ADSC Research Supports Improved Design of Drilled Shaft Foundations in Atlanta Area Piedmont Rock

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Introduction

After the successful work in the Nashville Limestone (reported in Foundation Drilling, May 2009), the ADSC Southeast Chapter focused on the Atlanta area for part II of the ongoing research project to improve design methods and cost-efficiency of rock-supported drilled shafts. As a part of this research, two drilled shaft load tests were performed in hornblende gneiss at a site in Lawrenceville, Georgia. Following the completion of the field testing work, a task force working group of local practicing engineers and engineering faculty joined the research team to develop a consensus on criteria to be implemented for improved design in the local geology. This article is based on a report by the authors “Load Testing of Drilled Shaft Foundation in Piedmont Rock, Lawrenceville, Georgia”, January, 2011.

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 metro Atlanta area and other similar areas, rock-bearing shafts are traditionally designed on the basis of end bearing alone with an allowable base resistance of around 60 to 100 ksf (occasionally as high as 150 to 200 ksf) on rock that is verified by probe holes to be free of soil seams within two diameters below the base. Often, the side resistance developed in the weathered rock zone (locally termed Partially Weathered Rock or PWR in this area of the Piedmont) is ignored or is assumed to be incompatible with base resistance.

The test data are intended to provide the basis for improvements in the design methodology used for drilled shafts in metro Atlanta and similar major drilled shaft markets in the southeastern U.S., particularly with respect to foundations on rock.

Site Geology and Geotechnical Conditions

The site selected for this test was located on the property of ADSC Associate Member firm, Foundation Technologies, Inc. in Lawrenceville, Georgia in the northeast part of the metro Atlanta area. The test site was selected to provide the opportunity to evaluate both the PWR and the underlying rock typical of similar Piedmont rock formations that might be considered for drilled shafts with high load capacity.

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 predominantly consists of amphibolites and biotite-muscovite schist with some gneiss also present.

The soils in this area of the country 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 these materials have a standard penetration test (SPT) resistance of 100 blows per foot or greater, they 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 often occur at a wide range of depths over a relatively small horizontal distance, as the weathering and decomposition preferentially follows fractures, seams, or other discontinuities in the parent bedrock. The thickness of the PWR zone and the depth to the rock surface can also be quite variable, as the folding and distortion of the bedrock surface associated with the metamorphic geology can result in changes over relatively short distances.

Although the rock is known to be quite strong and very suitable for high capacity drilled shaft foundations, one of the concerns that foundation engineers associate with this geology is the need to verify that the characteristics of the rock at each individual drilled shaft location is consistent with the design assumptions. The ideal “perfect” intact rock with no seams or fractures near the base of the foundation is often elusive, and unreasonable expectations or requirements can sometimes result in significant rock excavation costs or delays.

Geo-Hydro Engineers Inc., an ADSC Technical Affiliate Member, performed a subsurface exploration of the test site 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 suggest 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 80% to 100% and Rock Quality Designation (RQD) from 0% to 60%. The PWR at and below Test Shaft 2 had highly variable % recovery from 30% to 80% and RQD from 0% to 40%. Four uniaxial compressive strength tests were performed with results ranging from 7,000 psi to 11,300 psi. Point load tests were performed on

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rock specimens that were too short for conventional compression tests, and compressive strengths correlated from point load test results ranged from 3,100 to 16,000 psi.

Photographs of the rock core from the zone of the rock socket in Test Shaft 1 are shown in Figure 1. The photo illustrates fairly typical core samples from the Piedmont — relatively high recovery with low RQD. 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). Figure 2 shows the RQD 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. Additional plots of the rock core data are contained in the report.

Test Shaft Construction

Two test shafts with Osterberg-cell load test devices were constructed by ADSC Contractor Member firm, McKinney Drilling. Test Shaft 1 was designed to measure both the maximum base resistance and side resistance within the schist/gneiss bedrock. Test Shaft 2 was designed to measure side resistance in PWR and base resistance at what is usually termed “rock auger refusal.” The term “rock auger refusal” is a common term in project specifications to delineate when payment for rock socket excavation begins and, in several states in the southeastern U.S., is typically defined in terms of a penetration rate with a common drilling rig, e.g., two inches penetration in five minutes with an LLDH rig.*

Test Shaft 1 – 42in Rock Socket

Test Shaft 1 was drilled with an earth auger with side reamers (total diameter 48in) to a depth of 29 ft below ground surface where earth auger refusal was encountered – defined as the top of PWR. A 48in diameter temporary casing was inserted into the excavation and drilling was continued with a 40/5in rock auger through the PWR to the anticipated top of rock at a depth of 38ft. At this time, a 40/5in core barrel was used to start the rock socket, coring to a depth of 44ft where the first core was then extracted. Some of the recovered core is shown in Figure 3. Since the core barrel advanced rather rapidly (3ft in 30 minutes), the 40/5in rock auger (without side reamers) was used to complete the socket.

The excavation was advanced to a total depth of 55ft using the rock auger, with three observations of penetration rate made during drilling (locally termed “penetration test”). After penetration rates of 4 to 6.5in/5min were recorded between depths of 49 and 53ft, the research team decided to stop the shaft at a depth of 53ft, rather than continue to chase after rock that met the rock auger refusal criteria. The penetration rates in

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the bottom of this excavation indicated that the material considered “rock” at this site was soft enough to be augerable at acceptable production rates.

Test Shaft 2 – 66in Shaft to Rock Auger Refusal

Test Shaft 2 was started by drilling to a depth of 18ft with a 72in diameter earth auger. A segment of 72in diameter temporary casing was twisted into place with the tip of the casing at a depth of 19ft. Drilling then continued with a 66in diameter earth auger to refusal in the PWR a depth of 29ft as in Test Shaft 1.

The 66in earth auger was then exchanged for a 66in rock auger that advanced the shaft from a depth of 29ft to a depth of 44ft. A soft zone of PWR was noted from depths of 31ft to 35ft by the faster rate of drilling and much smaller quantity of rock fragments than were noted in the PWR in Test Shaft 1 or in the PWR below 35ft in this shaft. Below 35ft the penetration rate decreased and was recorded as 13in/5min at a depth of 41ft. The rate soon decreased to 3in/5min, a similar rate to the rock socket in Test Shaft 1. The shaft excavation was then terminated since the goal was to have the shaft bear on rock and not have a rock socket. Figure 4 shows some of the typical drill spoils from the PWR stratum in Test Shaft 2.

Final Test Shaft Configurations

Inspectors from GADOT, Geo-Hydro Engineers, an ADSC Technical Affiliate Member firm, and AMEC were present during construction of both shafts, as were personnel from ADSC Technical Affiliate firm, Dan Brown and Associates, PC (researchers). At the completion of each shaft, the excavations were hand cleaned and a 6ft deep inspection probe hole was drilled into the rock in the bottom of the shaft using pneumatic percussion tools. Inspectors from the participants each inspected the bases of the shafts to assess the quality of the rock below the base of the shaft (Figure 5). The inspectors reported that the hole was acceptable with only one or two small seams noted, and they approved the shaft for allowable bearing values ranging from 100 to 200 ksf.

The concrete mix was designed for a 28 day compressive strength of 4000psi and was delivered to the site at slumps ranging from 4½ to 7½ inches. Both shafts were placed the same day with one set of test cylinders cast from one of the loads placed in Test Shaft 1. A single compression test was performed the morning of the load tests on August 18, 2011 (20 days), indicating a compressive strength of 4280psi.

The final configuration of the two test shafts including the location of strain gauges are illustrated below in Figure 6. The following statements concerning the test shaft configurations can be made based on the excavation observations:

- Both shafts were bearing on similar rock material, but the rock did not provide the resistance to penetration that is typically classified as rock auger refusal since the auger penetrate rates were higher than 2in/5min. The rock appeared to be somewhat softer at Test Shaft 2.
- Test Shaft 1 had a rock socket in rock that was cored during the exploration as well as shaft exposed in good quality PWR.
- Test Shaft 2 did not have a rock socket and was constructed through two zones of PWR: an upper softer zone from 19 to 35ft

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below grade, and a lower stronger zone from 35 to 42.5ft (top of the O-Cell).

Load Test Results

Load tests of both test shafts were conducted by ADSC Associate Member firm, Loadtest, Inc. under direction of the research team on August 18, 2011. The tests were scheduled to occur as a part of a meeting of the Southeast Chapter of ADSC, and included lunch and a seminar on drilled shaft load testing presented by Dr. Brown. Fifty attendees were present to witness the completion of loading on Test Shaft 1 (photos in Figure 7 & 8), which was followed by load testing on Test Shaft 2. Attendees included local area practicing engineers as well as student groups from Georgia Tech and Auburn University. The results of the tests are summarized below.

Unit Side Resistance

Figure 9 shows the unit side resistance as a function of normalized upward O-Cell displacement expressed as a percent of the shaft diameter. Normalizing the displacement allows for a better direct comparison of the two different diameter test shafts. Table 1 lists the maximum average side resistance values from the tests.

The side resistance data suggest that the limit of the O-cell was reached before side resistance was fully mobilized, though the curves in Figure 9 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 in or less.

Unit Base Resistance

The measured unit base resistance from the two test shafts is provided in Figure 10 and Table 2. The plot shows the unit base resistance plotted versus normalized downward displacement of the O-Cell, where the displacement is expressed as a percentage of the diameter of the bearing area.

The base resistance of the two shafts was mobilized up to relative displacements of 4% (Test Shaft 1) and 8.3% (Test Shaft 2) or less of the diameter of the loaded area, but did not mobilize the geotechnical limit of the formation in terms of bearing capacity. For evaluation of the load vs deformation characteristics of the base resistance, the authors back-calculated an effective modulus of the rock mass using the formula for settlement of a rigid plate load test. The procedure for this exercise is outlined in the report. The values of rock mass modulus back-calculated for these two tests are shown on Figure 10.

The lower back-calculated value for Test Shaft 2 suggests that the base of the shaft was in softer rock than Test Shaft 1. The inspectors' reports for Test Shaft 2 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; however, this shaft was made with a 66in tool rather than a 40.9in 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 35ft was present immediately below.

(Continued on page 35)

<table>
<thead>
<tr>
<th>Shaft</th>
<th>Material</th>
<th>Maximum Unit Side Resistance (ksf)</th>
<th>Normalized Displacement (% Diameter)</th>
<th>Nominal Displacement (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Gneiss Rock</td>
<td>52</td>
<td>1.5</td>
<td>0.6</td>
</tr>
<tr>
<td>1</td>
<td>PWR</td>
<td>15</td>
<td>1.5</td>
<td>0.6</td>
</tr>
<tr>
<td>2</td>
<td>Soft PWR</td>
<td>2.5</td>
<td>0.5</td>
<td>0.3</td>
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<tr>
<td></td>
<td>(Upper PWR)</td>
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<td></td>
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Note: 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.
The test data suggest that the current rock auger refusal criteria of 2 in/5 min may be too restrictive. A less restrictive criterion of 3 in/5 min 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 (+ to 6 in/5 min for Test Shaft 1 and 3 in/5 min for Test Shaft 2) were faster than the typical 2 in/5 min 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 geometrical design parameters of the rock are not a limitation for design in this geology.

Table 2 – Maximum Unit Base Resistance

<table>
<thead>
<tr>
<th>Shaft</th>
<th>Material</th>
<th>Maximum Unit Base Resistance (ksf)</th>
<th>Normalized Displacement (%) Diameter</th>
<th>Nominal Displacement (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Gneiss Rock</td>
<td>850</td>
<td>4.0</td>
<td>1.7</td>
</tr>
<tr>
<td>2</td>
<td>Gneiss Rock</td>
<td>600</td>
<td>8.3</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Summary of Results and Implications For Design

The test results yield several implications for design of drilled shaft foundations in this geologic setting:

3. Unit base resistance values of 60 to 150 ksf was observed in the tests at a displacement of only 0.5% of the shaft diameter. These values are in the range of current maximum allowable values, including the maximum allowable on rock with more stringent criteria than encountered here. The test results indicate that much larger values nominal or ultimate are available at reasonable displacements.

4. The unit side resistance was over 50 ksf in rock with rock auger penetration rates of 4 to 6.5 in/5 min, 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 the construction with casing be performed in a way that does not contaminate the bond.

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Lawrenceville load tests to local practice, a meeting of a working committee was held on December 13, 2011 at the offices of GADOT Materials and Tests in Forest Park, Georgia. Those attending the meeting included Atlanta area practicing engineers, members of the ADSC Southeast Chapter, and the researchers (Dan Brown and Associates, PC).

The committee first spent time reviewing the Lawrenceville site test results and discussing the current local practice with respect to drilled shaft design. The group then discussed and developed recommended design values for base resistance and side resistance developed from the Lawrenceville load test results. Criteria were also developed that must be met to reliably apply these design recommendations for a particular site. Several key considerations were identified and included in the developed criteria:

1. Identification of weathered seams or substantially weaker material within the rock bearing formation below the base of the drilled shaft (through exploratory coring, air track drilling or probe holes).
2. Adequate exploration and site assessment to confirm conditions are consistent with or superior to those of the Lawrenceville test site in order to apply the test results.
3. Variations in geologic setting within the Piedmont geology in the Atlanta area.
4. The need to define consistent probe hole inspection criteria.
5. Concerns of using air track drilling instead of exploratory borings with rock core drilling for assessing adequacy of rock.
6. The report is not redefining the definition of rock for pay (Continued on page 39)

### Table 3 – Criteria for Base Resistance: 200 ksf allowable (400 ksf nominal)

| Rock Coring Criteria (Exploration and Site Assessment) | • NQ Double Barrel Coring  
• Recovery ≥ 90%  
• RQD ≥25%  
• No Significant decomposed layers observed in recovered core  
• Rock Mass Rating ≈ 50 or higher  |
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<tr>
<td>Probe Hole Criteria (Construction Verification)</td>
<td>Few small seams allowed, none &gt; ¼ inch for 1D below base. Seams are defined as occurring in most or all of the perimeter of the probe hole, not a soft isolated spot in the side of the probe hole.</td>
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</tbody>
</table>
| LLDH penetration rate w/ Rock Auger | • 3 inches/5 min for 66 in shaft  
• 5 inches/5 min for 40 in shaft  
• Engineering judgment for diameters outside range |
item purposes, but defining the design characteristics of the materials.

Tables 3 and 4 list the “Lawrenceville Criteria.” 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 rock coring criteria are provided for pre-construction engineering evaluation of the potential application of the Lawrenceville design criteria for a given site. The inspection criteria (probe hole and rig penetration rate observations) are provided for field verification that the foundation at a specific location has been installed to adequate depth to satisfy the design intent.

It is important to point out that these recommendations are not changing the definition of rock for pay item purposes, but defining the geotechnical design characteristics of the materials. The definition of rock as a pay item is a contractual issue based on certain definable parameters, such as penetration rates, drill torque, etc. The penetration rates and other measures given in the recommended Lawrenceville criteria are for design and verification purposes, not pay item purposes.

The working committee did not envision that the criteria would be used as an “either or” decision tool: 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 through 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

### Table 4 – Criteria for Side Resistance: 15 ksf allowable (50 ksf nominal)

| Rock Coring Criteria (Exploration and Site Assessment) | • NQ Double Barrel Coring  
• Recovery ≥ 90%  
• RQD ≥ 25%  
• No Significant decomposed layers observed in recovered core  
• Rock Mass Rating ≤ 50 or higher |
<table>
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<tbody>
<tr>
<td>LLDH penetration rate w/ Rock Auger</td>
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</table>
| Construction Requirements                             | Shaft must be constructed to avoid contamination or destruction of the sides of the excavation.  
• Drilled in dry or under slurry, no casing in rock.  
• Smaller casings for base inspection (when required) |
| Minimum Shaft Size                                     | 34 in to allow inspection with 30 in casing and not smear sides |

...a ceiling for drilled shaft design values in this geologic setting. These tests achieved unit bearing resistance values more than three times the recommended design value of 200 ksf 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 this report.

### Conclusions

This article has provided a brief summary of two drilled shaft load tests at a site in Lawrenceville, Georgia, along with some potential recommendations for the application of the results for the metro Atlanta drilled shaft market. It is hoped that the results of the test will be an incremental step towards more efficient design of drilled shafts in the metro Atlanta market, particularly with respect to utilizing the geotechnical resistance available in the Piedmont rocks. These tests should be a great addition to the body of knowledge in the Atlanta area in specific and the Piedmont geology in general and hopefully will encourage further research in these areas.

The authors and the Southeast Chapter of ADSC would like to thank all those that participated in the project, through providing material support or through contributions to the development of the recommendations. The time and effort contributed by Todd Barber, P.E. of Geo-Hydr Engineers deserves special recognition as he was instrumental in the site characterization as well as organizing the working group committee. Special thanks go to Foundation Technologies for graciously providing the test site, to McKinney Drilling for constructing the test shafts, to Loadtest, Inc. for the testing, and to Bruce Long of ADSC Contractor Member, Long Foundations for the management of the effort. GADOT, GeoHydro Engineers, and AMEC are also recognized for their contribution of laboratory services, field inspectors, and participation in the working committee to develop recommendations for applying the test results to local practice.

*The ADSC does not support a "rig-specific" approach to defining rock. (Editor)