Performance of Drilled Shaft Foundations in Limestone, Nashville, Tennessee

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Introduction

The Southeastern Chapter of ADSC has sponsored an ongoing research project to improve design methods and cost-efficiency of rock-supported drilled shafts in the southeastern U.S. In the first phase of this effort, two drilled shaft load tests have been performed at a limestone site near Nashville, Tennessee. The tests were intended to measure the performance of drilled shafts in rock that is representative of the lower bound conditions that might be expected for foundations of this type. The hope is that the test data will provide the basis for improvements in the design methodology used for drilled shafts in Nashville and similar major drilled shaft markets in the southeastern U.S., particularly with respect to foundations on rock.

The testing program is also intended to serve as a test case for further research with rock-bearing drilled shafts in other types of geologic conditions.

This article summarizes the results and potential applications of the Nashville tests. The complete report of this phase of the research, “Load Testing of Drilled Shaft Foundations in Limestone, Nashville, TN” (Brown, 2009), can be downloaded by clicking on the “Nashville Site Report” link at http://danbrownandassociates.com/research-projects. Detailed results of the site investigation, load test data, analyses and interpretation of the load tests, and a discussion of potential improvements to design methodologies are presented in the report.

Geotechnical Conditions

After making borings to evaluate several possible test sites, a site was selected with the desired qualities, i.e., rock that was on the lower bound of typical Nashville conditions. The site was located at the equipment yard of Long Foundations* in Hermitage, Tennessee on the east side of Nashville. According to USGS geologic maps, this site is underlain by Carters Limestone of the Stones River Group, a fine-grained, yellowish-brown limestone with thin beds of bentonite clay. This formation is typical of the Central Basin limestones in the Nashville area. A photo of the rock from the coring tool used to excavate the rock socket of Test Shaft 1 is provided in Figure 1.

Soil borings were drilled in the area of the two test shaft locations as illustrated in Figure 2. The two shafts were approximately 30 feet apart, with the borings approximately 8 ft from the center of the test shafts. The data from the borings suggest that the rock for 3 diameters below the base of Test Shaft 2 (TS2) was somewhat more sound with average percent recovery

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and RQD of 97% and 60% vs average values of 82% and 46% for Test Shaft 1 (TS1). Compressive strength data were quite variable, typically ranging from 5,000 psi to over 20,000 psi. A schematic diagram of the rock strength measurements from the cores are presented in Figure 3.

**Construction of the Test Shafts**

The two Osterberg-cell load tests at the site were designed to attempt to measure both the maximum base resistance and side resistance within the limestone. In order to accomplish both measurements, TS1 was constructed and tested first. The measured resistance from the first test was used to rezone the second test (TS2) and obtain measurements that were not achieved in the first test. The configuration of the two test shafts including the location of strain gauges are illustrated below in Figure 4.

The drilling was performed by Long Foundation Company using a Watson® crawler-mounted drill rig. Both shafts were constructed in the dry by using an auger to excavate the overburden, setting a 54 inch diameter temporary casing into the top of rock, and then excavating the 48 inch diameter socket into the rock using a coring tool. The bottom cleanout of the shaft was made first using a rock auger, with final cleanout using an earth auger. No hand cleaning of the shaft base was performed; although hand cleaning is common in this area, one objective of this study was to evaluate the effect of a less stringent cleanliness requirement.

After completion of the excavation of each test shaft, an inspection probe hole was drilled in the base of each test shaft with an air-operated percussion tool for inspection of the rock below the base of the shaft. Inspections were performed by PSI, Inc. * In addition, at least six other inspectors and engineers from local practicing geotechnical firms and the Tennessee DOT (TDOT) examined the test hole in TS1.

Due to the presence of some soil seams below the tip of the shaft in TS1, the consensus of the inspectors was that under normal circumstances the contractor would have been required to extend this shaft by at least another two feet in order to penetrate below the seams. Because one objective of this study was to evaluate relatively less favorable rock conditions, the shaft was not extended. Several inspectors also noted that the cleaning of the base was not sufficient due to scattered rock debris and cuttings that had not been removed by the auger.

No large seams were observed in the probe hole in TS2 (one small seam less than ½ inch was observed), meaning the shaft excavation would have been accept-

**Load Test Results**

The load tests were conducted by Load-test, Inc.* Figure 5 shows a graph of the average unit side resistance versus displacement for the socket of each shaft data plotted based on the nominal diameter of 48 inches and on a strain gage adjusted for over-break basis on concrete volume. The socket in TS2 was in somewhat better rock in the lower sections of the socket and the construction of the socket resulted in large over-consumption of concrete in the upper portion of the socket due to voids in the limestone. In addition to the uncertainties relating to the actual dimensions of this socket, the limit of the O-cell capacity was reached before the test fully mobilized the side resistance.

The strain gauge data from TS2 suggests that the actual side resistance of the lower portion of the socket was significantly higher than for TS1.

The side resistance data suggest that the side resistance is mobilized at a relatively small displacement of around 0.2 inches or less, and that the maximum average side shear in the socket at TS1 was around 20 ksf.

In order to compare measured base resistance for similar size area, the base resistance is plotted vs displacement normalized by base diameter on Figure 6. These data are based on the projected area from the base plate plus a distribution of 2(vert) 1(horiz) downward through the few inches of concrete below the base plate.

**Preliminary Implications for Design**

**Base Resistance**

Based on the typical local practice, the rock conditions at TS1 would not have been considered acceptable and typical design would have required the shaft excavation to continue to achieve the more favorable conditions typified by TS2. It is appropriate to consider the implications for design of both conditions, hereafter referenced as follows:

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**Construction of the Test Shafts**

The two Osterberg-cell load tests at the site were designed to attempt to measure both the maximum base resistance and side resistance within the limestone. In order to accomplish both measurements, TS1 was constructed and tested first. The measured resistance from the first test was used to resize the second test (TS2) and obtain measurements that were not achieved in the first test. The configuration of the two test shafts including the location of strain gauges are illustrated below in Figure 4.

The drilling was performed by Long Foundation Company using a Watson crawler-mounted drill rig. Both shafts were constructed in the dry by using an auger to excavate the overburden, setting a 4.5 inch diameter temporary casing into the top of rock, and then excavating the 48 inch diameter socket into the rock using a coring tool. The bottom cleanout of the shaft was made first using a rock auger, with final cleanout using an earth auger. No hand cleaning of the shaft base was performed; although hand cleaning is common in this area, there is one objective of this study was to evaluate the effect of less stringent cleanliness requirement.

After completion of the excavation of each test shaft, an inspection probe hole was drilled in the base of each test shaft with an air-operated percussion tool for inspection of the rock below the base of the shaft. Inspections were performed by PSI, Inc.* In addition, at least six other inspectors and engineers from local practicing geotechnical firms and the Tennessee DOT (TDOT) examined the test hole in TS1.

Due to the presence of some soil seams below the tip of the shaft in TS1, the consensus of the inspectors was that under normal circumstances the contractor would have been required to extend this shaft by at least another two feet in order to penetrate below the seams. Because one objective of this study was to evaluate relatively less favorable rock conditions, the shaft was not extended. Several inspectors also noted that the cleaning of the base was not sufficient due to scattered rock debris and cuttings that had not been removed by the auger.

No large seams were observed in the probe hole in TS2 (one small seam less than ½ inch was observed), meaning the shaft excavation would have been acceptable according to local practice. No seepage water was noted in either shaft.

**Load Test Results**

The load tests were conducted by Loadtest, Inc.* Figure 5 is a graph of the average side resistance vs displacement for the socket of TS1, with data plotted based on the nominal shaft diameter of 48 inches and on a shaft diameter adjusted for over-break based on the concrete volume. The socket for TS2 was in somewhat better rock in the lower sections of the socket and the construction of the socket resulted in large over consumption of concrete in the upper portions of the socket due to voids in the limestone. In addition to the uncertainties relating to the actual dimensions of this socket, the likely capacity was reached before the test fully mobilized the side resistance. The strain gauge data from TS2 suggests that the actual side resistance of the lower portion of the socket was significantly higher than for TS1.

The side resistance data suggest that the side resistance is mobilized at a relatively small displacement of around 0.2 inches or less, and that the maximum average side shear in the socket at TS1 was around 20 ksf.

In order to compare measured base resistance for similar size area, the base resistance is plotted vs displacement normalized by base diameter on Figure 6. These data are based on the projected area from the base plate plus a distribution of 2(vert) 1(horiz) downward through the few inches of concrete below the base plate.

**Preliminary Implications for Design**

**Base Resistance**

Based on the typical local practice, the rock conditions at TS1 would not have been considered acceptable and typical design would have required the shaft excavation to continue to achieve the more favorable conditions typified by TS2. It is appropriate to consider the implications for design of both conditions, hereafter referenced as follows:

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of the shaft diameter. At this value, a factor of safety of 2.5 would require that the rock provide an ultimate bearing capacity of at least 500ksf (250 tons per square foot). The geotechnical and inspection requirements listed in the report for “Fair Rock” are less stringent than for “Sound Rock,” particularly in the criteria for seams. As long as solution cavities are not present, the presence of seams deeper than ¼ the base diameter below the bearing elevation and with thickness of up to 10% of the base diameter is considered acceptable.

**Side Resistance**

It may be prudent to consider the addition of side resistance to the end bearing used in the design in “Fair Rock” conditions, particularly where 10 or more feet of socket length is required to achieve the required base resistance consistent with this condition. Note that the conditions through which the two rock sockets were constructed did not consist of rock that would qualify for the “Fair Rock” condition noted for base resistance above. The rock through which the side resistance was measured was characterized by relatively low % recovery and RQD, many seams, and highly variable compressive strengths.

For side resistance, the resistance is mobilized at small displacements and the maximum value used in design is based upon a geotechnical strength condition. For design based on the conditions measured at the Nashville site, the recommended approach is to compute the nominal (limit) side resistance using the equation:

\[ f_s = C \cdot p_{u} \cdot \frac{q_{u}}{\sqrt{p_{u}}} \]  

where \( q_{u} \) is unconfined compressive

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strength, \( p_a \) is atmospheric pressure, and 
\( C \) is an empirical constant taken to be 
equal to 0.4, for limestone similar to the 
Nashville test site.

For service load conditions, the allow-
able side resistance may be computed 
using the nominal side resistance com-
puted above divided by a factor of safety of 
2.5. The allowable unit side resistance 
times the surface area of the rock socket 
may be added to the allowable base resis-
tance to size the shaft for service loads.

It may be noted that the design based on 
allowable side and base resistance values 
will result in a greater proportion of the 
service load supported in side resistance 
because this resistance is mobilized at 
smaller displacements than the base. How-
ever, since the side resistance was observed 
to be ductile up to displacements in excess 
of \( \frac{1}{2} \) inch, the overall factor of safety is not 
affected by issues of strain compatibility.

Cost Implications of Improved 
Design in Rock

Numerous drilled shaft designs have been 
completed over the years using base resis-
tance alone and an allowable (service load) 
design of 100ksf or less. The final report 
includes the consideration of several hypo-
thetical example cases to evaluate the 
potential cost benefit of improved design. 
Unit costs believed to be typical of the 
Southeastern U.S. markets were used to eval-
uate four hypothetical projects for the cost 
implications of the following designs:

A. Previous practice, based on rock exca-
vation to “Strong Rock” and end bearing 
alone at an allowable base resistance of 
100ksf.

B. “Strong Rock” base resistance as out-
lined in the report.

C. “Fair Rock” base resistance as outlined 
in the report.

D. “Fair Rock” base resistance plus side 
resistance as outlined in the report.

In each of the new designs (B, C, D) a 
load test for confirmation of the new de-
sign values was included in the calculated 
costs. Design A (previous practice) does 
not include costs for a load test shaft.

The evaluated examples included a 
large structure with heavy concen-
trated loads, a large structure with 
moderate loads, and a medium 
structure with moderate loads. 
Each example compares the costs 
for each of the four design ap-
proaches. In all three examples, one 
or more of the proposed design ap-
proaches can potentially yield sub-
stantial foundation cost savings 
with the costs of the proposed de-
signs being to that of the cost of pre-
vious practice, even with including 
the cost of a load test. The key to 
achieving savings is the reduced 
rock excavation and increased pro-
ductivity associated with the shorter 
socket lengths resulting from in-
creased design resistance values

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and/or the less stringent acceptance criterion.

Summary and Conclusions

A program of load testing has been performed at a site near Nashville with rock that is representative of the limestone con-

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ditions in this area. The results demonstrate that high end bearing and side resistance is available from drilled shafts constructed in this formation, and higher design values than have historically been used can be readily achieved. Design guidelines are suggested in the report which can accommodate a range of rock conditions and can provide more economical use of drilled shaft foundations. Load testing is recommended on future projects to confirm the successful use of this approach and validate the guidelines. Cost analyses of a range of hypothetical projects suggest that the new guidelines provide an opportunity for substantial cost savings, even with the inclusion of investments in site specific load testing.

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