

Drilled Shaft Performance in Cemented Calcareous Formations In the Southeast US

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ABSTRACT: Cemented calcareous formations, typically referred to as chalk or marl, are widespread in the southeastern U.S. and are ideally suited to the use of drilled shaft foundations. Chalk formations are relatively stable in an open borehole and are relatively easy to drill but difficult to penetrate with driven piling. These cohesive intermediate geomaterials are sometimes difficult to characterize for design purposes because of the difficulty in sampling and testing. This paper provides a review of recent instrumented load tests performed in these cemented calcareous materials. A total of 15 tests were selected from southeastern states based on the availability of good quality load test data and geotechnical information. Most of the test data include measurements of compressive strength from cores. Ultimate unit side shear and end bearing resistance is found to show a relationship with unconfined compressive strength using existing published correlations for weak rock materials.

INTRODUCTION

As a part of ongoing research sponsored by the ADSC Industry Advancement Fund, researchers at several universities are performing an evaluation of design methods using drilled shaft load test data from around the U.S. This article provides a review of data from cemented calcareous formations, typically referred to as chalk or marl. Similar studies are ongoing for other types of rock and intermediate geomaterials.

These chalk materials are typically massive and often have the appearance of hard clay or very soft rock. Microfossils are typically abundant, as evident in the scanning electron micrograph of a marl sample from Charleston, SC shown in Figure 1. Sand may be present in varying amounts, as well as phosphate and clay minerals.

Sampling can be difficult, as the chinks are typically too hard to sample with pushed tube samplers. Rock core sampling techniques can sometimes be used, but cores of softer chinks are easily damaged when using rock coring techniques.

Common practices include standard penetration tests (although SPT N-values of greater than 50 blows per 0.3 m (foot) are common) and pitcher-barrel coring samplers as shown in the photos of Figure 2.

Load tests that were selected for inclusion in this study are based on a review of data collected from Loadtest, Inc., the Alabama Department of Transportation, the Mississippi Department of Transportation, and available literature. A total of 15 tests were selected from southeastern states, based on the availability of good quality load test data and geotechnical information. A case history summary of each load test is included in Brown and Thompson (2008) as are the reference citations of the load tests and geotechnical reports for each test.

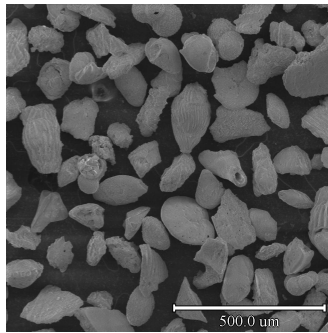


FIG. 1. SEM Photograph of a Cooper Marl Sample (Camp, 2004)



FIG. 2. Pitcher Barrel Sample of Chalk from Alabama

ANALYSIS

Analyses 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 hard cohesive soils and soft rocks.

The collected load test data listed in Table 1 has values of q_u ranging from 138.9 to 6080 kPa (2.9 to 127 ksf). Some sites had SPT data available. Except for the tests in Cooper Marl, SPTs (where performed) were usually recorded as 100+ blows per 0.3 m (foot), or refusal. The sites in the Cooper Marl tended to have blow counts less than 50 blows per 0.3 m (foot). The US 80 project site in Alabama had only SPT data; an estimated $q_u = 2150$ kPa (45 ksf) was used for analyses based on correlations from nearby projects.

Side Shear

Turner (2006) summarized several basic formulas for estimating ultimate unit side shear resistance utilizing 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 shown in (1):

$$\text{Unit Side Resistance:} \quad f_s = C \cdot p_a \cdot \sqrt{\frac{q_u}{p_a}} \quad (1)$$

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 and degree of weathering. An updated evaluation of these test data by Kulhawy et al (2005) suggests that a value of $C = 1$ represents the mean estimate of design ultimate side shear resistance, based on the most up-to-date analysis of the available data.

Table 1. Summary of Load Test Data (Brown and Thompson, 2008) (bpf = b/0.3m; 1 ksf = 47.88 kPa)

State	Project	Test No.	Reference	Test Type	Material	Shaft Dia. (in)	SPT N values (bpf)	q_u (ksf)	Unit Side Shear (ksf)	Unit End Bearing (ksf)
AL	US 80 over Mill Creek	AFT-106058	1	Stamamic	Demopolis/Mooreville Chalk (hard gray, clayey silt)	84	N > 100	---	5.8 - 10.7	41.6
AL	SR 10 Blue Springs Hyundai Motor Manufacturing	LT-8571	2	O-Cell	Claystone	54	N > 100	---	0.9 - 1.7	27.4
AL		LT-8904	2	O-Cell	Demopolis Chalk	42	42 - 62	18.4	3.4 - 9.4	90.9
AL	Andalusia, AL	WRT-1-1	3	Conventional	Claystone	28	---	17 - 127	7.0 - 9.6	---
MS	US 45 over Town Creek	LT-8194	2	O-cell	Mooreville Chalk (Hard gray, clayey silt)	48	---	22.9	5.1	36.9
MS	SR 25 over Talking Warrior Creek	LT-8573	2	O-cell	Basal Formation (Hard Clayey Silt and Silty Clay)	42	---	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)	48	---	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, Silty)	48	---	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	54	---	11.1	1.9 - 5.0	24.8
MS	I-55 at Old Agency Rd.	LT-8788	2	O-cell	Yazoo Formation (Hard, tan, silty clay)	24	---	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)	48	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)	48	N > 100	18.0	8.8 - 12.9	221
MS	Leake County, MS	WRT-4	3	O-cell	Chalk	66		12.1	3.2	46.2
SC	Mt. Pleasant, SC	WRT-5-1	3	Conventional	Cooper Marl	24	9 - 100+	2.9	3.6	28.6
SC	Mt. Pleasant, SC	WRT-5-2	3	Conventional	Cooper Marl	24	9 - 100+	2.9	3.6	---
SC	Cooper River Bridge Charleston/Mt. Pleasant	LT-8650	2,4	O-cell (10 tests)	Cooper Marl (Clayey sand, sandy clay, sandy silt)	72 and 96	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)	48	15 - 28	4.2 - 5.9	0.2 - 2.8	49.4

Reference:

1. AFT-XXXXXX: Statamic 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) *Avial Capacity of Drilled Shafts Socketed into Soft Rock*, M.S. Thesis, Auburn University, AL
4. Values shown are the ranges from the 10 tests. Seven shafts were 96 inches in diameter, three were 72 inches. Shaft LT-8650-1 is used as representative for Figure 4a.

The Horvath & Kenney relationship is compared with measured unit side resistance from the data evaluated in this study, with the data plotted versus q_u on Figure 3. Each test site with q_u data is shown as a small open symbol. The averages of the groups of tests (Mississippi (MS) Chalk, South Carolina (SC) Cooper Marl, and Alabama (AL) Claystone) are shown in large shaded symbols. The US 80 test is plotted using an estimated value for q_u . 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 $C = 0.85$.

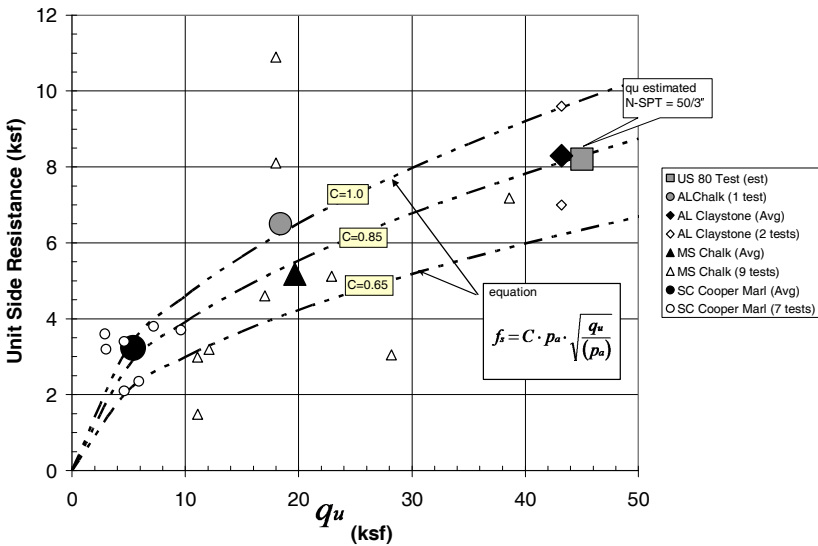


FIG. 3. Unit Side Resistance vs. q_u ($100\text{kPa} \approx 2 \text{ ksf}$)

End Bearing

A wide range of expressions are available for relating base resistance to strength parameters of weak rock or marl. The O’Neil and Reese (1999) guidelines for cohesive intermediate geo materials (IGM), which are massive and relatively free of joints, fissures, or weak seams, would suggest a value similar to (2):

$$\text{Unit Base Resistance: } q_{ult} = 2.5q_u \tag{2}$$

Since the mobilization of unit base resistance occurs over a much larger range of deflections than unit side resistance, the base resistance data are evaluated from the

load tests as a function of displacement in terms of the shaft diameter. Figures 4a and 4b illustrate the mobilized unit base resistance from the load tests 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 .

Two figures were used to plot the data for easier reading. Each test is labeled with the Test No. from Table 1. Three of the tests (LT-8571, WRT-1-1, and WRT-5-2) did not have sufficient end bearing data to plot. The curve labeled LT-8560-1 is plotted as representative of the ten load tests included in Table 1 as LT-8650.

For all but two of the remaining tests, the measured end bearing load-displacement curve was available and is included on the plots. Two tests reached the maximum applied load at a shaft deflection of less than 1%. These are shown as single points on the graphs, and may not have fully mobilized the available base resistance.

The data presented on these figures suggest that the use of the O'Neil and Reese (1999) guideline for cohesive IGM would be conservative in most cases. Some of the data indicate that this guideline could be very conservative; it is possible that the q_u data from some of these sites may have been affected by sample disturbance, or that a higher sand content affects the correlation with a simple q_u measure of strength.

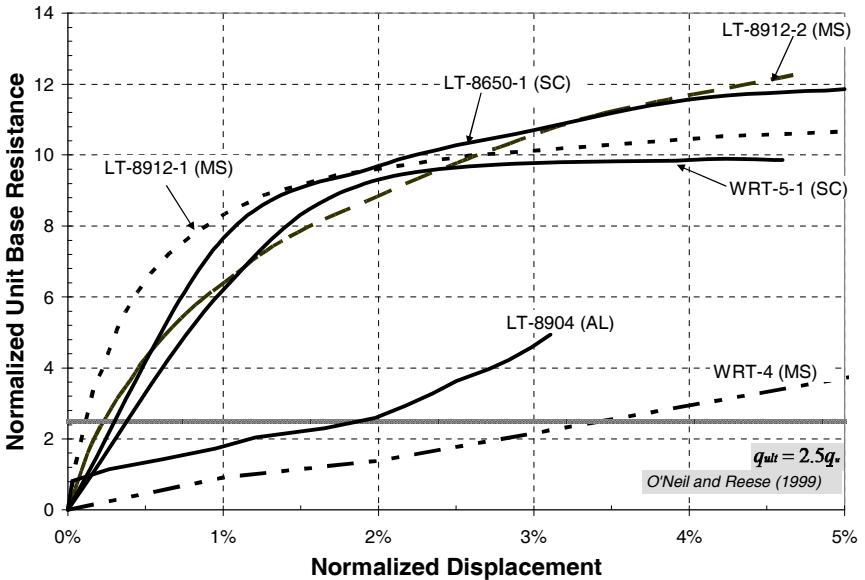


FIG. 4a. Normalized Base Resistance (Resistance/ q_u) vs Normalized Displacement (Shaft Deflection/Shaft Diameter in %)

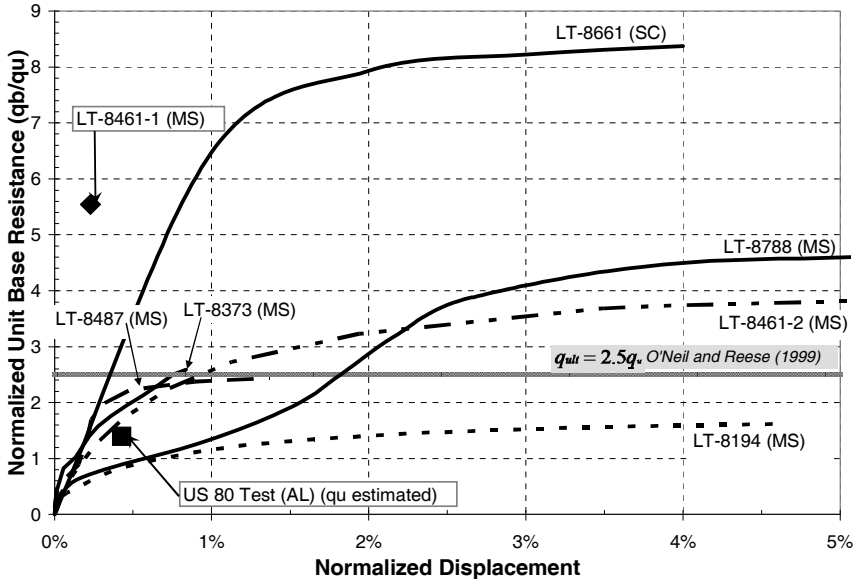


FIG. 4b. Normalized Base Resistance (Resistance/ q_u) vs Normalized Displacement (Shaft Deflection/Shaft Diameter in %)

CONCLUSIONS

The load test data from chalk and marl formations suggest that drilled shafts may conservatively be designed based on unconfined compressive strength data using Eq. 1 with $C=0.65$ for side shear and using Eq. 2 for base resistance. Additional discussion of the design methodology for these geomaterials is included in the final project report and in the forthcoming update to the FHWA Drilled Shaft Manual.

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