ADSC Research Project Update: Rock Sockets in the Southeastern U.S.

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ABSTRACT

The bearing capacity of drilled shafts in rock has traditionally been assessed very conservatively. Drilled shafts are often designed based on base resistance only, neglecting side resistance in the rock or in weathered rock. Inspection criteria are typically geared towards finding rock of very high quality in order to allow even modest bearing values as compared to the strength of the rock, even to the point of excavating rock with good-to-excellent side resistance. In order to improve design methods and cost-efficiency of rock-supported drilled shafts in the Southeastern U.S., the Southeast Chapter of ADSC had organized a research project that included full-scale load testing of drilled shafts in various rock formations typical of this region of the country. Two different sites have been tested, with two test shafts at each site. This paper presents a summary of the results of the two test sites, one near Nashville, Tennessee and the other near Lawrenceville, Georgia. The reports for each site present the results of the load tests along with recommendations for improving the efficiency of drilled shafts designed in the subject rock formations, including appropriate use of side resistance and adjustments to typical inspection criteria.

INTRODUCTION

Because of the large load capacity of drilled shaft foundations, it is relatively difficult and expensive to conduct full-scale load tests. As a result, the bearing capacity of drilled shafts in rock has traditionally been assessed very conservatively. In the Southeastern U.S., shafts are traditionally designed based on end bearing alone with an allowable base resistance of around 60 to 100 ksf (2.9 to 4.8 MPa) (occasionally as high as 150 to 200 ksf (7.2 to 9.6 MPa)). Often, the side resistance developed in the weathered rock zone, and even within the rock socket, is ignored or is assumed incompatible with base resistance.

In order to improve design methods and cost-efficiency of rock-supported drilled shafts in the Southeastern U.S., the Southeast Chapter of ADSC organized a research project that included full-scale load testing of drilled shafts in various rock formations typical of this region of the country. The Osterberg Cell (O-Cell) load testing device was used for all load tests. The test data is intended to provide the basis for improvements in the design methodology used for rock-bearing drilled shafts in major drilled shaft markets in the region.

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Through a carefully performed and well-documented load test program with reliable measurements and a very thorough site investigation, the objectives of this project were to determine actual performance characteristics of drilled shafts in conditions typical of the rock formations tested. 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 drilled shaft foundations. The testing program was also intended to serve as a test case for further research with rock-bearing drilled shafts in other types of geologic conditions.

Load tests have been conducted at two sites to date: Nashville, Tennessee and Lawrenceville, Georgia. The Nashville site was selected to provide data in areas with hard limestone bearing strata. Other areas with such conditions include Birmingham and Huntsville, Alabama, and Knoxville and Chattanooga, Tennessee. The Lawrenceville site was selected to provide data in the metamorphic rock formations found in the metro Atlanta area. Other areas with similar metamorphic rocks as the Lawrenceville test site include portions of Georgia, North Carolina, South Carolina, and Virginia. These rock formations are part of the Piedmont geologic province and are often termed “Piedmont rocks.”

The load test reports from both sites (Brown (2009) for Nashville and Thompson et al (2012) for Lawrenceville) include detailed descriptions of the geologic and geotechnical conditions, the test shaft configurations, construction of the shafts, the load test results, and applications of the tests. Appendices to the reports include exploratory borings, rock laboratory tests, and the load test reports. Since this paper is a brief summary of the results from the two test sites, only brief discussions of these items are included. The reader is referred to the respective reports for details on the items covered in this paper.

NASHVILLE SITE

Site Geologic and Geotechnical Conditions

Three candidate sites were investigated in order to locate a test site with rock conditions known to be representative of the least favorable rock that might be considered for drilled shafts with high load capacity in hard limestone geology. The site selected was the least favorable of the three, 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 subsurface exploration of the test site was performed with four borings. The data from the borings suggest that the rock below the base of Test Shaft 2 was somewhat more sound with average %Recovery of 97% and a rock-quality designation (RQD) of 60% versus average values of 82% and 46%, respectively, for Test Shaft 1. Compressive strength data were quite variable, typically ranging from 5,000psi (34.4 MPa) to over 20,000psi (138 MPa).

A photograph of a section of rock core from the zone of the rock socket in Test Shaft 1 is shown in Figure 1. The photo illustrates fairly typical core samples from these limestone formations - high recovery with variable RQD. RQD plotted as a function of depth below the top of the rock sockets is shown in Figure 2.

Figure 1 Rock Core from Nashville Boring B-1, Location of Test Shaft 1

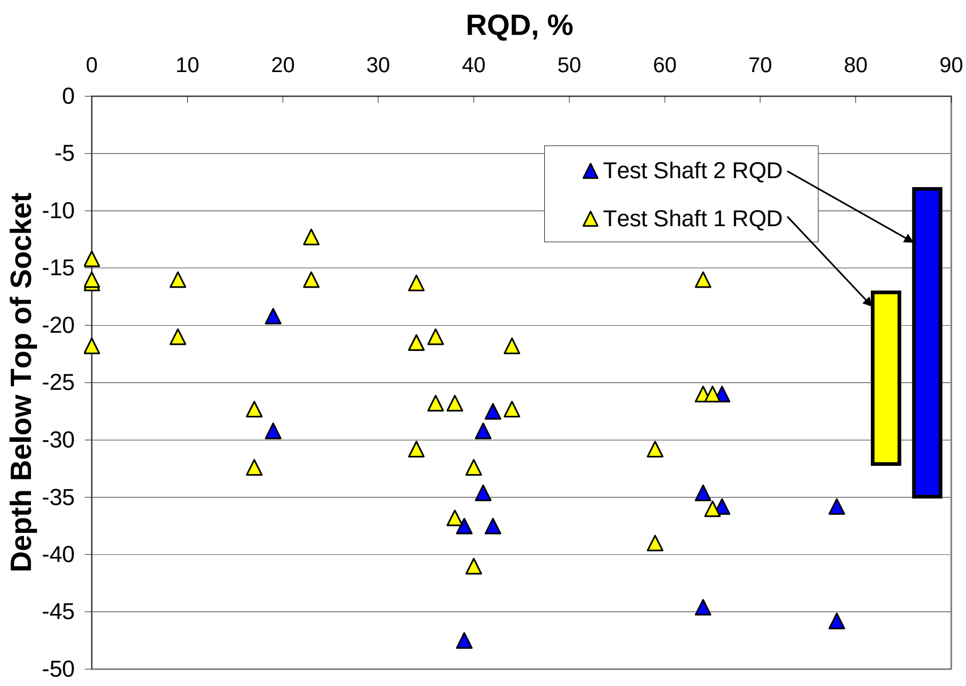

Figure 2 RQD from Rock Cores – Nashville Site

Test Shaft Construction

Two test shafts with O-Cell load test devices were constructed by Long Foundation Drilling Company using a 54-inch (1.37m) diameter earth auger to excavate the overburden and set a temporary casing, and a 48-inch (1.22m) diameter coring tool to excavate the rock. The bottom cleanout of the shaft was performed 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, inspectors and engineers from local practicing geotechnical firms and Tennessee Department of Transportation (TDOT) examined a test hole drilled in the base of the shafts. For Test Shaft 1, 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 (0.6m) in order to penetrate below a 3 to 4 inch (7 to 10cm) soil seam that was observed 19 inches (0.5m) below the base of the excavation. Because one objective of this study was to evaluate relatively less favorable rock conditions, the shaft was not extended. Test Shaft 2 was deemed acceptable with only a small (less than ½-inch (13mm)) seam noted within the probe hole. Several inspectors noted that the cleaning of the base of both shafts was not sufficient due to scattered rock debris and cuttings that had not been removed by the auger.

During concrete placement in Test Shaft 2, there was a significant over-consumption of concrete lost into a void somewhere below the bottom of the casing. A review of the three borings around Test Shaft 2 noted top of rock (auger refusal) at depths ranging from 19.2 feet (5.9m) to 27.5 feet (8.4m), whereas the temporary casing was installed to a depth of 20 feet (6.1m). The top of sound rock was therefore quite irregular in this location, and it is likely that the concrete blew out into a void near the top of rock or bottom of casing once a few feet of head inside the casing produced sufficient pressure.

LOAD TEST RESULTS

Unit Side Resistance

The overall side resistance in the rock was most reliably determined from the results of Test Shaft 1 because this test was conducted to fully mobilize the shaft resistance and was not complicated by the large concrete over-run below the casing as occurred on Test Shaft 2.

As shown in Figure 3, the unit side resistance is plotted as a function of normalized upward O-Cell displacement expressed as a percent of the shaft diameter. On this figure, the data is plotted based on the nominal shaft diameter of 48 inches (1.22m) and on a shaft diameter adjusted for over-break based on the concrete volume. Normalizing the displacement allows for a better direct comparison of different diameter shafts.

The data of the side resistance suggests that the side resistance is mobilized at a relatively small displacement of approximately 1% of the shaft diameter (around 0.2 inches (5mm) or less), and that the maximum average side shear in the socket at Test Shaft 1 was around 20 ksf (138 MPa).

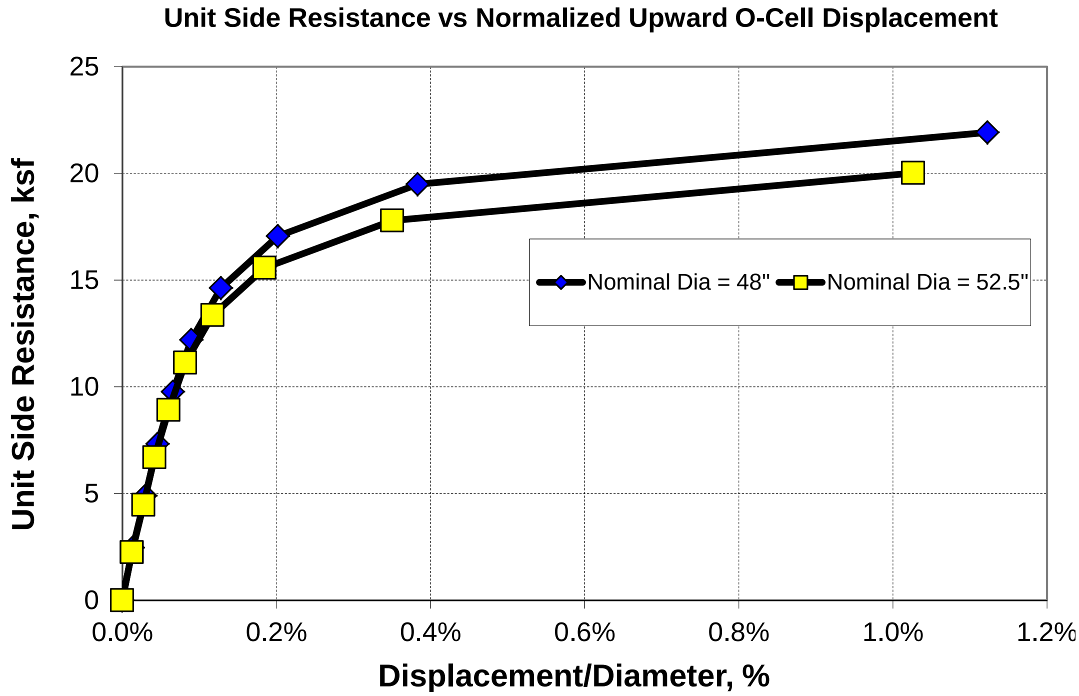

Figure 3 Average Unit Side Resistance vs. Normalized Upward O-Cell Displacement (Nashville Site)

Unit Base Resistance

The measured unit base resistance from the two test shafts is shown on Figure 4. The base resistance of the two shafts was mobilized up to relatively small displacements of just over 1% of the diameter of the loaded area, but did not mobilize the geotechnical limit of the formation in terms of bearing capacity. Test Shaft 1 had a unit base resistance of 500 ksf (23.9 MPa) while Test Shaft 2 had a unit base resistance of 1250 ksf (59.9 MPa).

For evaluation of the load versus deformation characteristics of the base resistance, it is appropriate to consider the settlement, ρ_s of a rigid circular loaded area bearing on the surface of an elastic half space, which may be expressed as follows:

$$\rho_s = 0.79 \cdot \frac{qB\left(1-\nu^2\right)}{E} \qquad \text{(Eq. 1)}$$

where q is the bearing pressure, ν is Poisson's ratio, and E is the elastic modulus of the rock mass. For a typical value of Poisson's ratio of 0.25, and values of ρ_s equal to 0.5%

to 1% of the diameter of the base, the effective elastic modulus of the rock mass can be back-calculated from the load test data to derive the values shown on Figure 4.

The lower back-calculated value for Test Shaft 1 is undoubtedly related to the soil seam present at a depth of about 19 to 23 inches (48 to 58cm) below the base of the shaft. Normalized by the base diameter, B, this represents a soil seam of thickness equal to 10% of the base diameter and located at a distance of approximately ½ B below the base of the shaft.

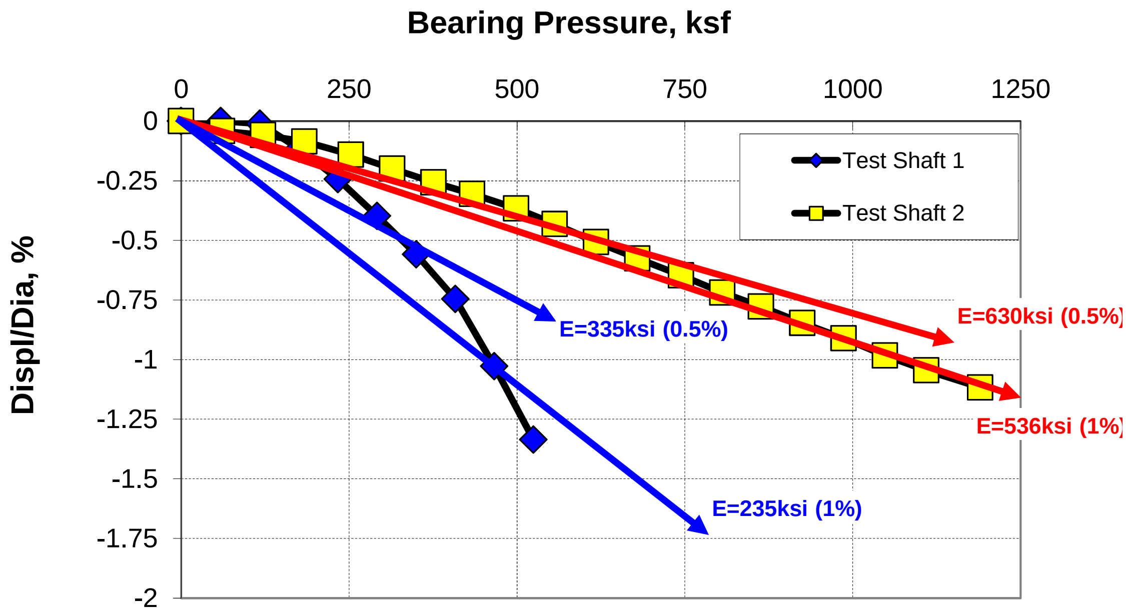

Figure 4 Unit Base Resistance vs. Normalized Downward Displacement of O-Cell (Nashville Site)

Implications for Design

Based on typical local practice, the rock conditions at Test Shaft 1 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 Test Shaft 2. Implications for design were considered and discussed in Brown (2009), along with proposed design recommendations and site-specific criteria for implementing the recommendations. Two conditions were designated based on the results of the two tests:

- "Sound Rock" typified by the conditions similar to Test Shaft 2 with only one or two small seams less than ½ inch (13mm) thick
- "Fair Rock" typified by the conditions similar to Test Shaft 1 with soil-filled seams up to 10% of the base diameter, B, at depths greater than ½ B.

Sound Rock

The load test of Test Shaft 2 in the "Sound Rock" conditions was observed to mobilize a bearing pressure of 1,250 ksf (59.9 MPa) at a displacement slightly over 1% of the shaft diameter without any signs of bearing failure in the rock. This bearing pressure is approximately 8,600 psi (59.3 MPa), and exceeds the compressive stress that likely could be placed on the column from a structural strength limit standpoint. Utilizing a factor of safety of 2.5, an allowable bearing pressure of 500 ksf (23.9 MPa) could be utilized if certain geotechnical and inspection requirements were met that characterize "Sound Rock," particularly minimal-to-no soil seams present and the lack of solutions cavities. Such a bearing pressure in "Sound Rock" conditions would generate a displacement equal to 0.5% of the shaft diameter, equating to settlements of ¼" to 3/8" (6 to 10mm) for the serviceability condition of a typical 4 to 6 foot (1.2 to 1.8m) diameter shaft.

Fair Rock

The load test of Test Shaft 1 in the "Fair Rock" conditions was observed to mobilize a bearing pressure of 500 ksf (23.9 MPa) at a displacement slightly over 1% of the shaft diameter without any signs of bearing failure in the rock. Utilizing a factor of safety of 2.5, an allowable bearing pressure of 200 ksf (9.6 MPa) could be utilized if certain geotechnical and inspection requirements were met that characterize "Fair Rock." Such a bearing pressure in "Fair Rock" conditions would generate a displacement equal to 0.5% of the shaft diameter, equating to settlements of ¼" to 3/8" (6 to 10mm) for the serviceability condition of a typical 4 to 6 foot (1.2 to 1.8m) diameter shaft.

Side Resistance

The design of drilled shaft foundations in the Nashville area using an allowable bearing pressure of up to 500 ksf (23.9 MPa) and the criteria outlined in Brown (2009) would represent a significant increase in bearing pressures over historical practice in the area. Such high bearing pressures may be constrained by structural strength limitations at service loads such that higher than normal concrete strength may be required to fully utilize these bearing pressures. There would be little reason to include side resistance in the design of a rock-bearing shaft on "Sound Rock," since little additional resistance would likely be realized and structural considerations will likely govern.

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 feet (3m) or more of socket length is required to achieve the required base resistance consistent with this condition.

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 Equation 2:

$$f_s = C \cdot p_a \cdot \sqrt{\frac{q_u}{(p_a)}} \qquad \text{(Eq. 2)}$$

where q_u is unconfined compressive 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. This equation was originally proposed by Rowe and Armitage (1987), and subsequently modified by Kulhaway and Phoon (1993).

For service load conditions, the allowable side resistance may be computed using the nominal side resistance computed 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 resistance 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. However, since the side resistance was observed to be ductile up to displacements in excess of ½ inch (13mm), the overall factor of safety will not be significantly affected by issues of strain compatibility.

Application to Practice

Numerous drilled shaft designs have been completed in the Nashville area over the years using base resistance alone and an allowable (service load) design of 100 ksf or less. The test results indicate that bearing resistance of at least twice what has been the typical value is readily available, provided the geotechnical criteria are met. This means that significant savings can be achieved just through adopting higher allowable design bearing pressures. Brown (2009) detailed three examples of significant costs savings based on application of the recommendations derived from the Nashville tests, including designs based on application of base resistance and side resistance together.

LAWRENCEVILLE SITE

Site Geologic and Geotechnical Conditions

The site selected for this test was located on the property of Foundation Technologies, Inc. in Lawrenceville, Georgia. According to USGS geologic maps, this site is located in the Piedmont Geologic Province and is underlain by the Wolf Creek Formation of the Atlanta Group. This formation predominately consists of amphibolites and biotite-muscovite schist. Gneiss is also mapped in the general area of the site, and was the rock that was encountered at the test site.

The soils in this area were formed by the in-place weathering of the underlying crystalline rock, resulting in the term “residual soils” as a common descriptor. As depth increases, the soil becomes less weathered, coarser grained, and the structure of the underlying rock becomes more evident. When the standard penetration resistance is 100 blows per foot (100 blows per 0.3m) or greater, these materials are locally referred to as partially weathered rock (PWR). PWR represents the zone of transition between the soil and the underlying parent metamorphic rocks. The transition, from soil to PWR, is usually gradual, and may occur over a wide range of depths over a relatively small horizontal distance. This gradual transition is also true regarding the thickness of the PWR zone and the depth to the rock surface.

A subsurface exploration of the test site was performed with seven borings that indicated that the conditions were variable, as is typical of this geologic setting. The depth to the top of the PWR varied within short horizontal distances. The rock cored from the borings was classified as hornblende gneiss.

The data from the borings suggests that the characteristics of both the PWR and the rock were variable. The rock within and below the rock socket of Test Shaft 1 had %Recovery generally ranging from 80% to 100%, and a RQD ranging from 0% to 60%. The PWR at and below Test Shaft 2 had highly variable %Recovery ranging from 30% to 80%, and a RQD ranging from 0% to 40%. Four uniaxial compressive strength tests were performed with results ranging from 7,000 psi (48.3MPa) to 11,300 psi (77.9 MPa).

Photographs of the rock core from the zone of the rock socket in Test Shaft 1 are shown in Figure 5. The photos illustrate fairly typical core samples from the Piedmont – relatively high recovery with low RQD.

Figure 5 Rock Core from Lawrenceville Boring B-1, Location of Test Shaft 1

Much of the rock was recovered, but the weathering of the rock results in highly segmented core samples resembling a stack of hockey pucks (or perhaps we should say biscuits, as this is Atlanta). As shown in Figure 6, the RQD is plotted as a function of depth below the top of the rock socket for Test Shaft 1 and the top of the PWR for Test Shaft 2.

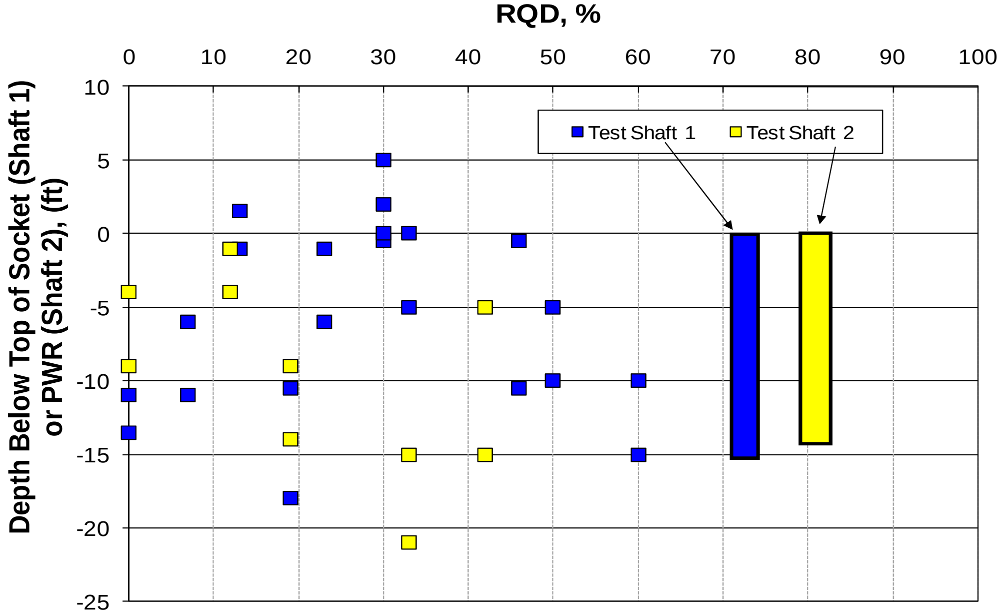

Figure 6 %RQD from Rock Cores – Lawrenceville Site

Test Shaft Construction

Two test shafts with O-cell load test devices were constructed by McKinney Drilling. Test Shaft 1 was designed to measure both the maximum base resistance and side resistance within the schist/gneiss bedrock using a 40.5-inch (1.03m) diameter rock socket. Test Shaft 2 was designed to measure side resistance in PWR and base resistance, at what is usually termed “rock auger refusal,” with a 66-inch (1.7m) diameter shaft. The term “rock auger refusal” is a common term in project specifications to delineate when payment for rock socket excavation begins, and is typically defined in terms of a penetration rate with a common drilling rig, in this case 2 inches (5cm) penetration in 5 minutes with an LLDH rig.

Once rock was encountered in Test Shaft 1, a core barrel was used to start the rock socket. Since the core barrel advanced rather rapidly (3ft (0.9m) in 30 minutes), a rock auger was used to complete the socket to a total depth of 53 feet (16.2m) with three observations of penetration rate made during drilling (locally termed “penetration test”). After penetration rates of 4 to 6.5 inches/5 min. (10 to 17cm/5 min.) were recorded between depths of 49 and 53 feet (14.9 to 16.2m), the research team decided to stop the shaft at a depth of 53 feet (16.2m) rather than continue to chase after rock that met the typical rock auger refusal criteria. The penetration rates in the bottom of this excavation indicated that the material considered “rock” at this site was soft enough to be augered at acceptable production rates.

The goal for Test Shaft 2 was to stop the shaft at “rock auger refusal.” At a depth of 41 feet (12.5m), penetration rates of 3in/5min. (7.5cm/5 min.) were recorded. This rate was similar rate to that within the rock socket in Test Shaft 1, though with a larger tool. The shaft excavation was then terminated since the goal was to have the shaft bear on rock, but not have a rock socket.

At the completion of each shaft, the excavations were hand cleaned and an inspection probe hole was drilled into the rock in the bottom of the shaft. Inspectors from several local practicing geotechnical firms and Georgia Department of Transportation (GADOT) inspected the base of each shaft to assess the quality of the rock below the base of the drilled shafts. The consensus of the inspectors was that the hole was acceptable with only one or two small seams noted.

LOAD TEST RESULTS

Unit Side Resistance

When the shafts were planned, based on the boring log data (e.g., auger refusal, core recovery and RQD, compressive strength tests, etc.), it was believed that the PWR zones in both shafts would yield similar unit side resistance values. Evaluation of the excavation observations indicated that Test Shaft 2 had two zones of PWR. The upper zone was softer than the lower zone, which was similar to the PWR encountered in Test Shaft 1.

The unit side resistance as a function of normalized upward O-Cell displacement expressed as a percent of the shaft diameter is shown in Figure 7. The data of the side resistance suggests that the limit of the O-cell was reached before side resistance was fully mobilized, though the curves in Figure 7 indicate that the tests were close to reaching the maximum. Significant resistance was mobilized in the gneiss rock at a relatively small displacement of around 0.2-inches (5mm) or less. The maximum average side resistance values from the tests is listed in Table 1. The maximum value for the Lower PWR in Test Shaft 2 is not listed since it is interpreted to be the value from Test Shaft 1.

Table 1 Maximum Average Unit Side Resistance

Shaft	*Material*	*Maximum Unit Side Resistance (ksf)*	*Normalized Displacement (% Diameter)*	*Nominal Displacement (inches)*
1	Gneiss Rock	52	1.5	0.6
1	PWR	15	1.5	0.6
2	Soft PWR (Upper PWR)	2.5	0.5	0.3

<u>Note:</u> The maximum value for the Lower PWR in Test Shaft 2 is not listed since it is interpreted to be the value from Test Shaft 1.

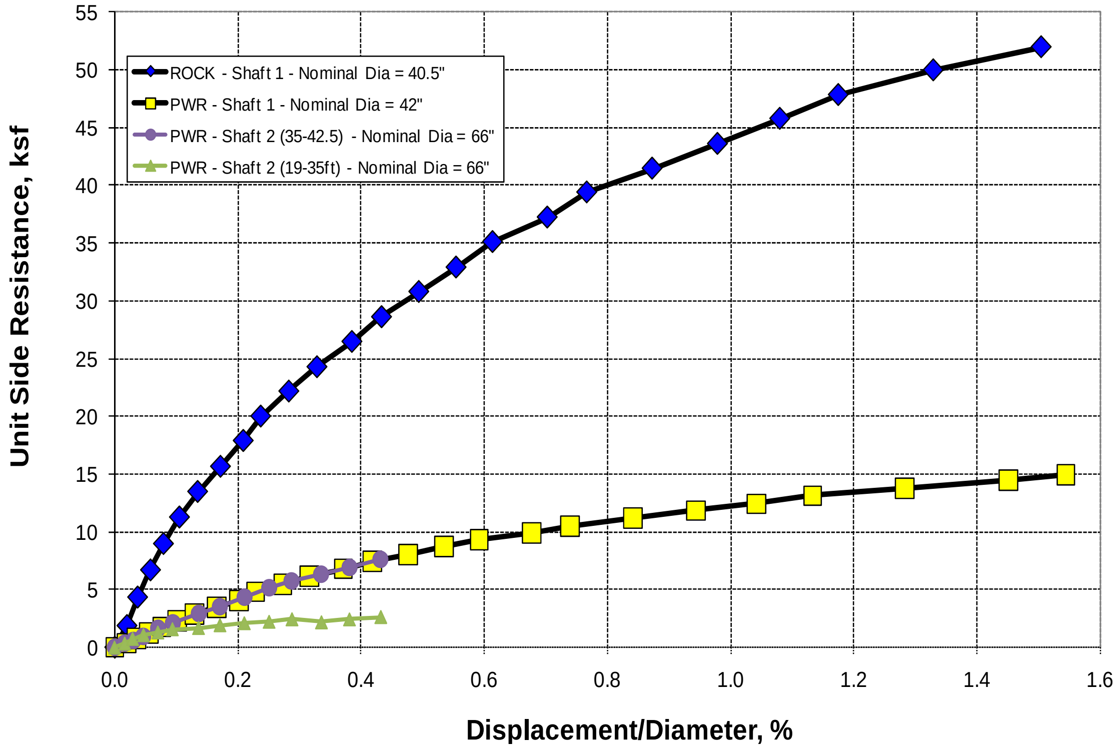

Figure 7 Average Unit Side Resistance vs. Normalized Upward O-Cell Displacement (Lawrenceville Site)

Unit Base Resistance

The measured unit base resistance from the two test shafts is provided in Figure 8, with the unit base resistance plotted versus normalized downward displacement. The maximum values of unit base resistance are listed in Table 2.

The base resistance of the two shafts was mobilized up to relative displacements of 4% (Test Shaft 1) and 8% (Test Shaft 2) of the diameter of the loaded area, but did not mobilize the geotechnical limit of the formation in terms of bearing capacity. Similar to the data presentation for the Nashville tests, back-calculated values of the elastic modulus are shown on Figure 8 for these tests.

The lower back-calculated value of the modulus for Test Shaft 2 suggests that the base of the shaft was in softer rock than Test Shaft 1. The inspectors' reports for this shaft were almost identical to those for Test Shaft 1, and the penetration rate near the end of excavation was similar to that of Test Shaft 1. Test Shaft 2, however, was made with a 66-inch (1.68m) diameter tool rather than a 40.5-inch (1.03m) diameter tool – a tool that covers more than twice the area. It is possible that a softer zone similar to that encountered at depths of 31 to 35 feet (9.5 to 10.7m) was present immediately below the bearing elevation. Regardless, the results from Test Shaft 2 illustrate the variability of the PWR and rock in this geologic setting.

Table 2 Maximum Unit Base Resistance

Shaft	*Material*	*Maximum Unit Base Resistance (ksf)*	*Normalized Displacement (% Diameter)*	*Nominal Displacement (inches)*
1	Gneiss Rock	850	4.0	1.7
2	Gneiss Rock	600	8.3	2.0

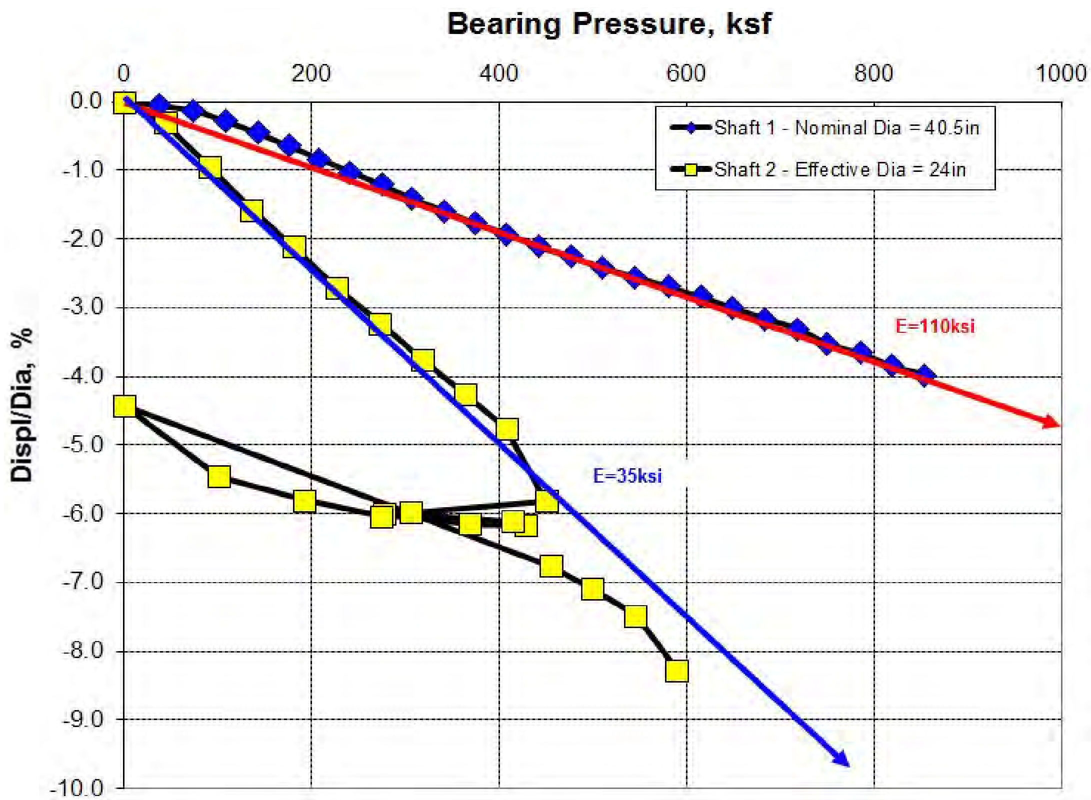

Figure 8 Unit Base Resistance vs. Normalized Downward Displacement of O-Cell (Lawrenceville Site)

Implications for Design

Thompson et al (2012) discusses several implications for the design of drilled shaft foundations in the Piedmont geologic setting based on the test results. Three of these implications to note are:

1. The test data suggests that the current rock auger refusal criteria of 2inch/5min. (5cm/5min.) may be too restrictive. A less restrictive criterion of 5inch/5min. (13cm/5min.) appears appropriate from the actual penetration rates and the test results. Both test shafts were terminated in material that did not meet the current criteria for rock auger refusal. The penetration rates (4 to 6 inch/5min. (10 to 15cm/5min.)) for Test Shaft 1 and 3inch/5min. (7.5cm/5min.) for Test Shaft 2) were faster than the typical 2in/5min. (5cm/5min.) for this area. Yet, significantly

higher unit base and side resistance values were achieved than are commonly used for design at these conditions.

2. These two test shafts were terminated in rock that did not meet the current criteria for rock auger refusal, yet the ultimate or strength limit state base resistance, indicated by the tests, exceeds the structural capacity of typical reinforced concrete shaft. This illustrates the fact that the geotechnical design parameters of the rock are not a limitation for design in this geology.

3. The unit side resistance was over 50 ksf (2.4 MPa) in rock with rock auger penetration rates of 4 to 6.5in/5min. (10 to 15cm/5min.), faster than the current definition of rock auger refusal for material to be considered hard rock. The results indicate that such high nominal or ultimate values can be achieved. In order to develop such values of side resistance, it is important that construction utilizing casing be performed in a way that does not contaminate the bond at the concrete/rock interface.

Application to Practice

In order to develop a consensus on applying the results of the Lawrenceville load tests to local practice, a committee was formed consisting of Atlanta area practitioners, members of the ADSC Southeast Chapter, and the researchers. The committee reviewed the Lawrenceville site test results, reviewed the current local practice with respect to drilled shaft design, and identified key considerations for applying the test results within local practice in a meaningful way.

The group then discussed and developed recommended design values for base resistance and side resistance developed from the load test results. Criteria were also developed that must be met to reliably apply these design recommendations for a particular site. The criteria and detailed discussions of their application are contained in Thompson et al (2012).

The site-specific criteria are based on information that should be obtained from an appropriate site assessment by the geotechnical engineer. As with any design recommendations or process, the application of the experience and engineering judgment of the geotechnical engineer is required.

The working committee did not envision that the criteria would be used as an “either or” decision tool (e.g., that a site would have rock that meets the criteria and use the recommended design values, or the site did not meet the criteria and the design would revert to current more conservative practice). Instead, the group envisioned various typical design/construction scenarios utilizing the criteria for drilled shaft design and construction. Such scenarios ranged from base resistance only designs (as is currently typical) utilizing the higher resistance values in the criteria, to designs utilizing both side and base resistance though a rational application of the criteria utilizing the experience and engineering judgment of the geotechnical engineer.

It should also be recognized that the criteria are not meant to be a ceiling for drilled shaft design values in this geologic setting. These tests achieved very high unit bearing resistance and showed no indications that bearing failure was imminent. The recommended design values should be seen as what can routinely be used for materials that meet the specified criteria. With the application of engineering judgment and experience, engineers should be able to use even higher values at sites that meet or exceed the PWR/rock criteria established in the report.

CONCLUSION

This paper has summarized the results of a program of load testing performed at two sites as part of a research project aimed at improving design methods and cost-efficiency of rock-supported drilled shafts in the Southeastern U.S. The results of the two sites tested to date demonstrate that high end bearing and side resistance is available from drilled shafts constructed in the tested geologic formations, and that higher design values than have historically been used can be readily achieved. Design guidelines are suggested which can accommodate a range of rock conditions, and can provide more economical use of drilled shaft foundations in the two markets. Site-specific criteria are provided for determining if a site has rock of sufficient quality to provide the design values of base and side resistance. The criteria require a thorough site investigation to characterize the rock, and require an inspection program during shaft excavation to confirm the findings of the site investigation.

REFERENCES

Brown, D.A. (2009). “Load Testing of Drilled Shaft Foundations in Limestone, Nashville, TN”, Report for ADSC Southeast Chapter, February, 2009.

Kulhawy, F.H. and K.K. Phoon, (1993). “Drilled Shaft Side Resistance in Clay Soil to Rock,” *GSP38*, ASCE pp. 172-183.

Rowe, R.K. and H.H. Armitage, (1987). “A Design Method for Drilled Piers in Soft Rock,” *Canadian Geotechnical Journal*, Vol. 24, pp. 126-142

Thompson, W.R., Brown, D.A., and Hudson, A.B. (2012). “Load Testing of Drilled Shaft Foundations in Piedmont Rock, Lawrenceville, GA, Report for ADSC Southeast Chapter, January, 2012.