Performance of Drilled Shafts in Cemented Calcareous Formations in the Southeast

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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, South Carolina shown in Figure 1. Sand may be present in varying amounts, as well as phosphate and clay minerals.

Sampling can be difficult, as the chalks are typically too hard to sample with pushed tube samplers. Rock core samples can sometimes be used, but cores of softer chalks are easily damaged when using rock coring techniques. Common practices include standard penetration tests (although SPT N-values of greater than 50/ft 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 were based on a review of the data collected. 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 will be available in the project final report.

Analysis

Analyses 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 c_u, is typically used to describe hard cohesive soils and soft (continued on page 44)
analyses based on correlations from nearby projects.

**Side Shear**

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:

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

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 Figure 3. 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 using an estimated value for \( q_u \). The curve representing the equation derived from the Horvath & Kenny relation for ultimate unit side shear resistance is shown, with the empirical constant \( C = 0.65 \). The data presented in this figure suggests that this relationship provides a reasonably conservative estimate of side shearing resistance.

**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:

\[
\text{Unit Base Resistance (ksf) } g_{ub} = 2.5 q_u
\]

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

(continued on page 46)
resistance in terms of the shaft diameter. Figure 4 illustrates 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 $q_u$.

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, and may not have fully mobilized the available base resistance.

The data presented on these figures suggest that the use of the FHWA 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.

Conclusions

The load test data from chalk and marl (continued on page 51)
formations suggest that drilled shafts may conservatively be designed based on unconfined compressive strength data using the expression

$$f_s = C \cdot \sqrt{\frac{q_u}{p_d}}$$

for side shear with $C = 0.65$

and using $q_{ub} = 2.5q_u$ for base resistance.

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References


