using an estimated value for q_u and an average value of q_u obtained from the nearby Tombigbee site. The curve representing the equation derived from the Horvath & Kenney relation for ultimate unit side shear resistance is shown, with the empirical constant ranging from C = 0.65 to 1.0. The data presented in this figure suggest that the relationship provides a reasonably conservative estimate of side shearing resistance with the constant at the lower end of this range (0.65), and a better fit through the mean with C = 0.85.



End Bearing

A wide range of expressions are available for relating base resistance to strength parameters of weak rock or marl. The FHWA guidelines for cohesive IGM which are massive and relatively free of joints, fissures, or weak seams would suggest a value similar to:

$$\text{Unit Base Resistance (ksf): } q_{ult} = 2.5q_u$$

Other methods which might be utilized for the massive chalk materials addressed in this study are those suggested by Rowe and Armitage (1987) and summarized in the NCHRP Synthesis study:

$$\begin{aligned} \text{Unit Base Resistance (ksf): } q_{ult} &= 13.7\sqrt{q_u} \text{ (lower bound)} \\ q_{ult} &= 18.3\sqrt{q_u} \text{ (mean)} \\ q_{ult} &= 30.2\sqrt{q_u} \text{ (upper bound)} \end{aligned}$$

Since the mobilization of unit base resistance occurs over a much larger range of deflections than does unit side resistance, the base resistance data are evaluated from the load tests as a function of base resistance in terms of the shaft diameter. Figures 2 and 3 illustrate the mobilized unit base resistance plotted as a function of displacement. The shaft displacements are normalized by dividing the displacements by the shaft diameter, and thus are expressed as a percent of the shaft diameter. The mobilized unit base resistance was normalized by q_u in Figure 2 and by $\sqrt{q_u}$ in Figure 3. This allowed for evaluation of the data with respect to the NCHRP Synthesis formula for unit base resistance.

For all but two of the tests, the measured end bearing load-displacement curve was available and is included on the plots. Two tests, one being the US 80 site, reached the maximum applied load at a shaft deflection of less than 1%. These are shown as single points on the graphs.

The data presented on Figure 2 indicate that the FHWA base resistance formula is a fairly conservative lower bound in most cases. One test site (LT-8194 US 45 over Town Creek) performed lower than the formula prediction with a maximum normalized unit base resistance of 1.6. The boring logs at this site indicated the material was a silty clay and did not indicate that there was a substantial amount of sand or other issues that could be pointed to as a cause of the lower performance. Another test (LT-8487 SR 42 over Thompson Creek) was terminated with a normalized unit base resistance of 2.4 at less than 2% normalized displacement. This test likely did not fully mobilize the ultimate base resistance and would likely have fit the rest of the data had the test continued.

The data on Figure 3 do not fit particularly well with the Rowe and Armitage formulas. Three of the tests exceed the upper bound, three are below the lower bound, and five fit within the limits.

Figure 2
Normalized Unit Base Resistance vs. Normalized Displacement
 (Unit Base Resistance normalized by q_u)

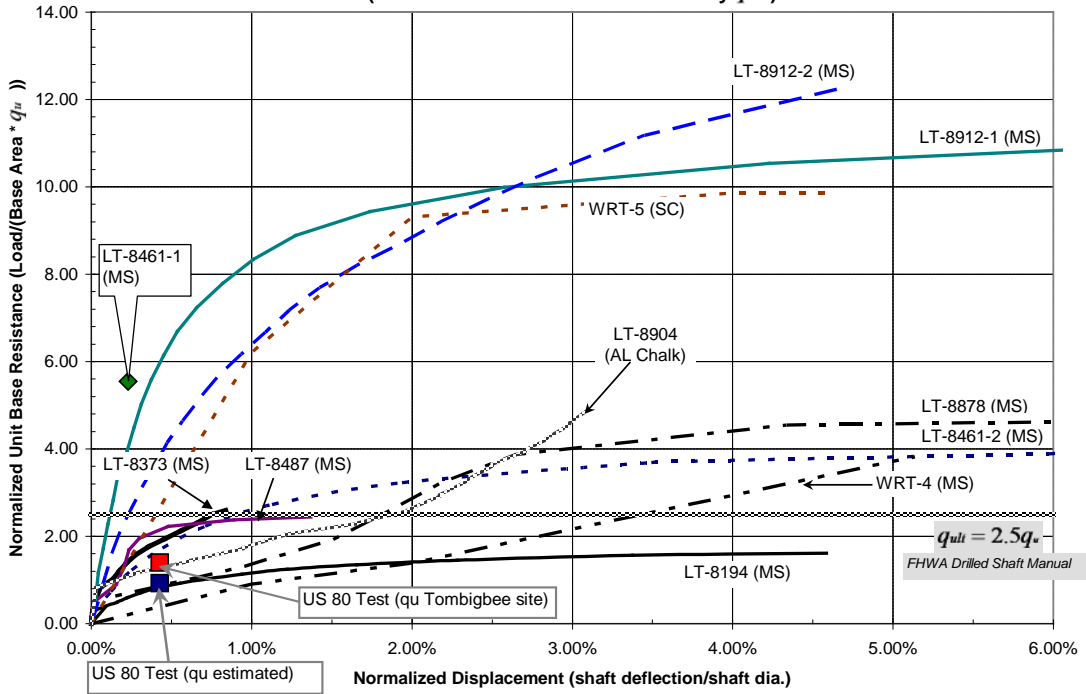
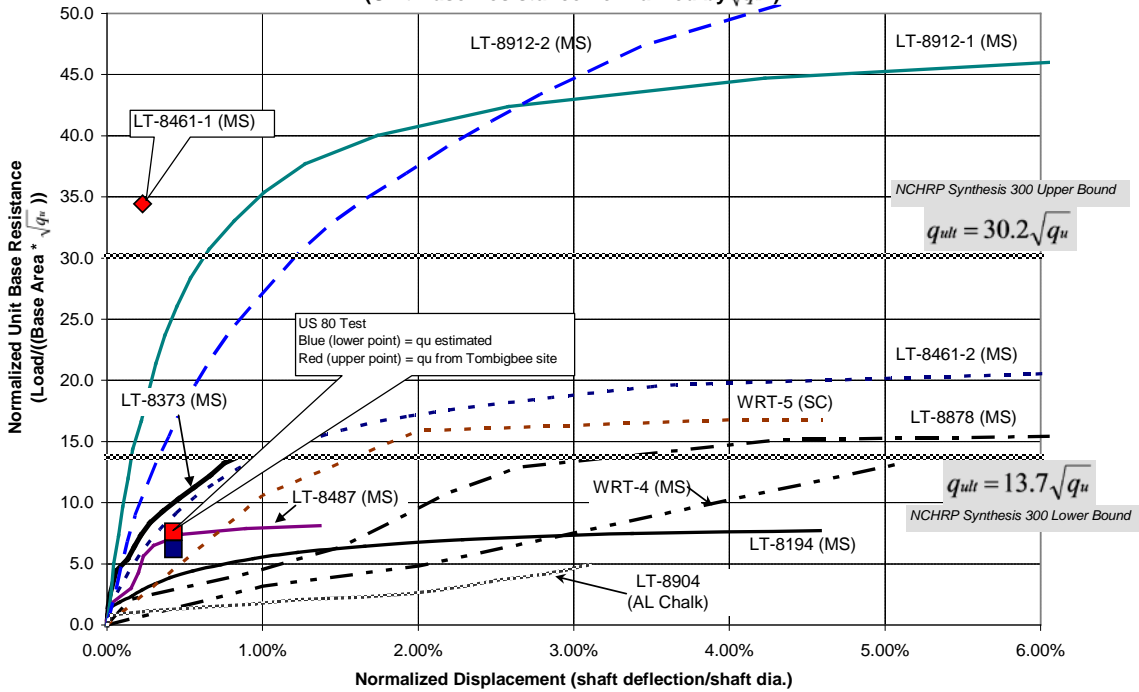


Figure 3
Normalized Unit Base Resistance vs. Normalized Displacement
 (Unit Base Resistance normalized by $\sqrt{q_u}$)



Conclusions and Recommendations

The load test at the US 80 site mobilized only a fraction of the available side and end resistance. The adjusted design values provided by SES (Table 1B) are reasonable lower bound design (allowable) values that can be adopted by ALDOT for the chalk/marl IGM materials within Alabama having an SPT refusal ($N > 100$, or $N = 50/x''$).

The relationships shown in Figures 1 through 3 can be used by ALDOT to evaluate future bridge sites in the subject geologic formations. The use of the approach described in this report will require that an effort be made to obtain samples of the cohesive IGM materials encountered at future project sites for unconfined compressive strength tests. In many cases, this will require the use of a Pitcher sampler or wireline coring to obtain useful samples. Having q_u data will allow ALDOT to better estimate side and base resistance in accordance with the FHWA manual or NCHRP Synthesis design methods.

Additional load tests in the cohesive IGM formations of interest will be beneficial, particularly tests that reach failure or otherwise provide a realistic upper bound for the unit design values. Future tests shafts should be sized with smaller diameters and shorter sockets. A test shaft can be scaled to a prototype which is smaller and more economical than a full size shaft and still have results that are applicable to a full scale shaft. Thus, an 84- or 96-inch production shaft could have a 48- to 72- inch test shaft. Socket lengths should be sized short enough to allow the shaft to fail in side shear.

As previously noted, Dr. Brown and Dr. Turner are leading a team that is working on the next edition of the FHWA drilled shaft manual. This revision includes re-evaluation of the design procedures, including cohesive IGM, incorporating much of Dr. Turner's work in the NCHRP Synthesis. ALDOT should review the results of this study within the context of the revised design methodology when the revised manual is published.

Closing

We appreciate the opportunity to provide our services for ALDOT. Please do not hesitate to call if you have any questions.

Respectfully submitted,

Dan A. Brown, Ph.D., P.E.

W. Robert Thompson, III, P.E.

Appendix:

References

Load Test Case History Summary Sheets

cc: R. Graham Forsythe, P.E. – Southern Earth Sciences, Inc.

Appendix

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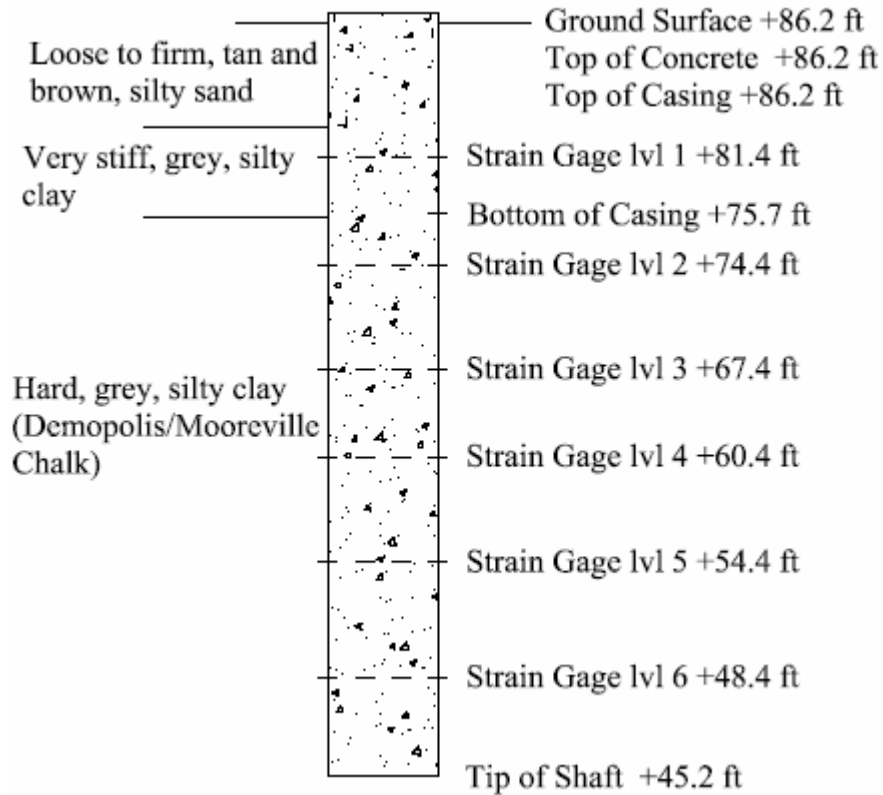
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Turner, J. (2006). *NCHRP Synthesis 360: Rock Socketed Shafts for Highway Structure Foundations*, Transportation Research Board, Washington, D.C.

PROJECT CASE HISTORIES

Statnamic axial and lateral load tests were performed on an 84-inch (7-foot) diameter, 41 foot long shaft. The upper 10.5 feet of shaft was permanently cased with a 90 inch outside diameter steel casing to isolate the shaft from the alluvial soils overlying the chalk of the Porters Creek and Clayton formations. The dedicated test shaft was constructed in the dry using an auger.



Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Max Derived Static Load (kips)	Max Deflection (in)	Perm Deflection (in)	Zone 1 Unit SS Cap (ksf)	Zone 2 Unit SS Cap (ksf)	Zone 3 Unit SS Cap (ksf)	Zone 4 Unit SS Cap (ksf)	Zone 5 Unit SS Cap (ksf)	Zone 6 Unit SS Cap (ksf)	Unit EB (kips)
84 ^(a)	41.00	8760.00	0.36	0.13	0	10.6 ^(b)	9.4	6.9	5.8	10.7	41.60

Data Summary

Depth (ft.)	Material	Location	SPT "N" (Blows / ft)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
86.2	Loose to Firm Silty Sand	Top of Shaft	5				
81.4	Loose to Firm Silty Sand	Strain Gage lvl 1	16			0	0.36
80.0	Silty Clay (CL)		18				
75.0	Hard grey, clayey, silt		50/3"				
74.4	Hard grey, clayey, silt	Strain Gage lvl 2	50/3"			10.6	
67.4	Hard grey, clayey, silt	Strain Gage lvl 3	50/3"			9.4	
60.4	Hard grey, clayey, silt	Strain Gage lvl 4	50/3"			6.9	
54.4	Hard grey, clayey, silt	Strain Gage lvl 5	50/3"			5.8	
48.4	Hard grey, clayey, silt	Strain Gage lvl 6	50/3"			10.7	
45.2	Hard grey, clayey, silt	Bottom of Shaft	50/3"	41.6	0.36		

Figure 1 - Load versus Displacement

84-inch Diameter Test Shaft
SR 8 from CR 71 to SR 28
ALDOT Project No.: BRF-0008 (536)
Sumter County, Alabama

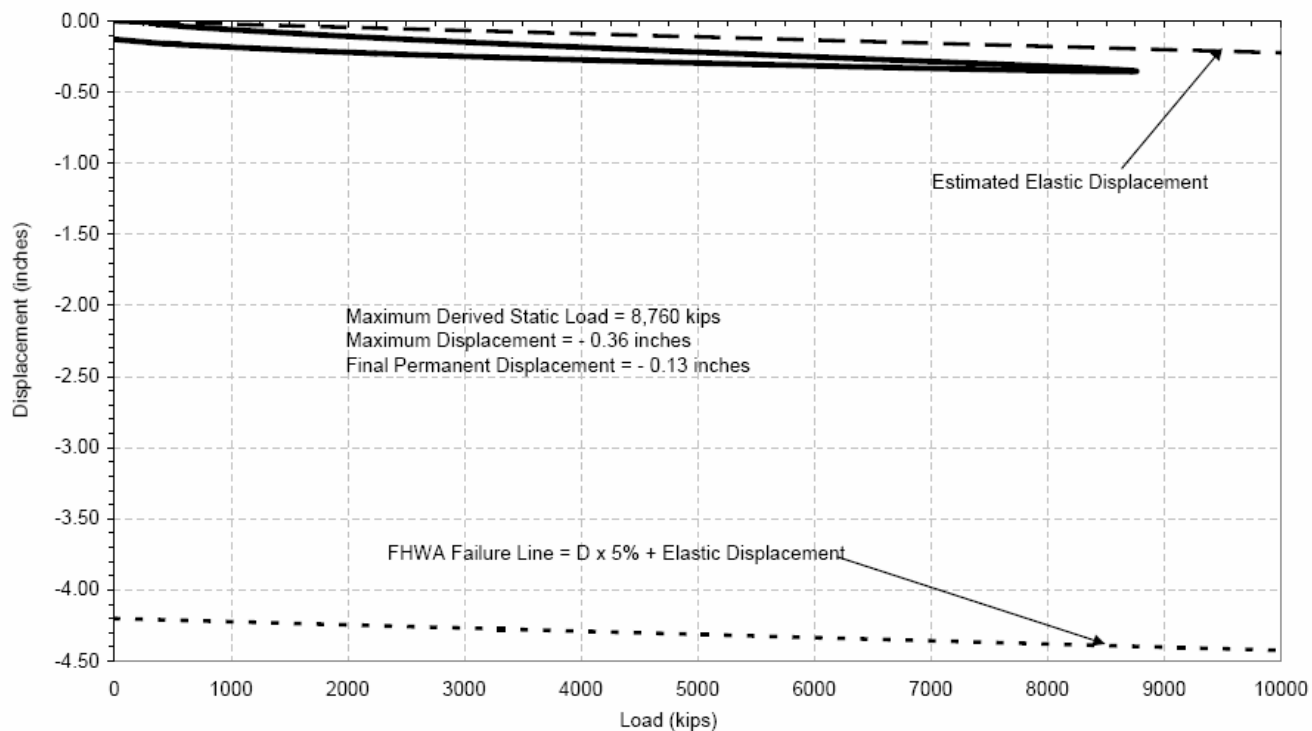
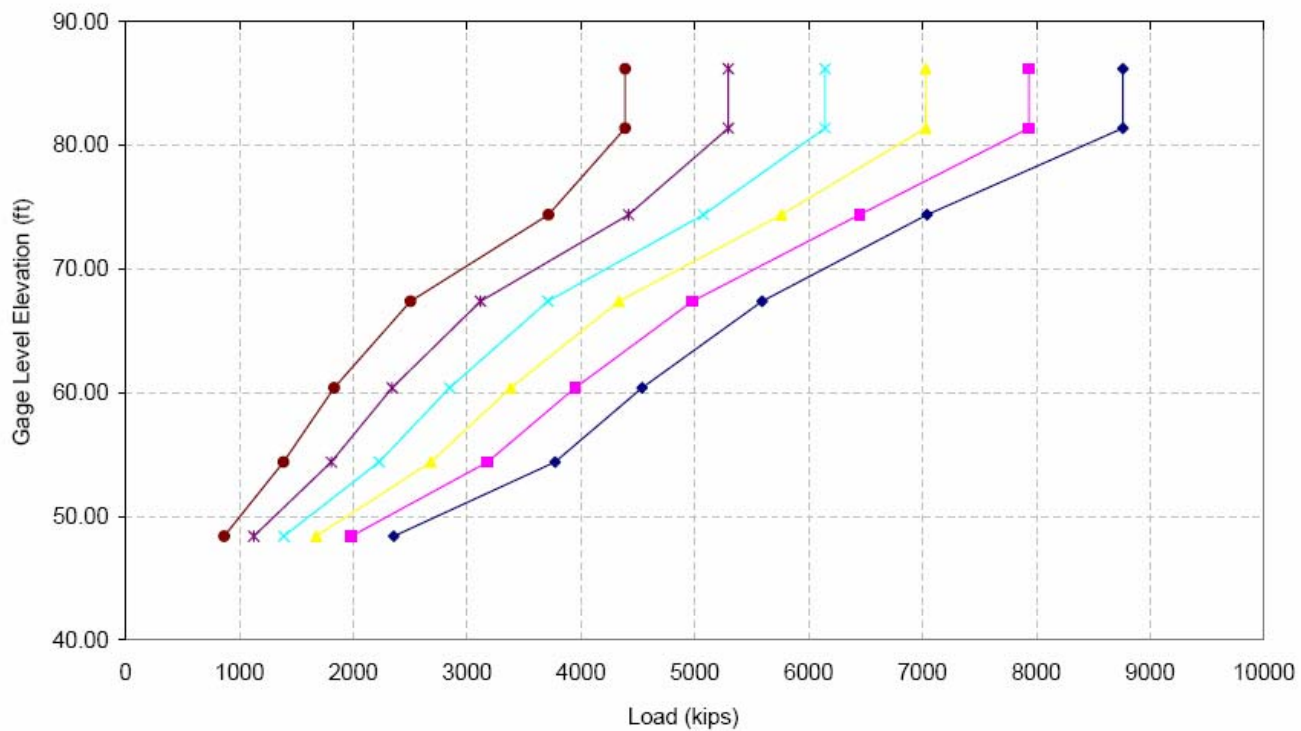
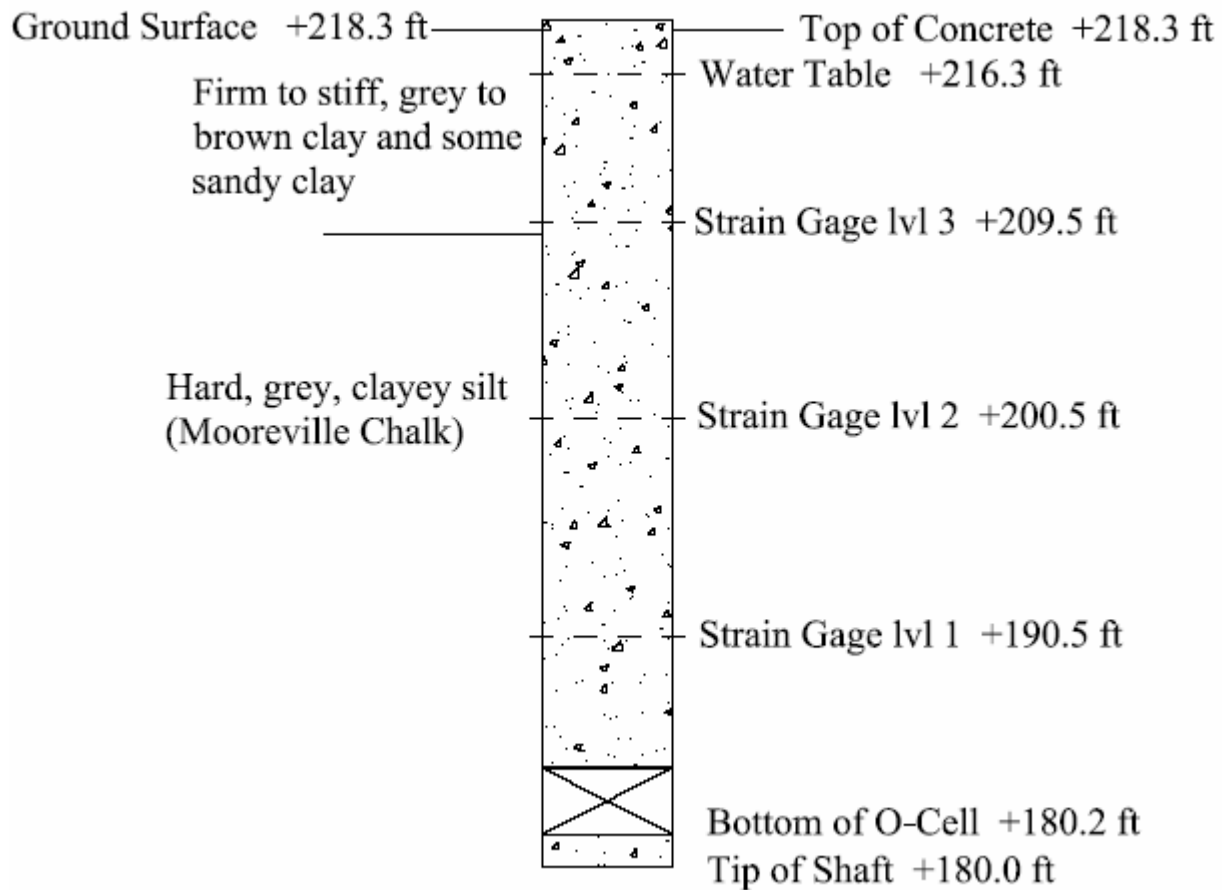


Figure 2 - Load Transfer versus Elevation

84-inch Diameter Test Shaft
SR 8 from CR 71 to SR 28
ALDOT Project No.: BRF-0008(536)
Sumter County, Alabama



The dedicated test shaft was constructed on November 9, 1994. The 48-in test shaft was constructed dry to a total length of 34.2 ft. An auger was used for drilling and cleaning the shaft. A 48-inch diameter temporary casing was inserted into the excavation to keep the excavation open for concrete placement. A seating layer of 3 inches of concrete was placed in the shaft, then the reinforcing cage with attached O-cell assembly was inserted into the excavation and into the seating layer of concrete with the base of the O-cell at elevation +180.2 ft. Concrete continued to be delivered by freefall until the top of the concrete reached an elevation of +218.3 ft (ground surface). The temporary casing was pulled as concrete was placed. No unusual problems occurred during the construction of the test shaft.



Shaft Summary

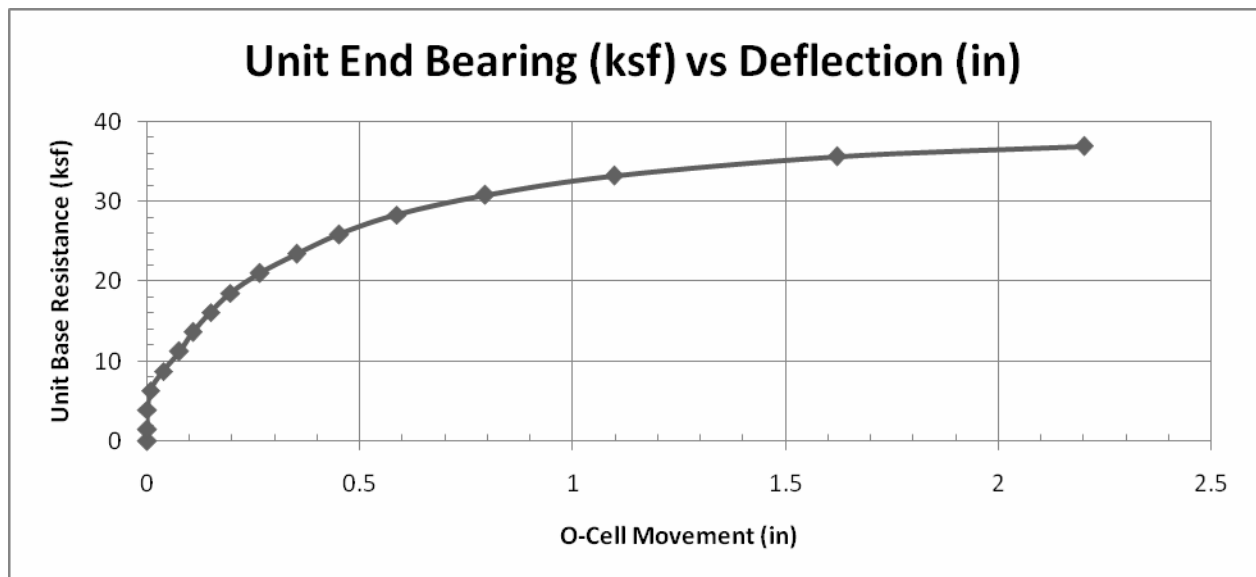
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
48.0	38.3	0.0	38.3	34.00	1896.00	0.11	2280.00	1470.00

Data Summary

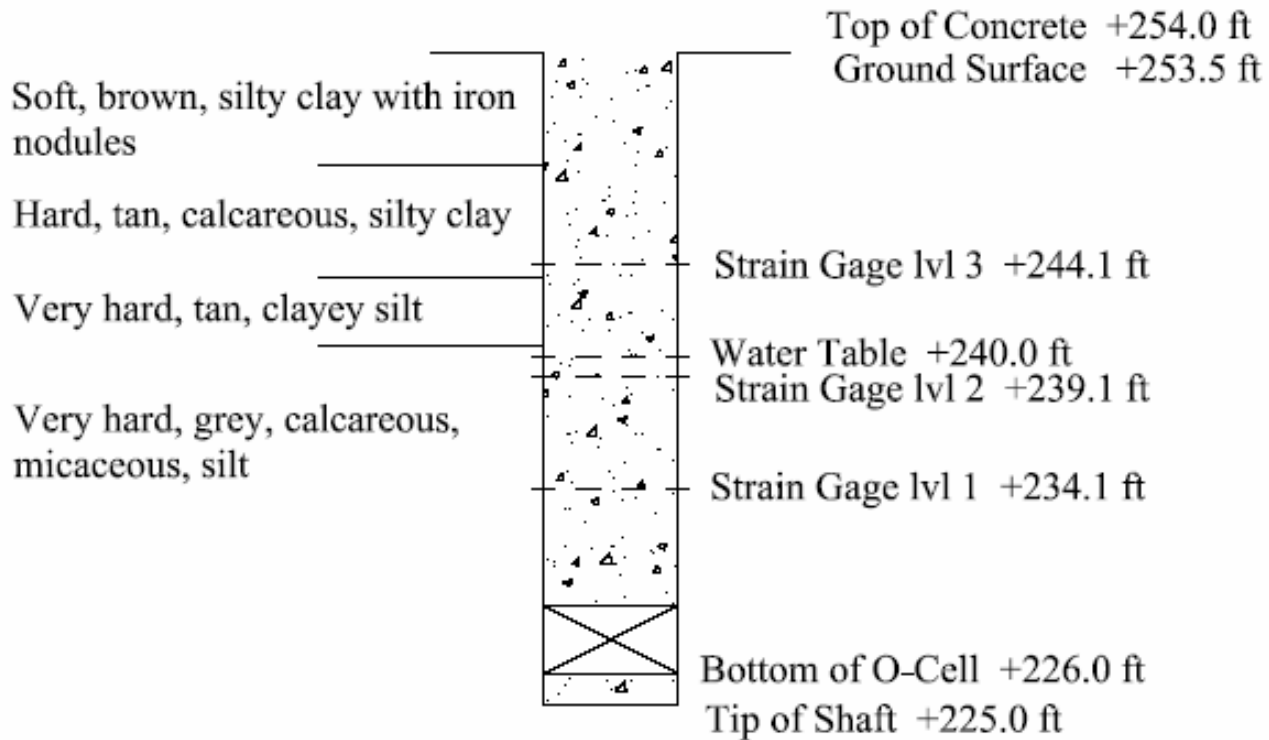
Elevation (ft.)	Material	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
218.3	Stiff Clay							
209.0	Hard, grey, clayey, silt	6	29				5.12	0.11
202.7	Hard, grey, clayey, silt			17.06				
200.5	Hard, grey, clayey, silt			34.26				
187.4	Hard, grey, clayey, silt			17.24				
180.0	Hard, grey, clayey, silt				197.0	2		
174.1	Hard, grey, clayey, silt			24.48				
165.5	Hard, grey, clayey, silt			24.6				

Unit Side Shear Curves

There are no unit side shear curves for this test. During the test, the strain gages did not record any change in strain at any of the gage locations. The recording equipment was believed to be functioning properly, so this indicates that there was no strain at the gages. This would mean that the side shear was concentrated in the 10-foot section between the top of the O-cell and Strain Gage Level 1. At the maximum load of 928 tons, this would result in a unit side shear of 14.7 ksf. If the side shear was assumed to be distributed over the entire length of the chalk socket, the resulting unit side shear would be 5.12 ksf.



The test shaft was constructed on July 21, 1998. The 42-in test shaft was constructed dry with a total length of 29.0 feet. No unusual problems occurred during construction of the test shaft.

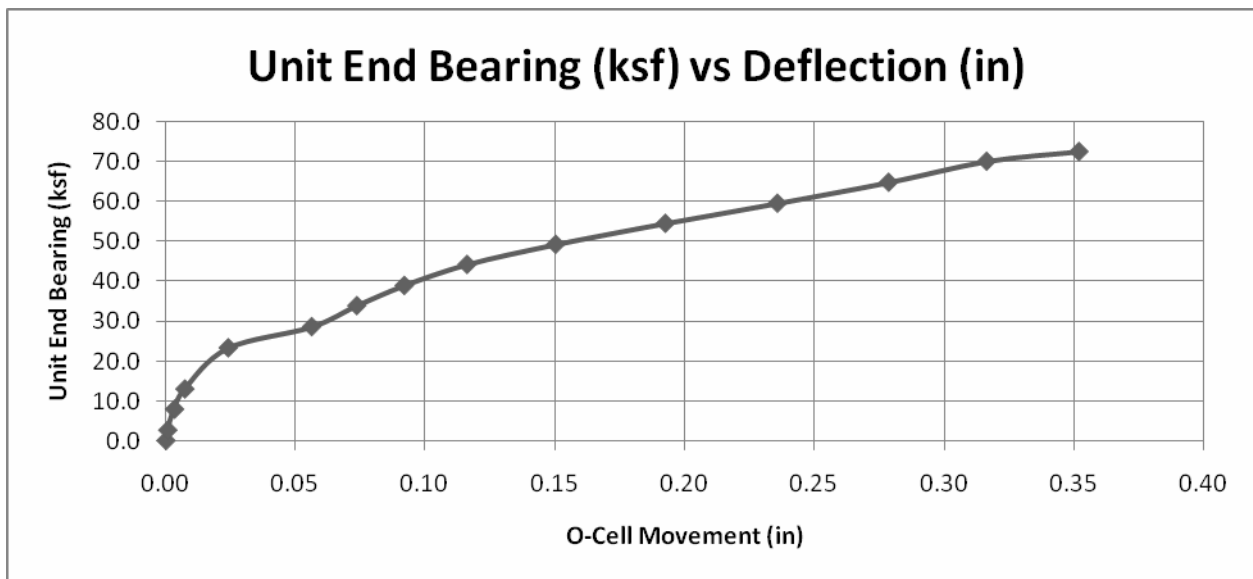
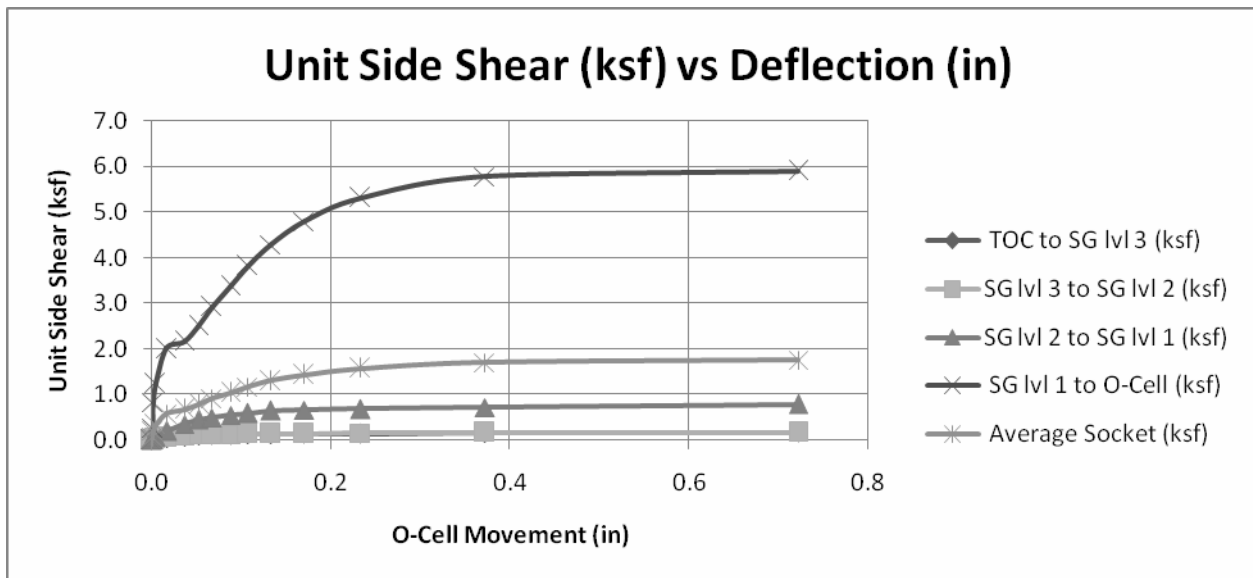


Shaft Summary

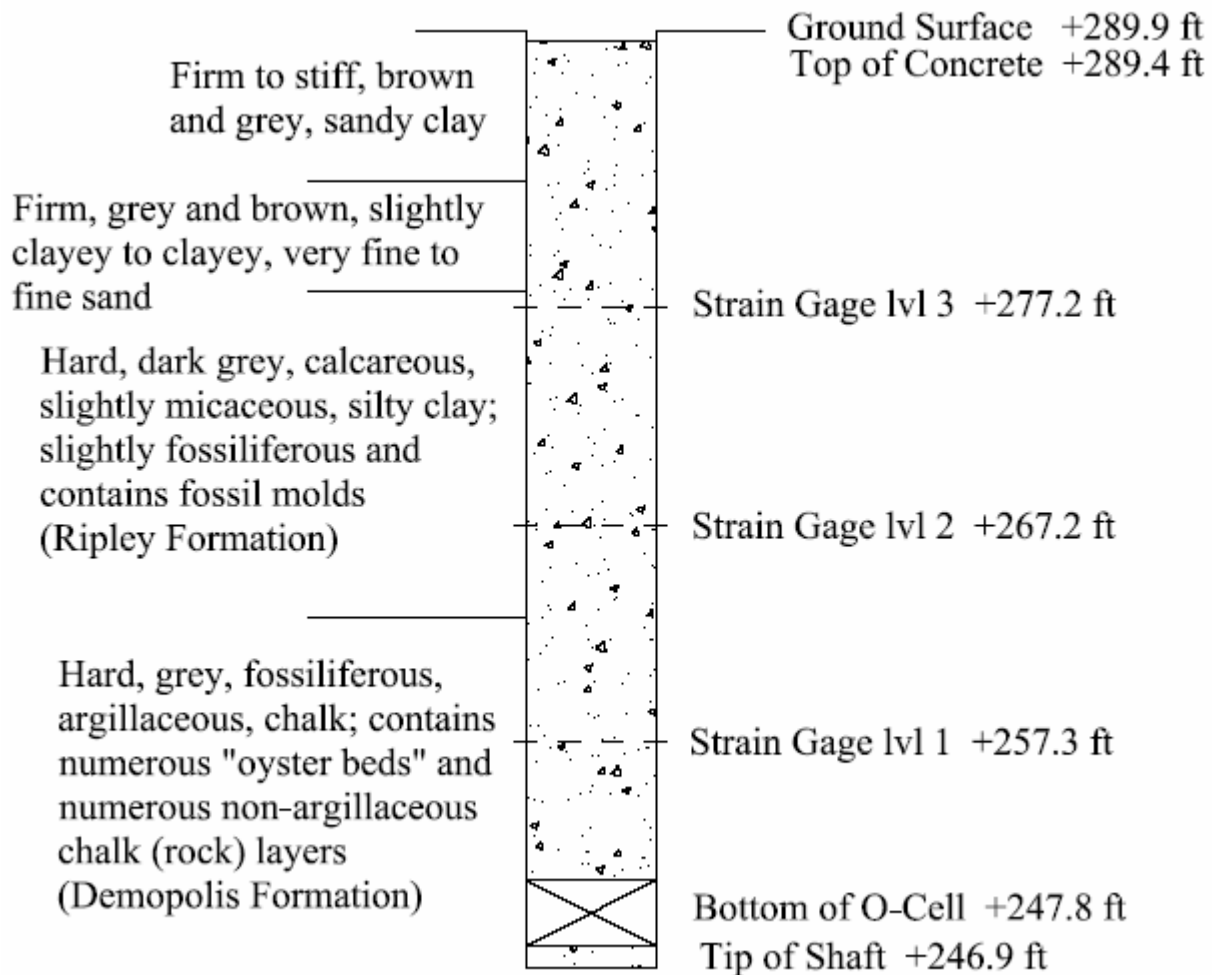
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
42.0	29.0	1.0	28.0	21.0	716.00	0.72	562.00	0.00

Data Summary

Elevation (ft.)	Material	Location	Unconfined Compressive Strength, q_u (ksf)	Average ϕ ($^\circ$)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
254.0	Limonic Clay	Top of Shaft	2.00	26			0.34	0.72
244.1	Limonic Clay	Strain Gage lvl 3					0.6	
239.1	Hard Calcareous Clay	Strain Gage lvl 2	6.14	0			2.2	
234.1	Hard Calcareous Clay	Strain Gage lvl 1	27.92	0			5.92	
226.0	Basal Formation (Hard Clayey Silt and Silty Clay)	Top of O-Cell						
225.0	Basal Formation (Hard Clayey Silt and Silty Clay)	Bottom of Shaft			67.8	0.35		



The test shaft was constructed on June 13, 1998. The 48-in test shaft was constructed to a total length of 43.0 ft. There were no unusual problems during the construction of the test shaft.

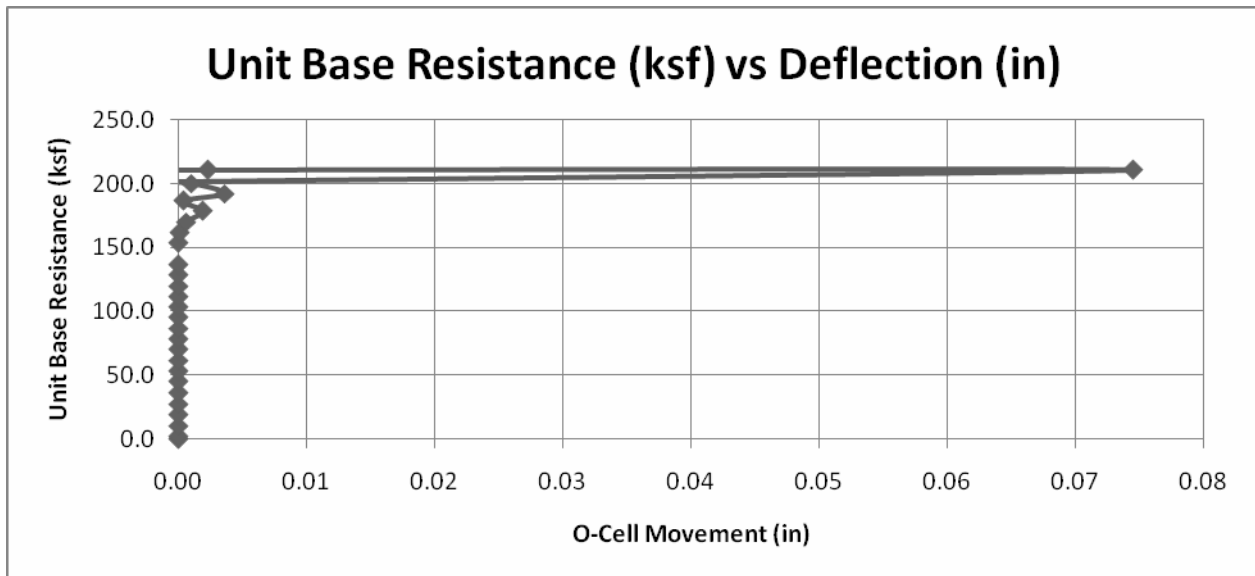
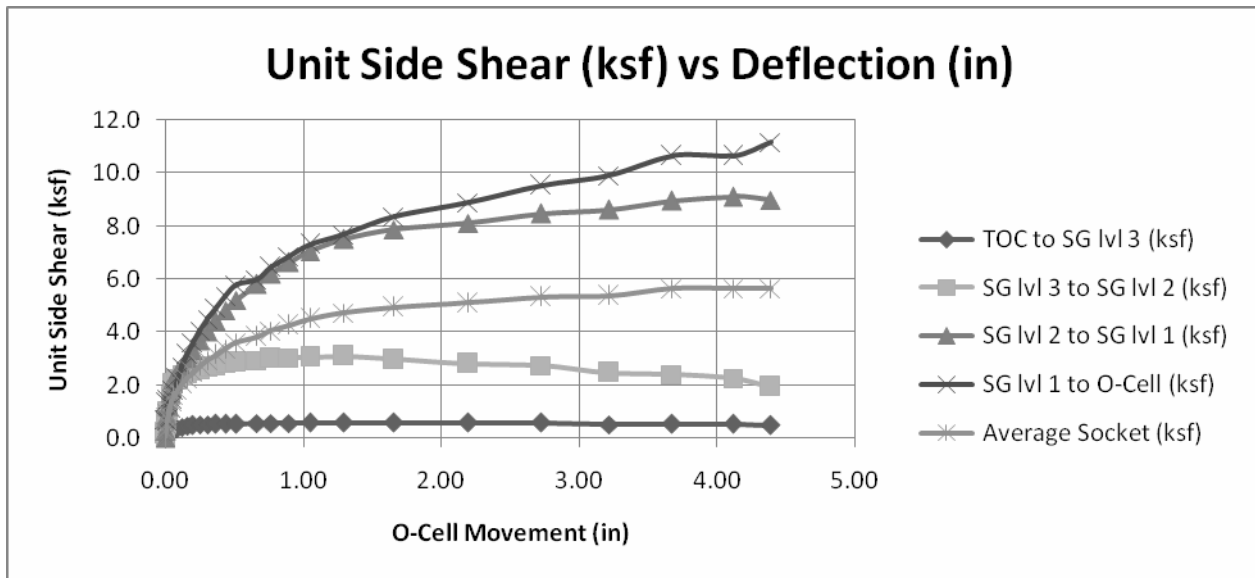


Shaft Summary

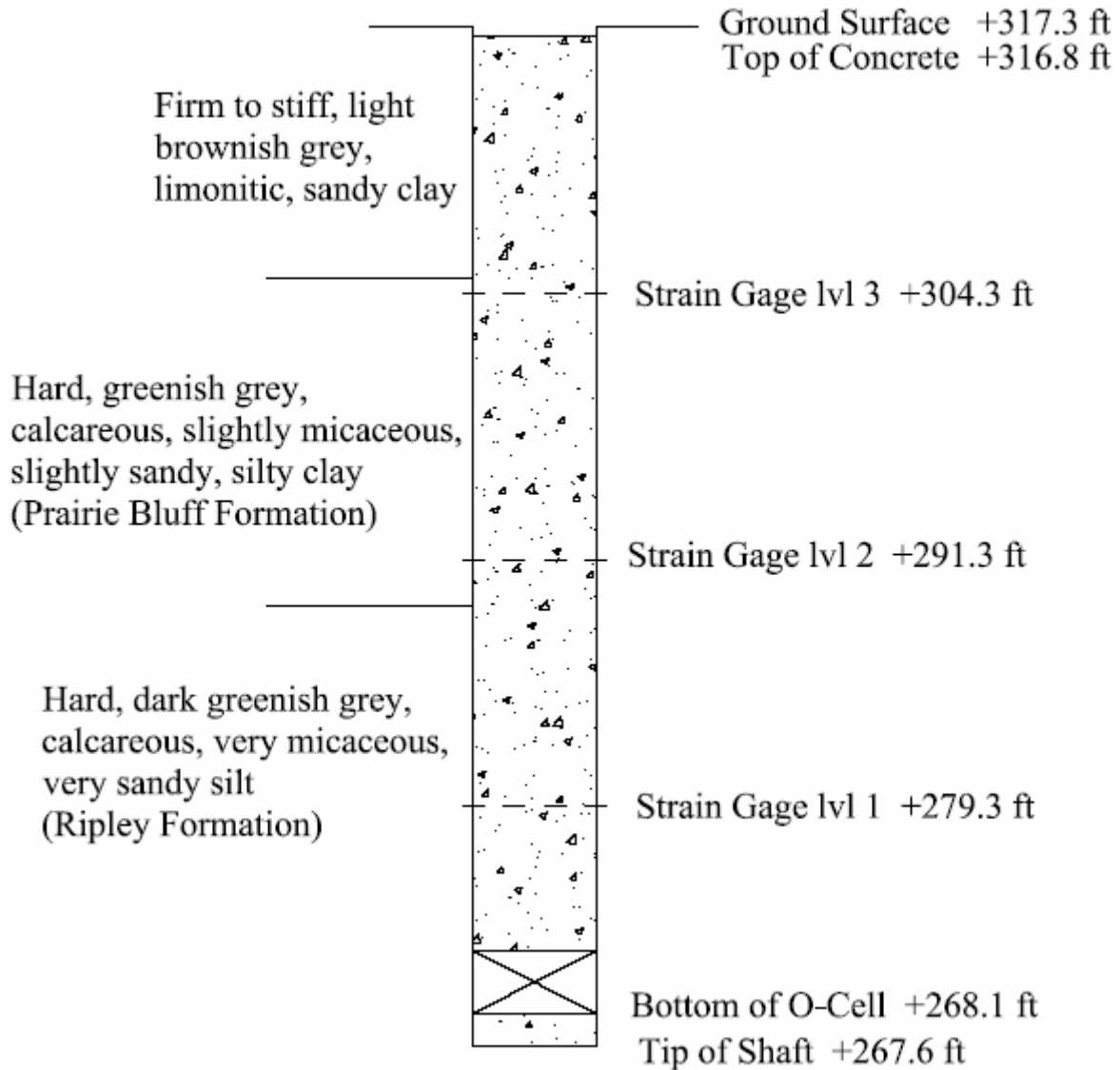
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
48.0	43.0	0.8	42.1	26.0	2770.00	4.38	0.00	2640.00

Data Summary

Elevation (ft.)	Material	Location	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
289.9	Firm, sandy, clay	Top of Shaft	2.21					0.58	3.67
281.4	Very fine to fine, sand		1.17						
277.2	Hard, silty, clay	Strain Gage lvl 3	4.50	36	10.9			3.06	
267.2	Hard, silty, clay	Strain Gage lvl 2						7.04	
262.4	Hard, argillaceous chalk		12.49		38.6				
257.3	Hard, argillaceous chalk	Strain Gage lvl 1						7.32	
247.8	Hard, argillaceous chalk	Bottom of O-Cell							
247.0	Hard, argillaceous chalk	Bottom of Shaft				214	0.11		



The test shaft was constructed on June 23, 1998. The 48-in test shaft was constructed to a total length of 39.7 ft. There were no unusual problems during the construction of the test shaft.

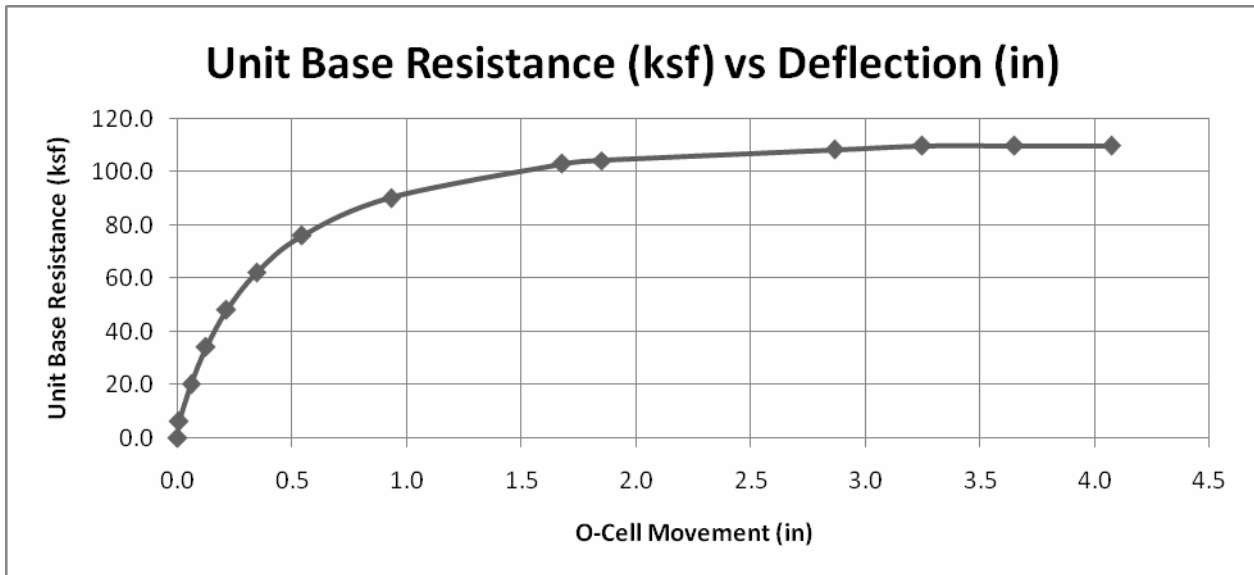
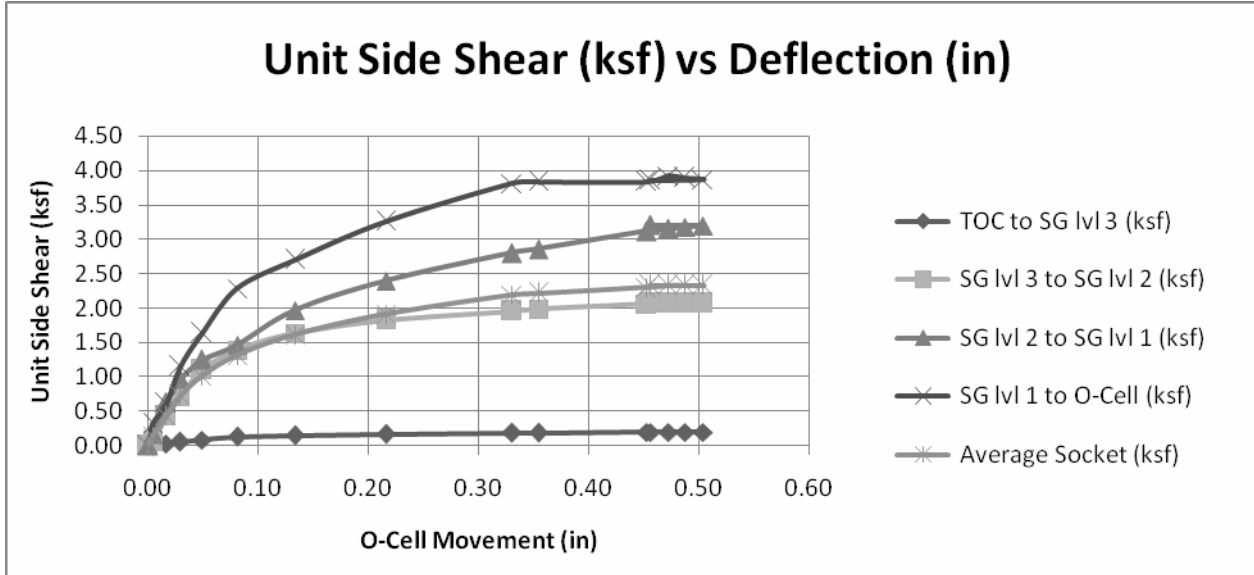


Shaft Summary

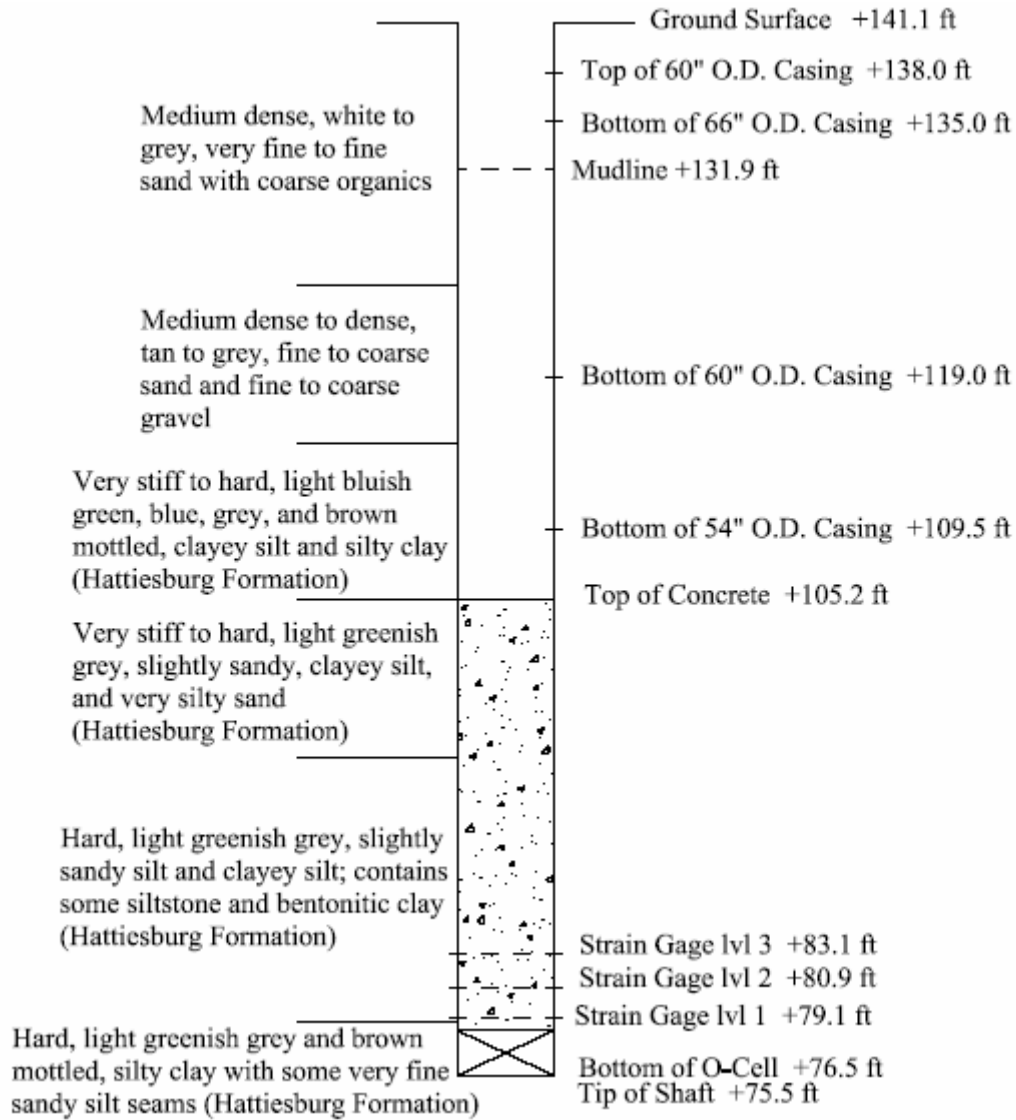
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
48.0	39.7	0.5	39.2	26.0	1396.00	0.51	1320.00	800.00

Data Summary

Elevation (ft.)	Material	Location	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
316.8	Firm, sandy, clay	Top of Shaft	2.46					0.2	0.47
305.0	Hard, silty, clay		4.00	35	28.8				
304.3	Hard, silty, clay	Strain Gage lvl 3						2.06	
291.3	Hard, silty, clay	Strain Gage lvl 2						3.16	
289.0	Hard, sandy, silt		3.00	30	27.1				
279.3	Hard, sandy, silt	Strain Gage lvl 1						3.92	
268.1	Hard, sandy, silt	Bottom of O-Cell							
267.6	Hard, sandy, silt	Bottom of Shaft				108	3.25		



The test shaft was constructed on August 5, 1998. The 54-in test shaft was constructed dry with a total length of 29.8 feet. No unusual problems occurred during the construction of the test shaft.



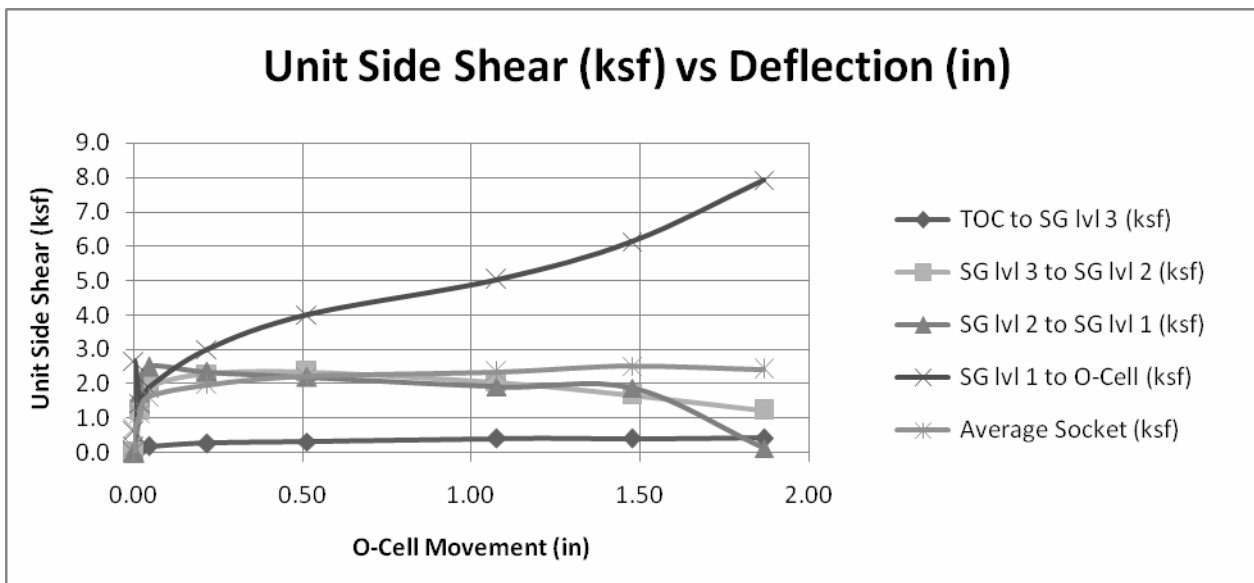
Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
54.0	29.8	1.0	28.5	21.0	464.00	1.87	320.00	396.00

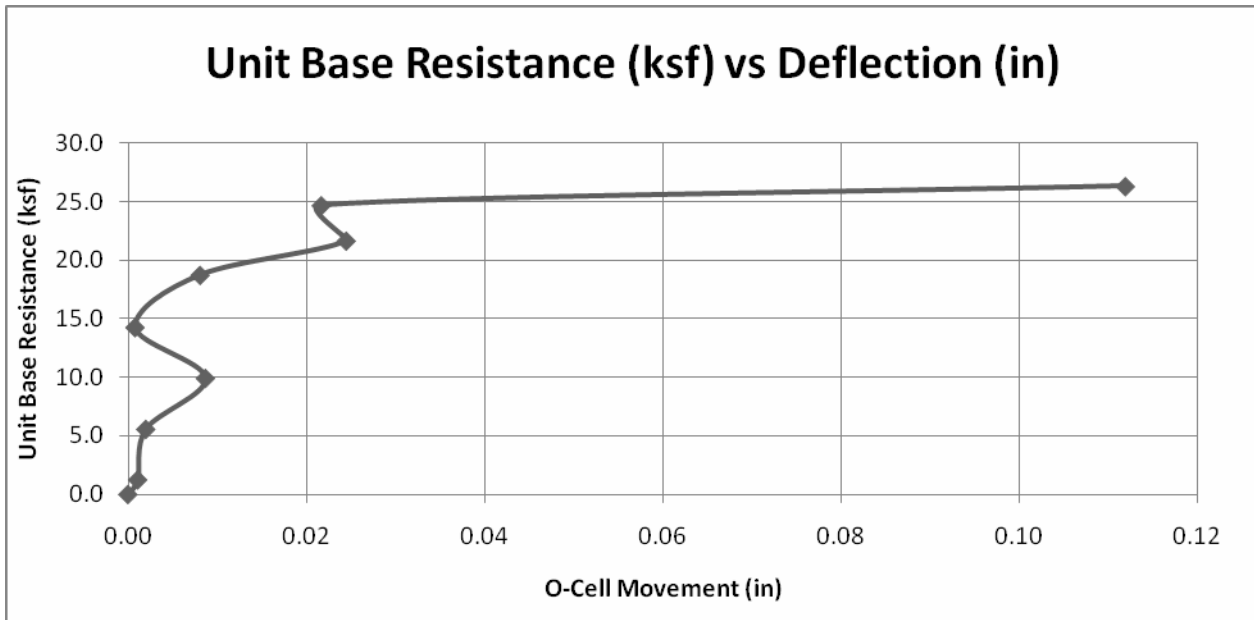
Data Summary

Elevation (ft.)	Material	Location	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
131.9	Very fine to fine sand	Top of Mudline						
124.0	Sand and gravel		0	37				
114.1	Clayey silt and silty clay		3.86	9				
107.5	Clayey silt and silty sand		3.55	12				
105.2	Clayey silt and silty sand	Top of Concrete					0.4	1.87
94.4	Sandy silt and clayey silt		5.53	0				
89.5	Clayey silt and silty sand		3.55	12				
83.1	Clayey silt and silty sand	Strain Gage lvl 3	3.55	12			2.1	
80.9	Clayey silt and silty sand	Strain Gage lvl 2	3.55	12			1.92	
79.1	Clayey silt and silty sand	Strain Gage lvl 1	3.55	12			4.96	
76.5	Clayey silt and silty sand	Bottom of O-Cell						
75.5	Clayey silt and silty sand	Bottom of Shaft			24.8	0.74		

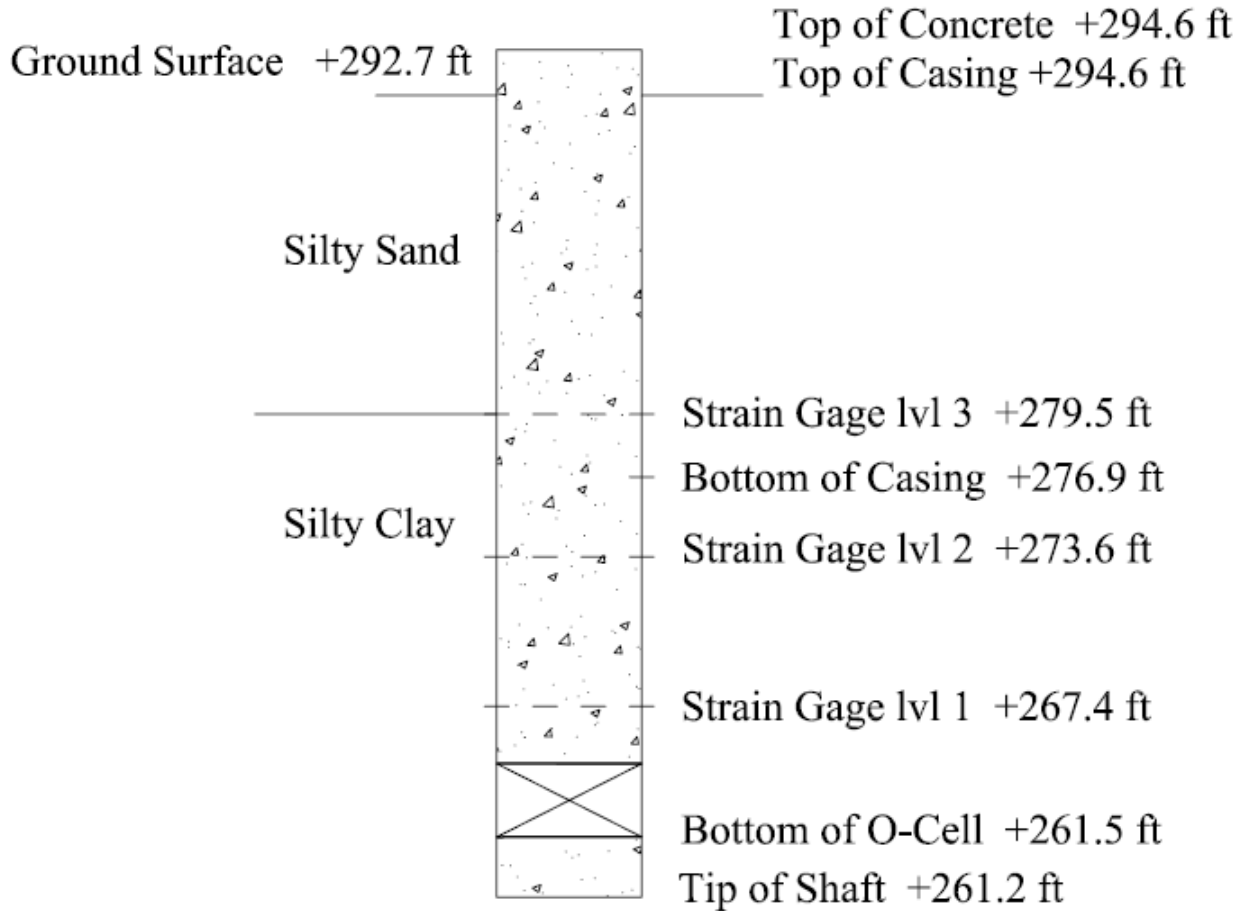
Unit Side Shear (ksf) vs Deflection (in)



Unit Base Resistance (ksf) vs Deflection (in)



The test shaft was constructed on June 11, 1999. The 54-in test shaft was constructed dry with a total length of 33.3 feet. No unusual problems occurred during the construction of the test shaft.

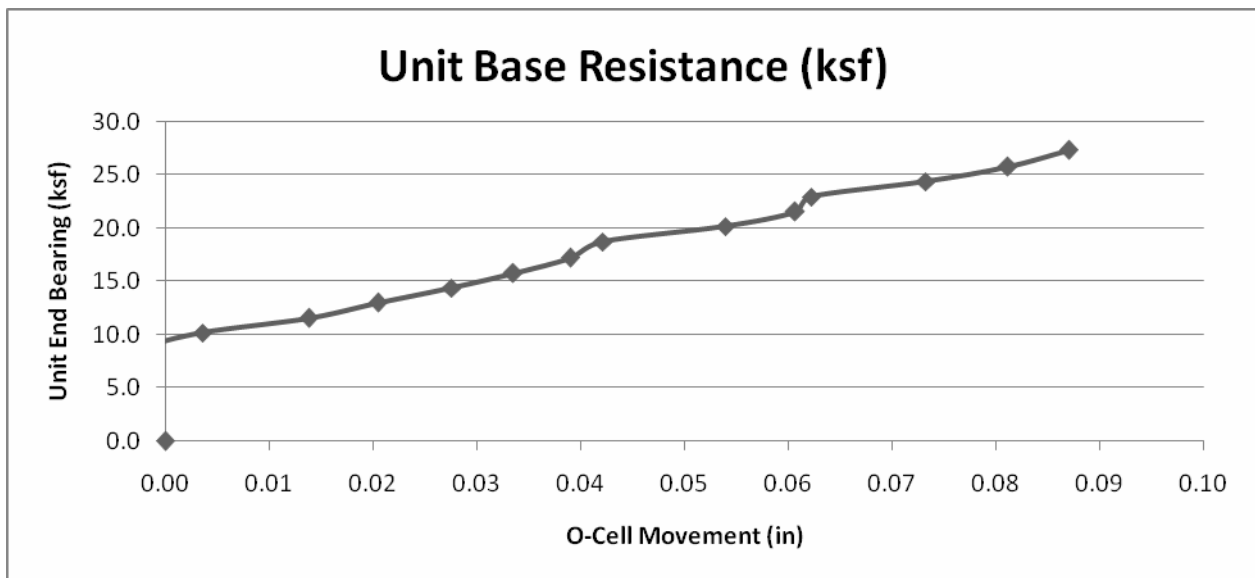
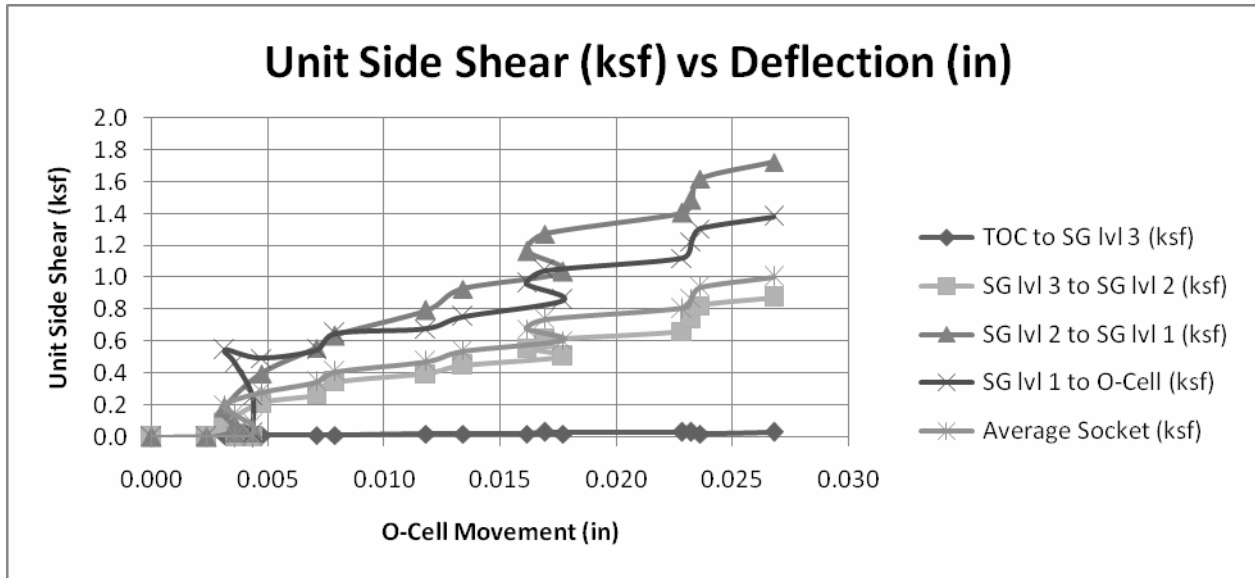


Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)
54.0	33.2	0.1	33.1	13.0	436.00	0.03

Data Summary

Elevation (ft.)	Material	Location	SPT "N" (Blows / ft)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
294.6	Silty Sand	Top of Concrete				0.03	0.027
279.5	Silty Sand	Strain Gage lvl 3				0.88	
273.6	Silty Clay	Strain Gage lvl 2	100			1.72	
267.4	Silty Clay	Strain Gage lvl 1	100			1.39	
261.5	Silty Clay	Bottom of O-Cell	100				
261.2	Silty Clay	Tip of Shaft	100	27.4	0.087		



A total of 12 dedicated test shafts were constructed along the 7+ mile long project alignment. Ten of the shafts were for axial load tests, while two of the shafts were for lateral load tests. The shafts were installed to various lengths into the Cooper Marl, utilizing various casing and drilling methods. A list of the axial test shafts is provided in the table below.

Summary of Test Shafts
From Camp, W.M., Brown, D.A. and Mayne, P.W. (2002)

Site	Shaft	Dia. (in)	Total Length (ft)	Casing Length (m)	Casing Type	Drilling Fluid	Loading
Mount Pleasant	MP1	96	158.1	58.1	P	bent.	2 level O-cell, lateral w/ liquefaction
	MP2	96	157.0	71.0	T	polymer	2 level O-cell, lateral
	MP3	96	110.0	71.9	P	polymer	O-cell, axial & lateral Statnamic w/ liquef.
	MP4	72	107.0	69.0	P	dry	O-cell, axial Statnamic
Charleston	C1	96	157.3	73.6	P	water	2 level O-cell, lateral, lateral Statnamic
	C2	96	157.5	76.5	P	polymer	2 level O-cell, lateral Statnamic
	C3	96	112.6	72.9	T	polymer	O-cell, axial Statnamic, lateral
	C4	72	108.7	69.0	P	dry	O-cell, axial & lateral Statnamic
Drum Island	DI1	96	158.5	72.6	P	dry	2 level O-cell, lateral Statnamic
	DI2	72	115.2	71.9	P	dry	O-cell, lateral Statnamic

All of the shafts were constructed to bear into the Cooper Marl, a stiff calcareous marine clay that can vary to a silt and contain various amounts of sand and/or shells. Standard Penetration Test N-values in the Cooper Marl can range from the low teens to 50/3". Samples from borings drilled at the test shaft sites had undrained shear strength (S_u) values of 2.3 to 5.8 ksf. CU triaxial tests of samples yielded $c' = 0$ to 1.8 ksf and $\phi' = 16.7$ to 32.5 degrees.

The results of the O-Cell tests are summarized in the table below. Ultimate unit side shear values in the Cooper Marl ranged from 2.0 to 6.5 ksf. Unit end bearing values ranged from 43.5 to 80 ksf.

Summary of O-Cell Load Test Results

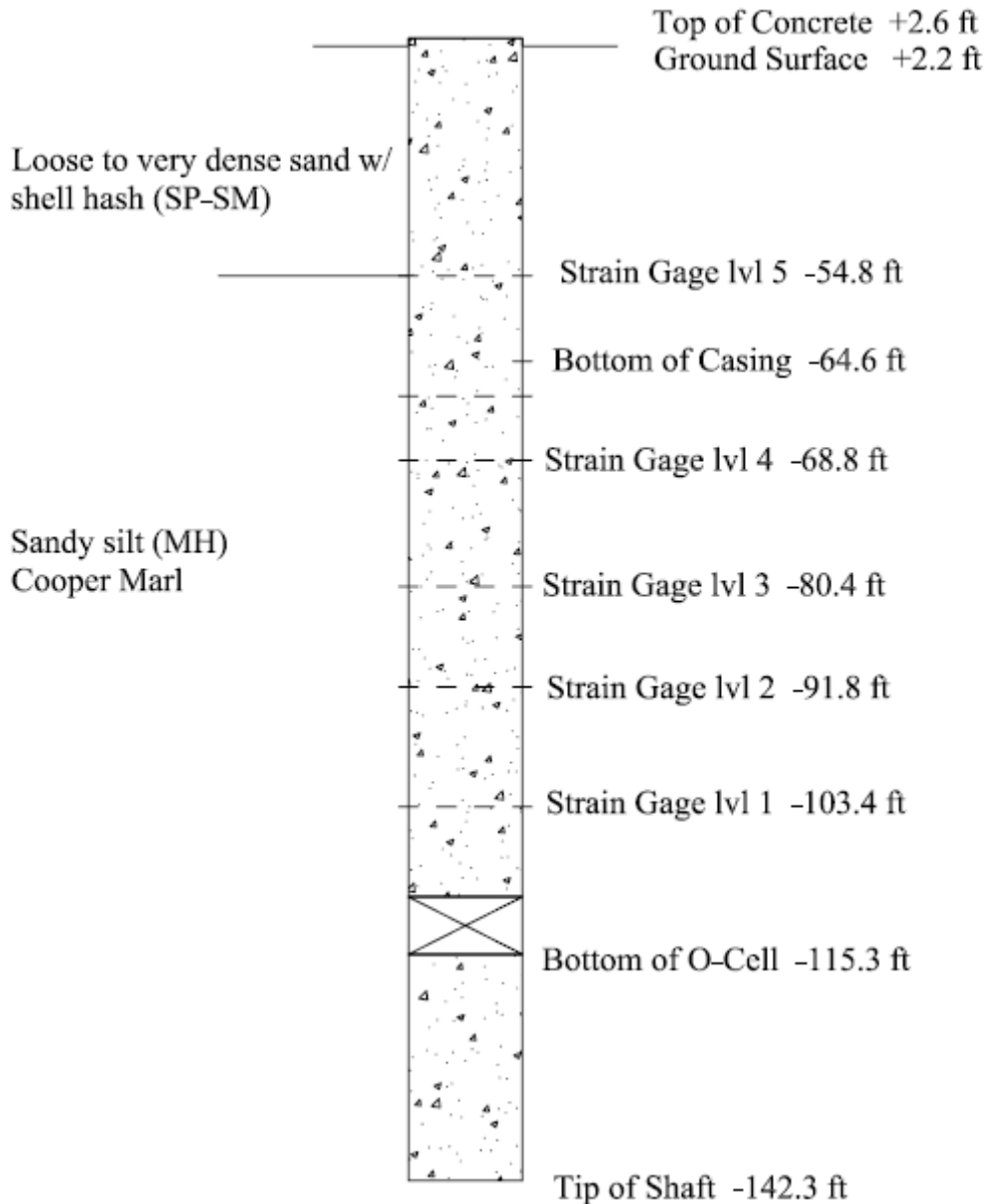
Test Shaft	Unit End Bearing (ksf)	Max Deflection Down for EB (in)	Highest Unit Side Shear Cap (ksf)	Max Deflection Up for SS (in)
C-1	79.6	6.20	6.47	0.67
C-2	74.7	5.06	5.83	0.36
C-3	43.5	3.25	2.96	0.04
C-4	50.6	4.31	2.50	0.06
DI-1	58.8	4.94	3.43	1.43
DI-2	56.0	3.38	3.81	0.06
MP-1	74.9	4.50	5.24	1.58
MP-2	68.3	3.75	6.01	0.83
MP-3	54.4	3.73	3.42	0.08
MP-4	63.4	4.22	2.10	0.08

Detailed data on the Cooper Marl and analysis of the load test program is contained in:

Camp, W.M., Brown, D.A. and Mayne, P.W. (2002). "Construction Method Effects on Axial Drilled Shaft Performance", *Proceedings of the International Deep Foundations Congress 2002*, Orlando, FL, ASCE GSP 116, Vol 1, 195-208.

Camp, W.M., Mayne, P.W. and Brown, D.A (2002). "Drilled Shaft Axial Design Values: Predicted Versus Measured Response in a Calcareous Clay", *Proceedings of the International Deep Foundations Congress 2002*, Orlando, FL, ASCE GSP 116, Vol 2, 1518-1532.

The production test shaft was constructed on June 12, 2000. The 48-in test shaft was constructed dry with a total length of 144.5 ft. The shaft was started with an O.D. 48-in permanent casing. Both an auger and bucket were used for drilling and cleaning the shaft. Concrete was gravity poured through an O.D. 5-in pipe into the base of the shaft. After the top of the concrete reached an elevation of -114.4 ft, the reinforcing cage was inserted in the shaft and the O-cell was located at an elevation of -115.2 ft. The O.D. pipe was reinserted and concrete was gravity poured into the shaft until it reached an elevation of +2.6 ft. No unusual problems occurred during the construction of the shaft.

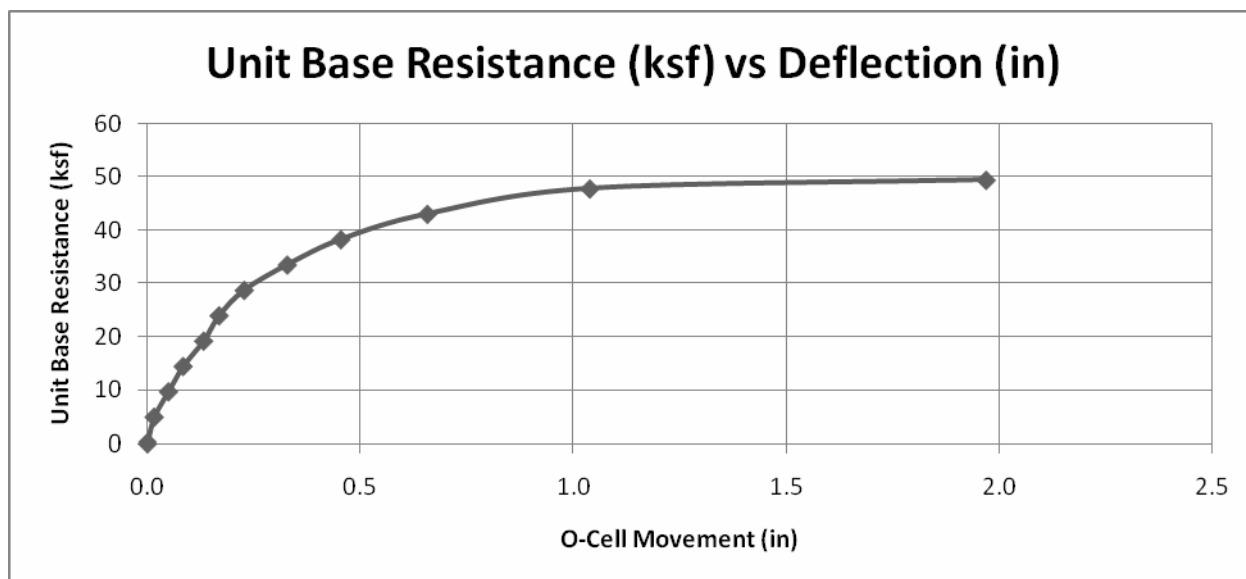
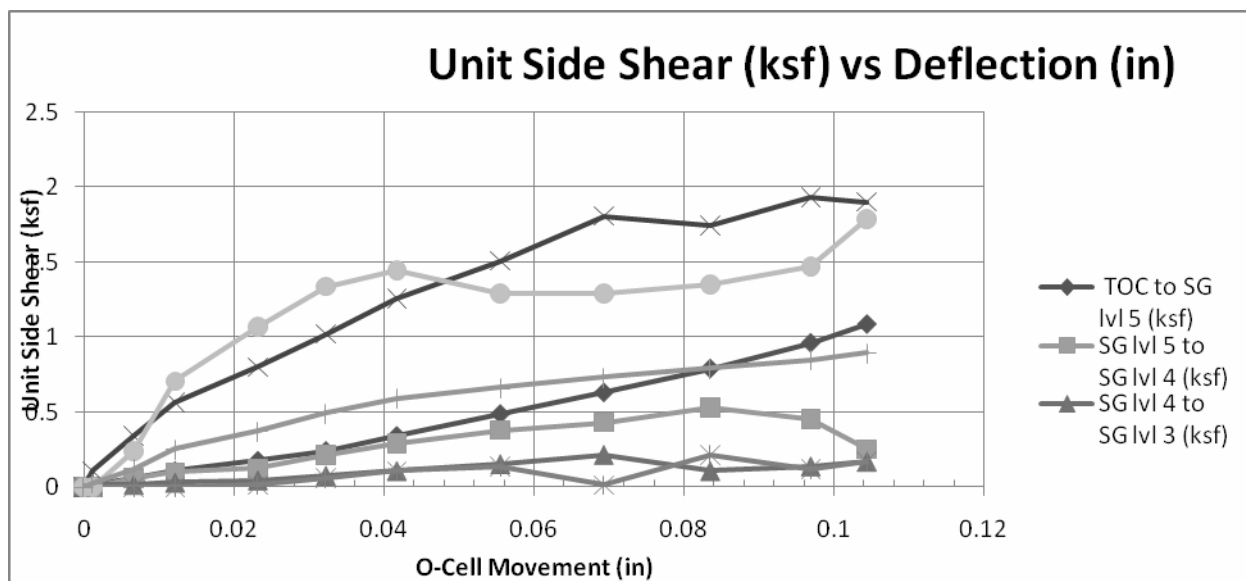


Shaft Summary

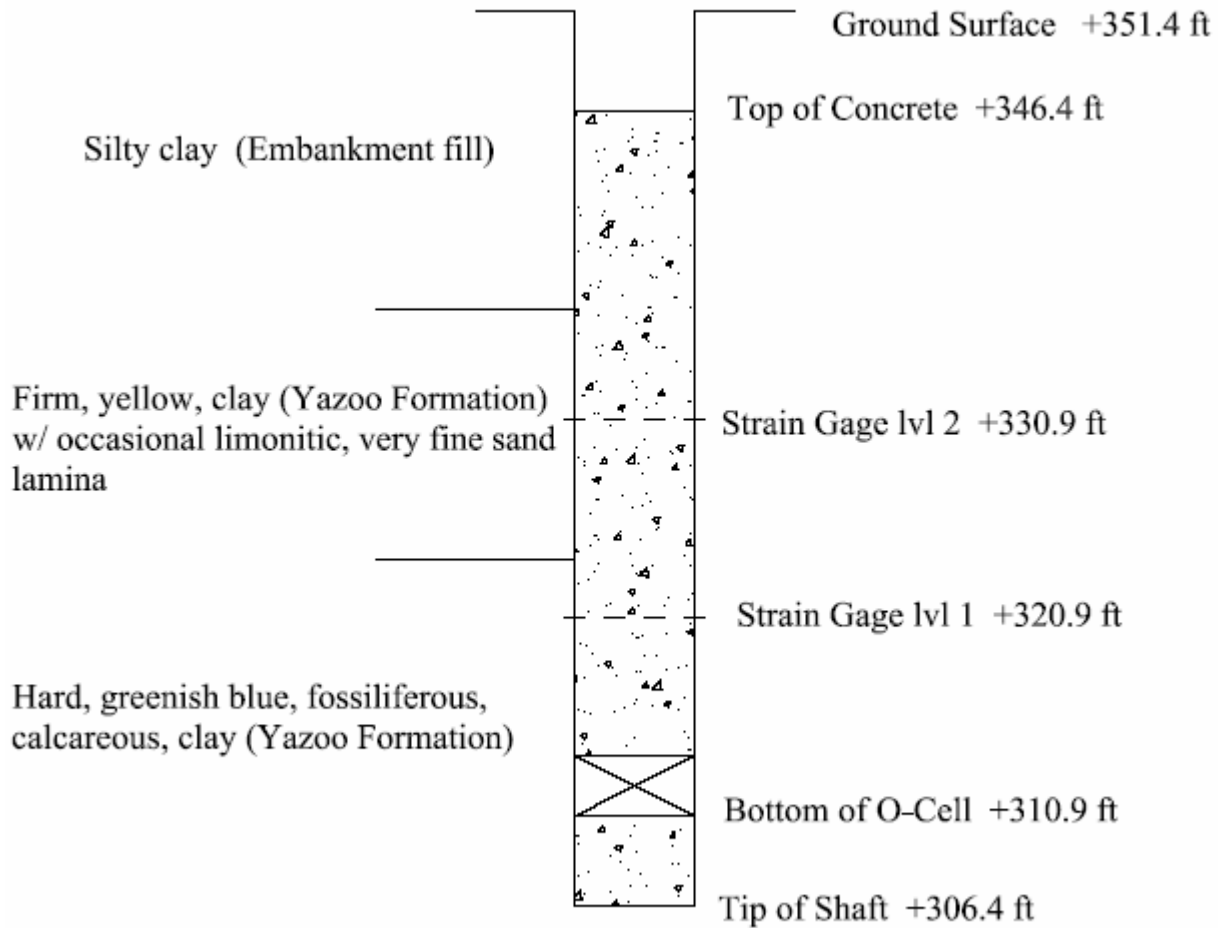
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
48.0	144.8	26.5	117.7	26.00	1569.00	0.10	1570.00	850.00

Data Summary

Elevation (ft.)	Material	Location	SPT "N" (Blows / ft)	Unconfined Compressive Strength, q_u (ksf)	Unit Wt. (lb/cu ft)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
2.6	Sand and silty sand w/ shell hash (SP-SM)	Top of Concrete						1.1	0.1
-41.3	Sand and silty sand w/ shell hash (SP-SM)		43						
-54.8	Sand and silty sand w/ shell hash (SP-SM)	Strain Gage lvl 5						0.25	
-57.4	Top of Cooper Marl								
-57.7	Cooper Marl - Sandy Silt (MH)			4.1	69				
-63.0	Cooper Marl - Sandy Silt (MH)		16						
-64.6	Cooper Marl - Sandy Silt (MH)	Bottom of Casing							
-68.2	Cooper Marl - Sandy Silt (MH)		16						
-68.8	Cooper Marl - Sandy Silt (MH)	Strain Gage lvl 4						0.16	
-72.2	Cooper Marl - Sandy Silt (MH)		18						
-77.8	Cooper Marl - Sandy Silt (MH)			4.3	64.2				
-78.1	Cooper Marl - Sandy Silt (MH)		15						
-80.4	Cooper Marl - Sandy Silt (MH)	Strain Gage lvl 3						1.91	
-82.7	Cooper Marl - Sandy Silt (MH)		17						
-87.9	Cooper Marl - Sandy Silt (MH)		23						
-91.8	Cooper Marl - Sandy Silt (MH)	Strain Gage lvl 2						0.17	
-93.2	Cooper Marl - Sandy Silt (MH)		25						
-97.8	Cooper Marl - Sandy Silt (MH)		23	4.1	61.5				
-103.0	Cooper Marl - Sandy Silt (MH)		19						
-103.4	Cooper Marl - Sandy Silt (MH)	Strain Gage lvl 1						2.79	
-107.6	Cooper Marl - Sandy Silt (MH)		28						
-112.9	Cooper Marl - Sandy Silt (MH)		27						
-115.3	Cooper Marl - Sandy Silt (MH)	Bottom of O-Cell							
-117.8	Cooper Marl - Sandy Silt (MH)			5.9	78.9				
-118.1	Cooper Marl - Sandy Silt (MH)		21						
-124.0	Cooper Marl - Sandy Silt (MH)		21						
-142.3	Cooper Marl - Sandy Silt (MH)	Tip of Shaft				49.4	1.97		
						49.4	1.97		



The dedicated test shaft was constructed on April 12, 2001. The 24-in test shaft was constructed dry to a total depth of 45 ft. An auger was used for drilling and cleaning the shaft. A seating layer of concrete was placed in the base of the shaft by gravity pour, the reinforcing cage was inserted in the shaft and the remainder of the concrete was placed by gravity pour until reaching elevation +346.4 ft. No unusual problems occurred during construction of the shaft.

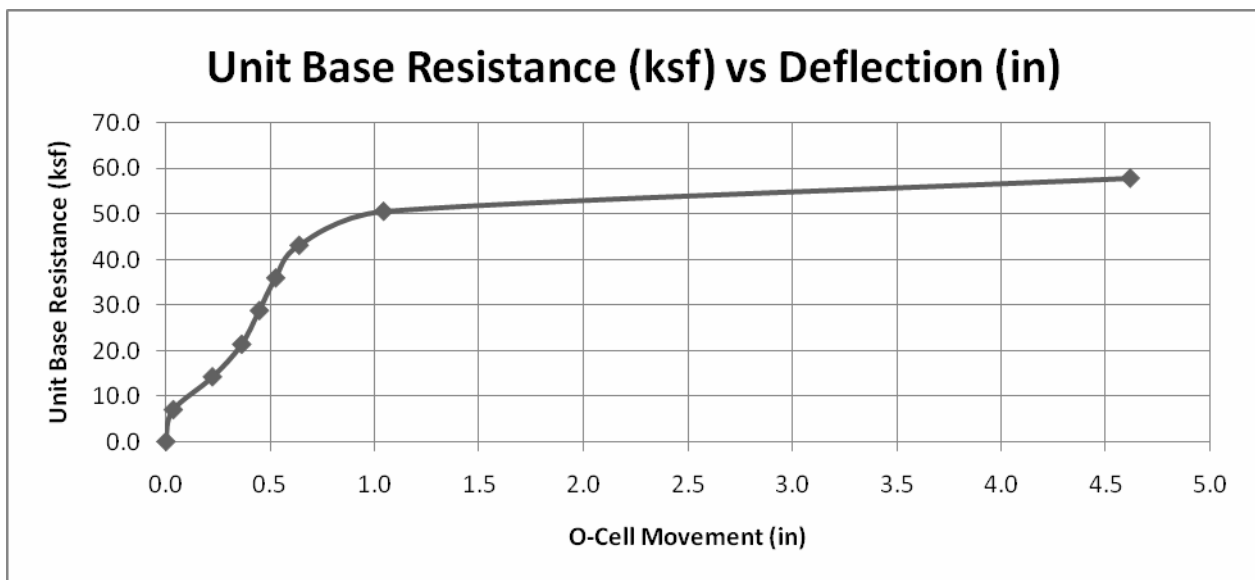
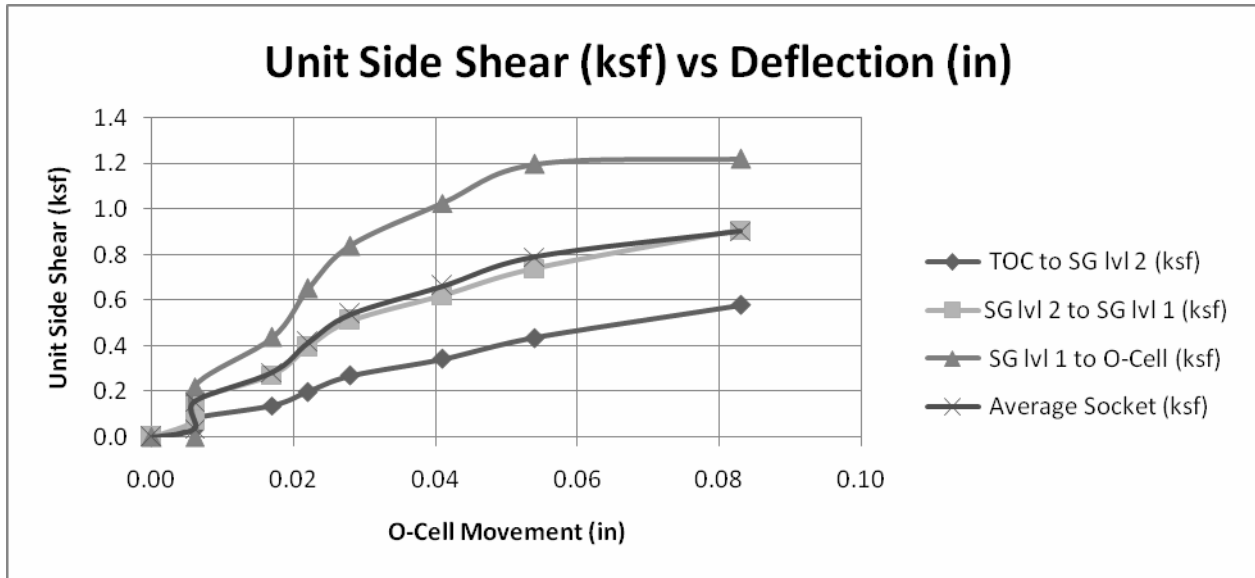


Shaft Summary

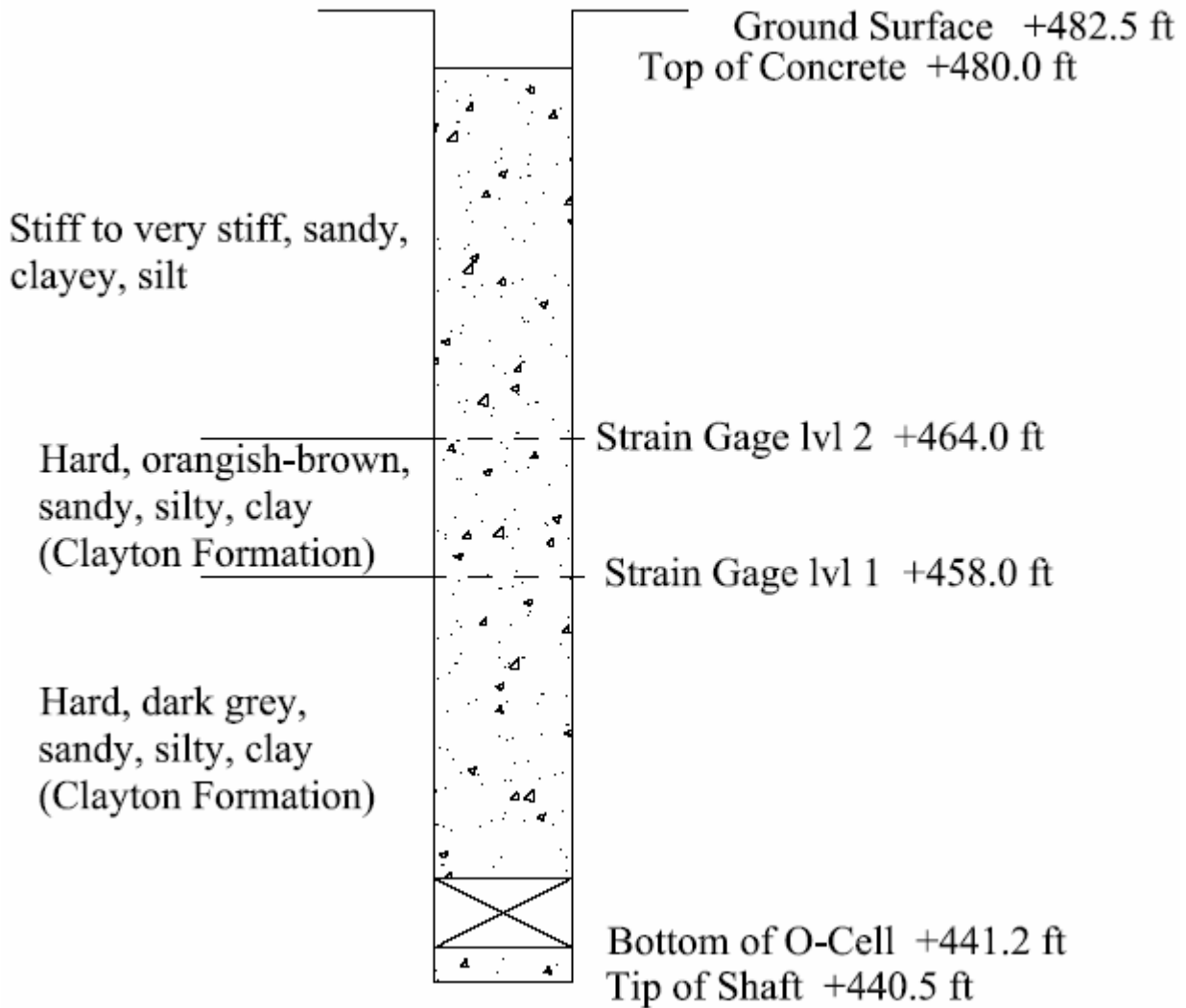
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
24.0	45.0	4.5	35.5	13.0	207.00	0.08	0.00	154.00

Data Summary

Elevation (ft.)	Material	Location	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
351.4	Yazoo Formation (Hard, tan, silty clay)	Ground Elevation	2.5	0					
346.4	Yazoo Formation (Hard, tan, silty clay)	Top of Concrete						0.32	0.083
330.9	Yazoo Formation (Hard, tan, silty clay)	Strain Gage lvl 2						0.83	
328.4	Yazoo Formation (Hard, bluish green, silty clay)		4.5	13					
322.4	Yazoo Formation (Hard, bluish green, silty clay)				11.1				
320.9	Yazoo Formation (Hard, bluish green, silty clay)	Strain Gage lvl 1						1.48	
310.9	Yazoo Formation (Hard, bluish green, silty clay)	Bottom of O-Cell							
306.4	Yazoo Formation (Hard, bluish green, silty clay)	Bottom of Shaft				52.6	4.613		



The dedicated test shaft was constructed on May 16, 2003. The 48-in test shaft was constructed dry to a total length of 42.0 ft. An auger and bucket were used for drilling and cleaning the shaft. After cleaning the base, approximately four linear feet of concrete was delivered to the base of the shaft by freefall. The reinforcing cage with attached O-cell assembly was then inserted into the excavation and into the wet concrete, and temporarily supported by a boom truck with the base of the O-cell at elevation +441.2 ft. Concrete continued to be delivered by freefall until the top of the concrete reached an elevation of +480.0 ft. No unusual problems occurred during the construction of the test shaft.

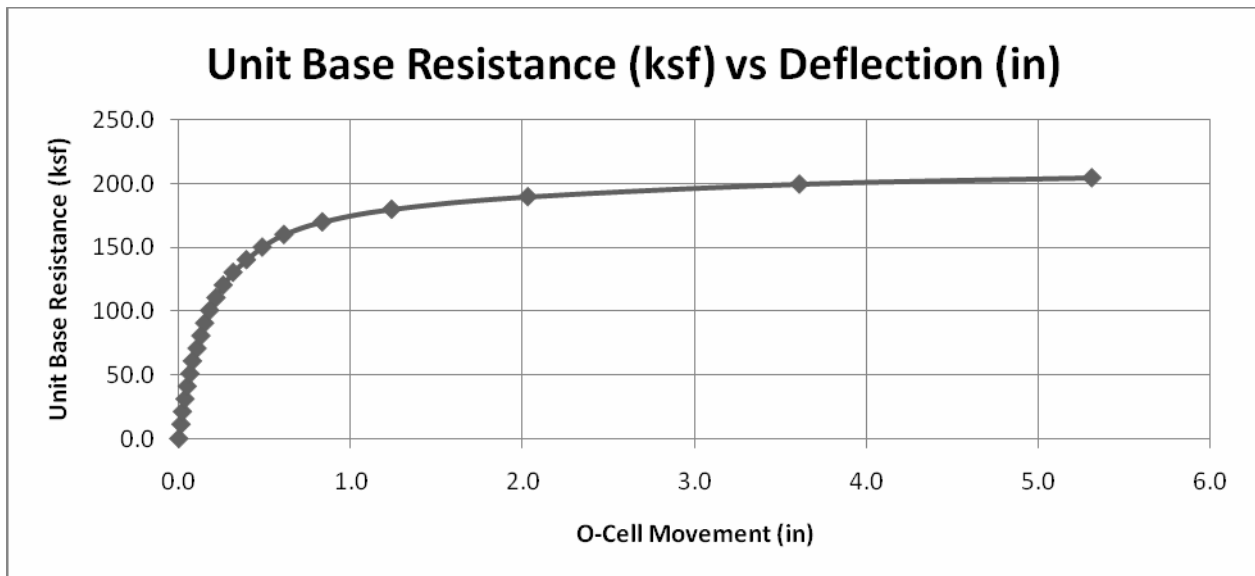
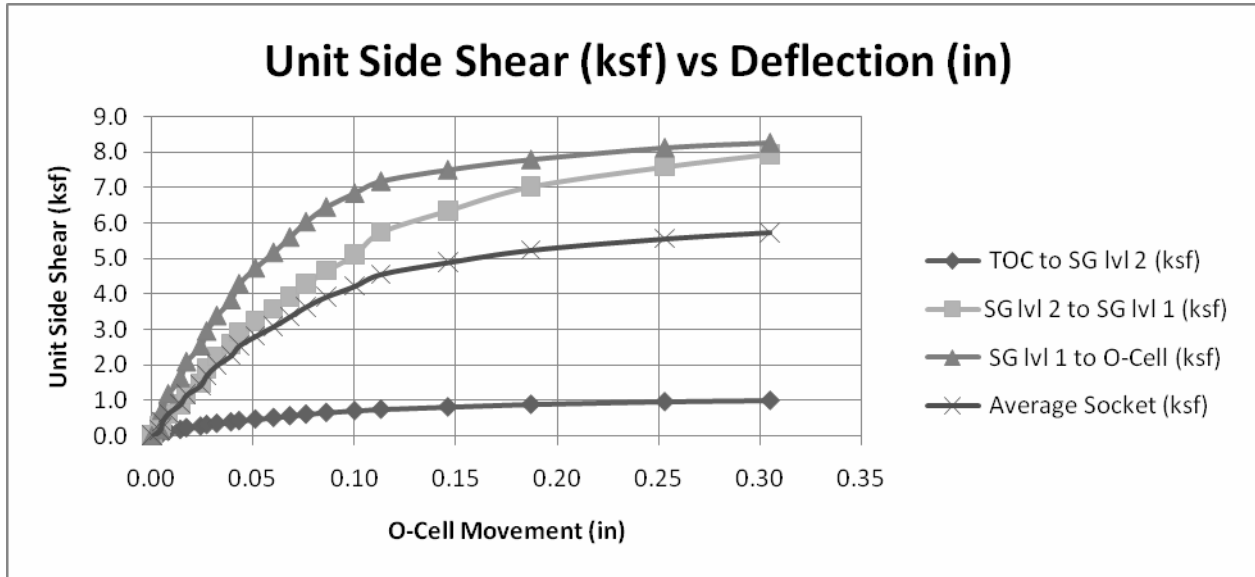


Shaft Summary

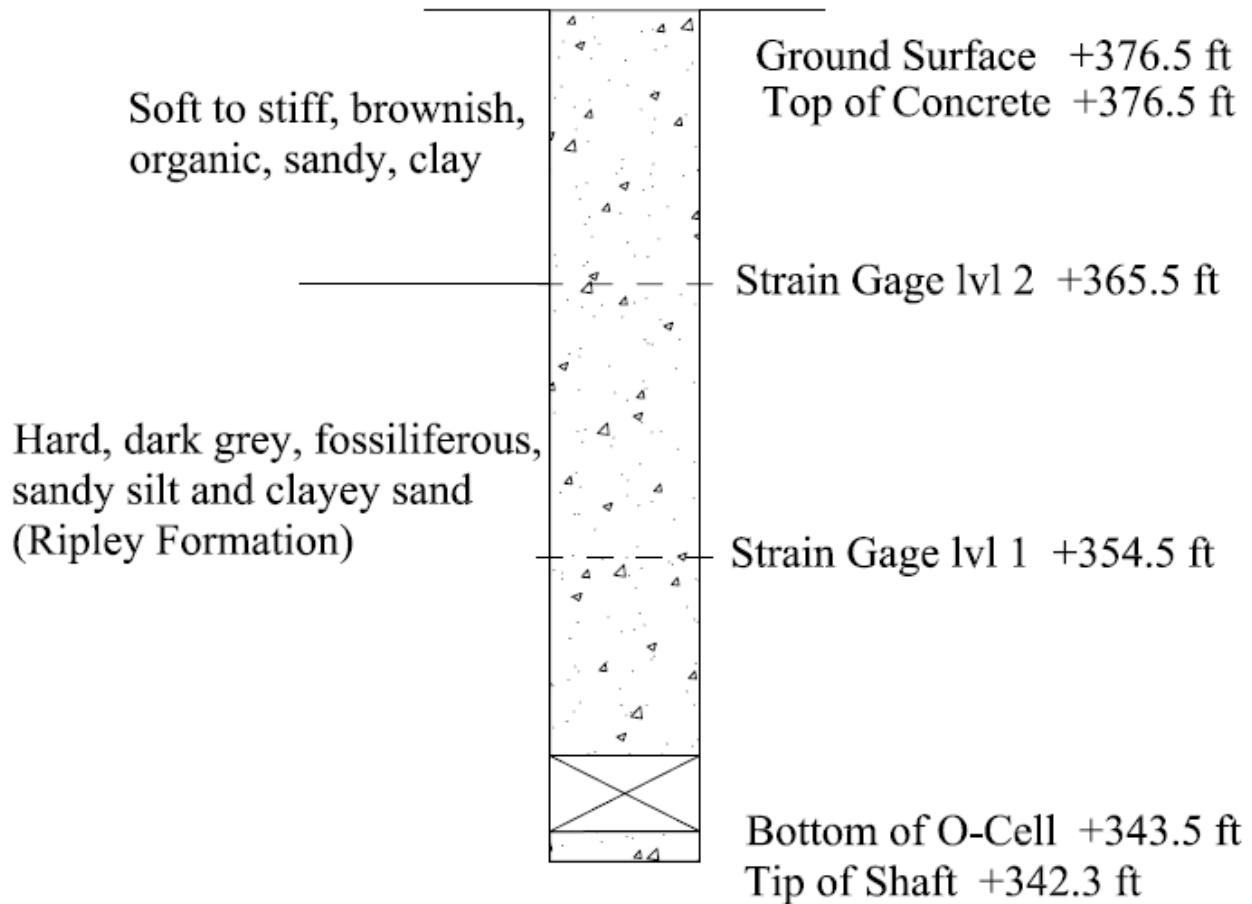
Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
48.0	42.0	0.67	38.8	21.25	2619.53	0.30	2480	1300

Data Summary

Elevation (ft.)	Material	Location	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
482.5	Stiff Clayey Silt	Ground Elevation	1.5	19				
480.0	Stiff Clayey Silt	Top of Concrete					0.8	0.305
464.0	Clayton Formation (Hard, clayey silt)	Strain Gage lvl 2	9.00	0			7.7	
458.0	Clayton Formation (Hard, clayey silt)	Strain Gage lvl 1					8.4	
457.8	Clayton Formation (Hard, silty clay)		9.00	0				
441.2	Clayton Formation (Hard, silty clay)	Bottom of O-Cell						
440.5	Clayton Formation (Hard, silty clay)	Bottom of Shaft			202.8	5.308		



The dedicated test shaft was constructed on May 28, 2003. The 48-in test shaft was constructed dry to a total length of 34.2 ft. An auger was used for drilling and cleaning the shaft. After cleaning the base, approximately four linear feet of concrete was delivered to the base of the shaft by freefall. The reinforcing cage with attached O-cell assembly was then inserted into the excavation and into the wet concrete, and temporarily supported by a boom truck with the base of the O-cell at elevation +342.5 ft. Concrete continued to be delivered by freefall until the top of the concrete reached an elevation of +376.5 ft. No unusual problems occurred during the construction of the test shaft.

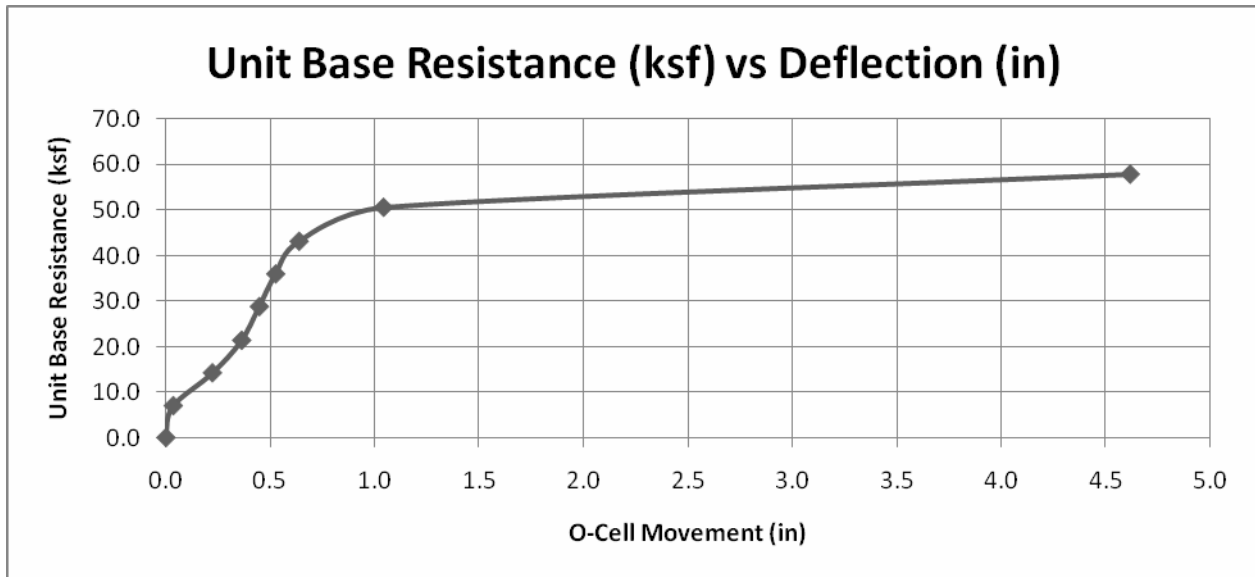
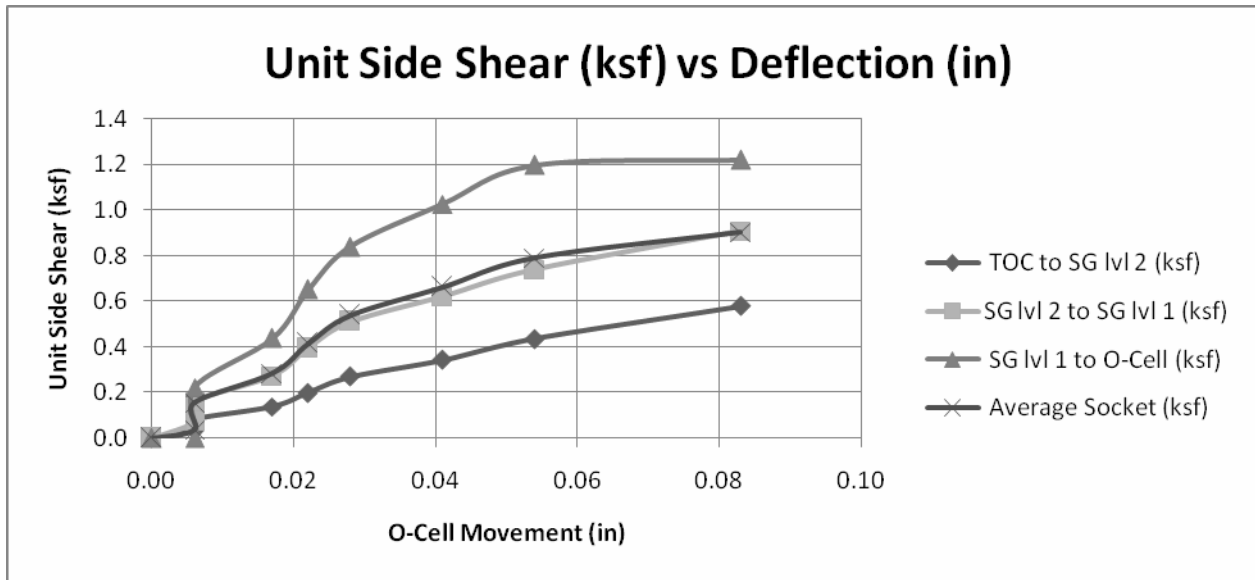


Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Shaft Length Below O-cell (ft)	Shaft Length Above O-cell (ft)	O-cell Size (in)	Max Net Load (kips)	Max Deflection Up (in)	SS Creep Load (kips)	EB Creep Load (kips)
48.0	34.2	1.2	33.0	21.25	3263.00	0.17	0.00	1400.00

Data Summary

Elevation (ft.)	Material	Location	Average Undrained Shear Strength, S_u (ksf)	Avg. ϕ (°)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
376.5	Soft, organic rich, sandy clay	Top of Shaft	1.00	0			0.1	0.157
365.5	Ripley Formation (Hard, very fossiliferous, sandy silt)	Strain Gage lvl 2	9.00	0			8.8	
354.5	Ripley Formation (Hard, very fossiliferous, sandy silt)	Strain Gage lvl 1					12.9	
343.5	Ripley Formation (Hard, very fossiliferous, sandy silt)	Bottom of O-Cell						
342.3	Ripley Formation (Hard, very fossiliferous, sandy silt)	Bottom of Shaft			221	2.241		



A dedicated test shaft was constructed through 65 feet of overlying silts and sands with a 32-foot socket into clay/chalk. The shaft was constructed using a temporary casing and slurry. Concrete was placed below the O-cell level creating a 21-foot test socket above the O-cell. There were no unusual problems during the construction of the test shaft.

Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Max Net Load (kips)	Max Deflection Up (in)
66.0	96.0	2200.00	0.18

Data Summary

Material	Location	Average Undrained Shear Strength, S_u (ksf)	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
Chalk	Strain Gage lvl 4	6.05	12.1			2.4	0.18
Chalk	Strain Gage lvl 3	6.05	12.1			1.4	0.18
Chalk	Strain Gage lvl 2	6.05	12.1			6	0.18
Chalk	Strain Gage lvl 1	6.05	12.1			7.6	0.18
Chalk	Base	6.05	12.1	11.0	0.66		
Chalk	Base	6.05	12.1	16.8	1.32		
Chalk	Base	6.05	12.1	45.0	3.3		
Chalk	Base	6.05	12.1	46.2	3.39		

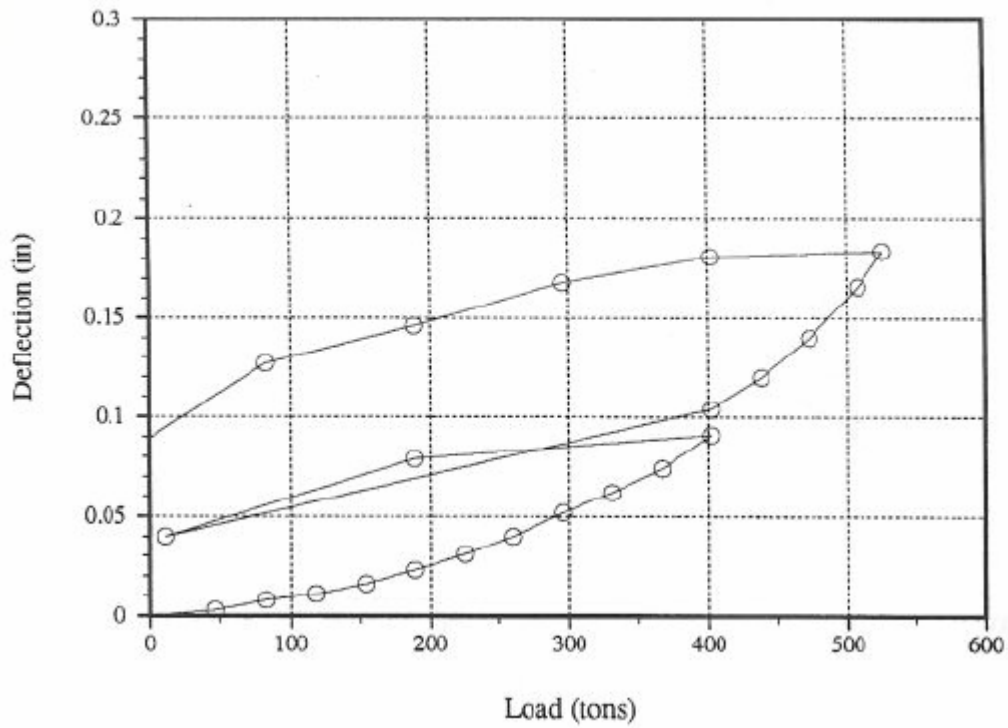


Figure A-24 - Load-deflection curve (shaft upward), Leake Co., MS

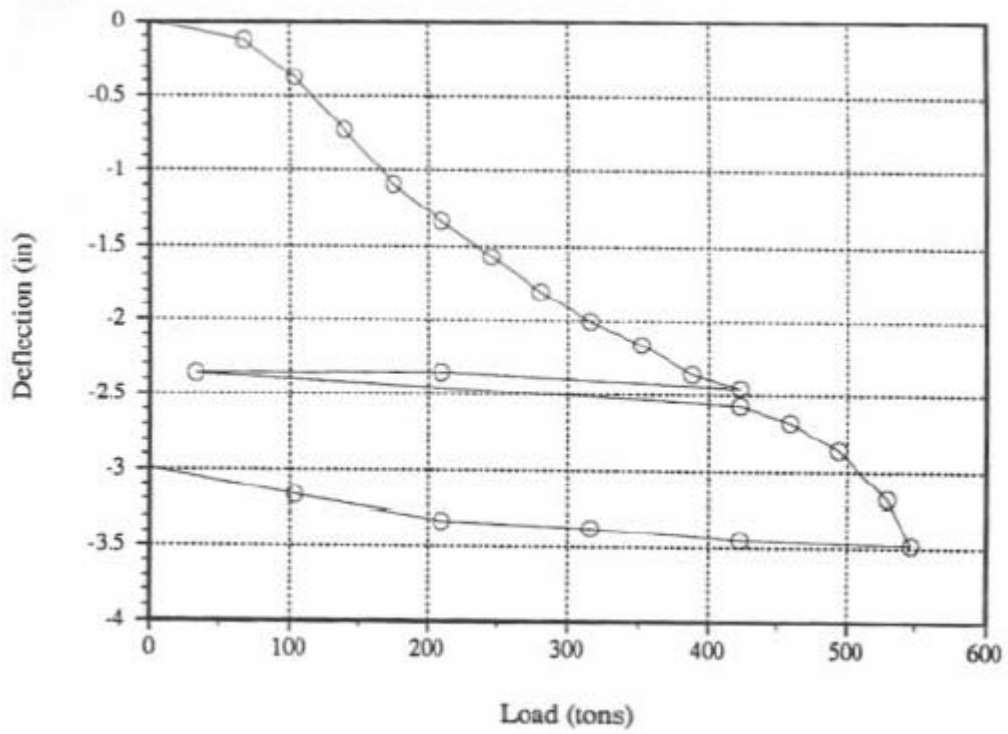


Figure A-25 - Load-deflection curve (cell downward), Leake Co., MS

A dedicated test shaft was constructed with a 10-foot socket into the Cooper Marl formation. A permanent casing was used to isolate the shaft from the overburden above the Cooper Marl. The shaft was constructed in the dry using a temporary casing. There were no unusual problems during the construction of the test shaft.

Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Max Net Load (kips)	Max Deflection Down (in)	Unit EB Cap (ksf)
24.0	150.0	1680.00	1.10	57.1

Data Summary

Material	Location	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
Cooper Marl	Socket	2.9			3.6	0.15
Cooper Marl	Base	2.9	18.00	0.24		
Cooper Marl	Base	2.9	27.00	0.48		
Cooper Marl	Base	2.9	28.56	1.10		

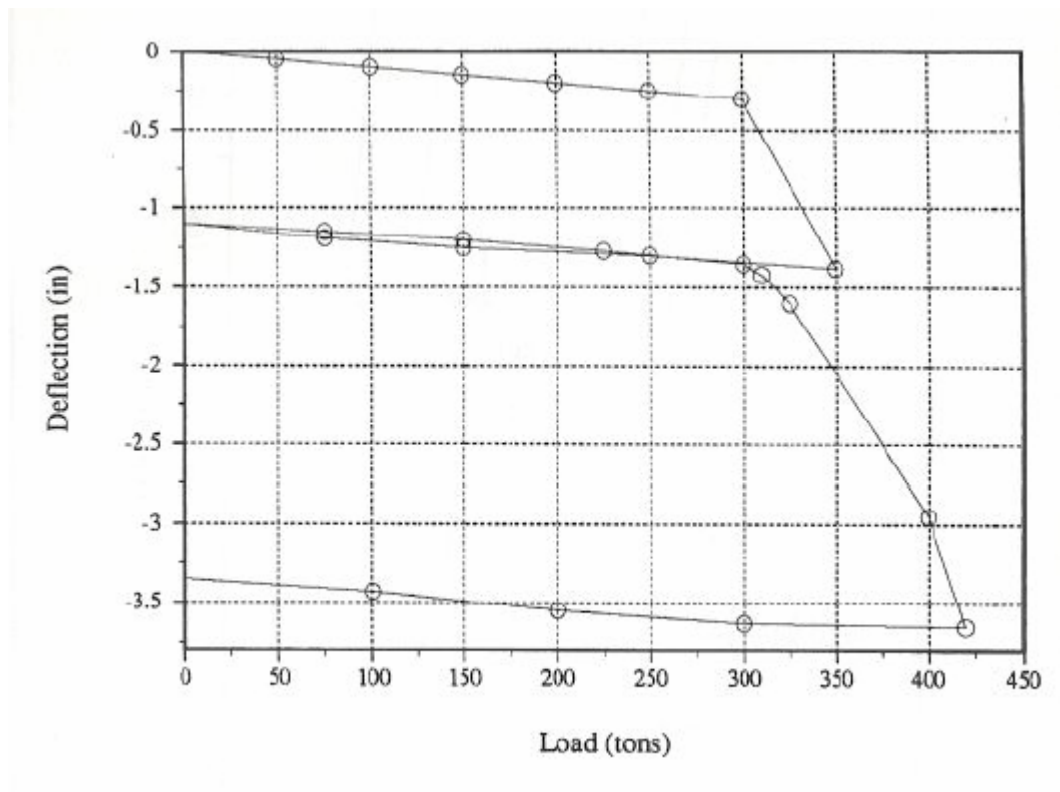


Figure A-26 - Load-deflection curve, Shaft #1, Mt. Pleasant, SC

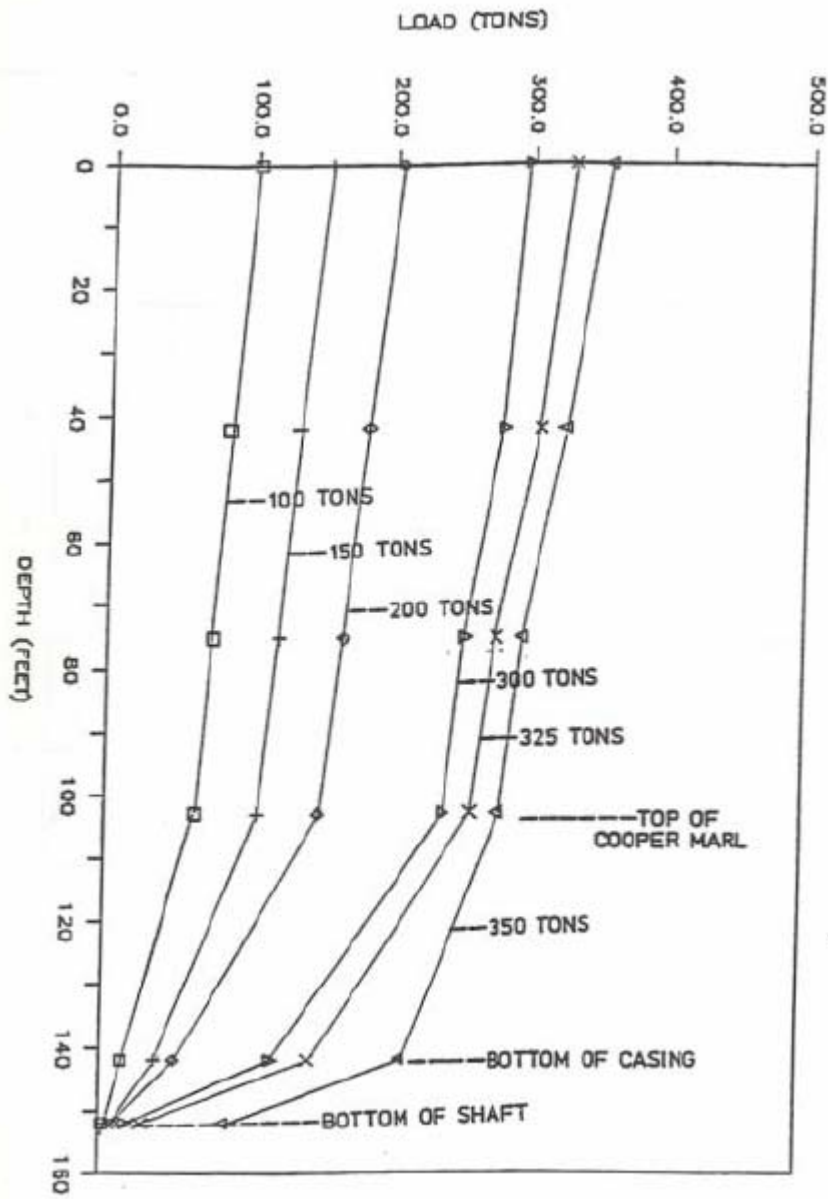


Figure A-27 - Load transfer distribution, Shaft #1, Mt. Pleasant, SC

A dedicated test shaft was constructed with a 23-foot socket into the Cooper Marl formation. A permanent casing was used to isolate the shaft from the overburden above the Cooper Marl. The shaft was constructed in the dry using a temporary casing. There were no unusual problems during the construction of the test shaft.

Shaft Summary

Shaft Diameter (in)	Shaft Length (ft)	Max Net Load (kips)	Max Deflection Down (in)	Unit EB Cap (ksf)
24.0	137.0	1700.00	0.50	20.0

Data Summary

Material	Location	Average Unconfined Compressive Strength, q_u (ksf)	Unit End Bearing (ksf)	Defl. For End Bearing (in)	Unit Side Shear (ksf)	Defl. For Side Shear (in)
Cooper Marl	Socket	2.9			3.60	0.10
Cooper Marl	Base	2.9	18.00	0.24		
Cooper Marl	Base	2.9	20.00	0.50		

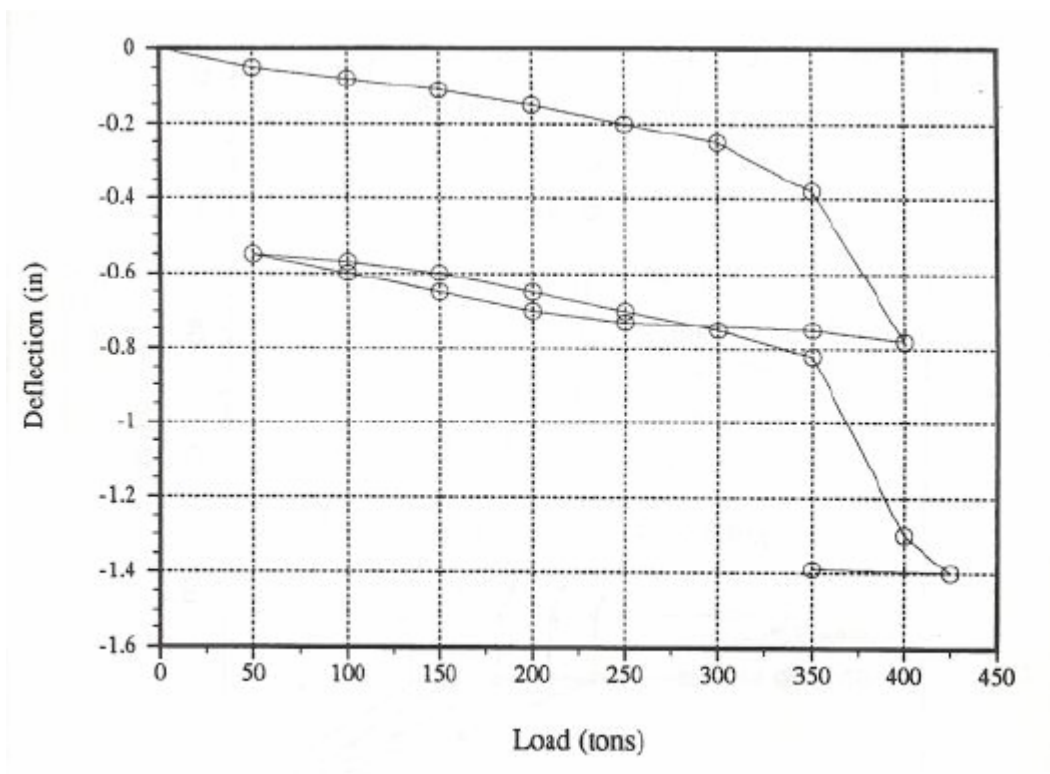


Figure A-30 - Load-deflection curve, Shaft #2, Mt. Pleasant, SC

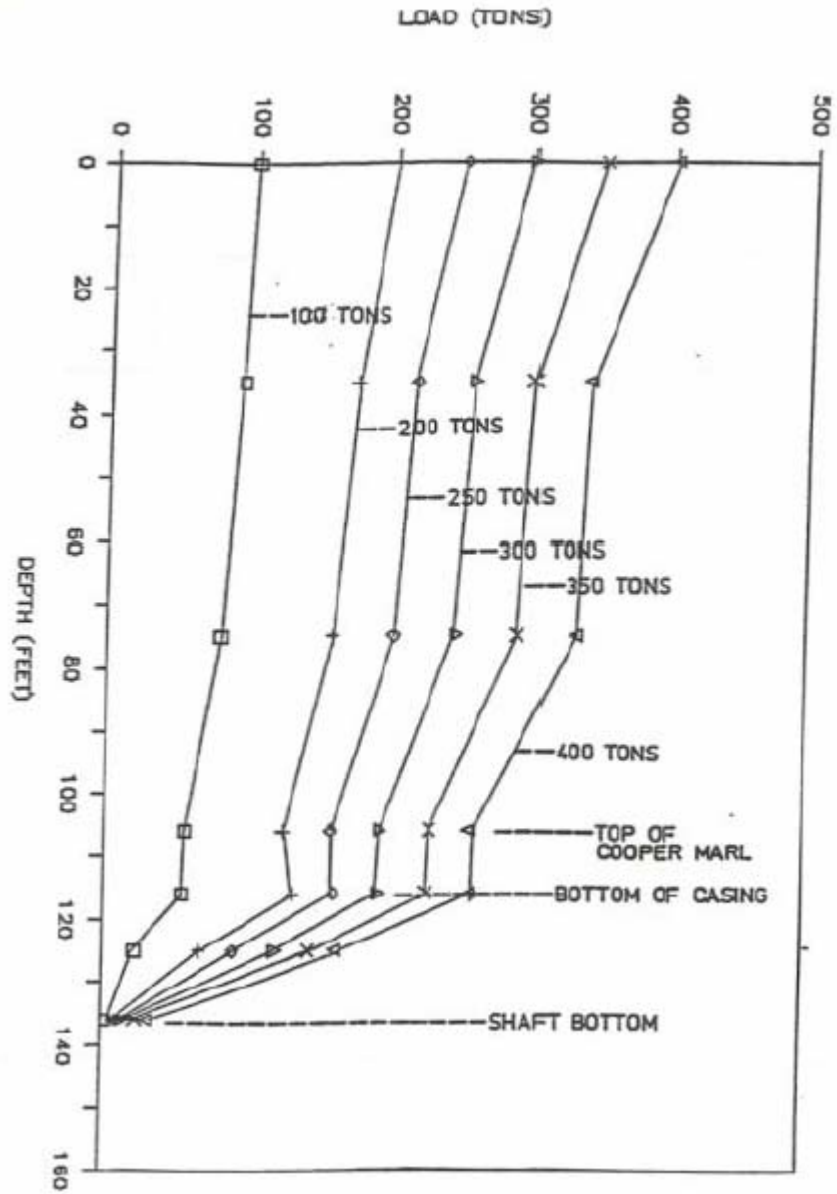


Figure A-31 - Load transfer distribution, Shaft #2, Mt. Pleasant, SC