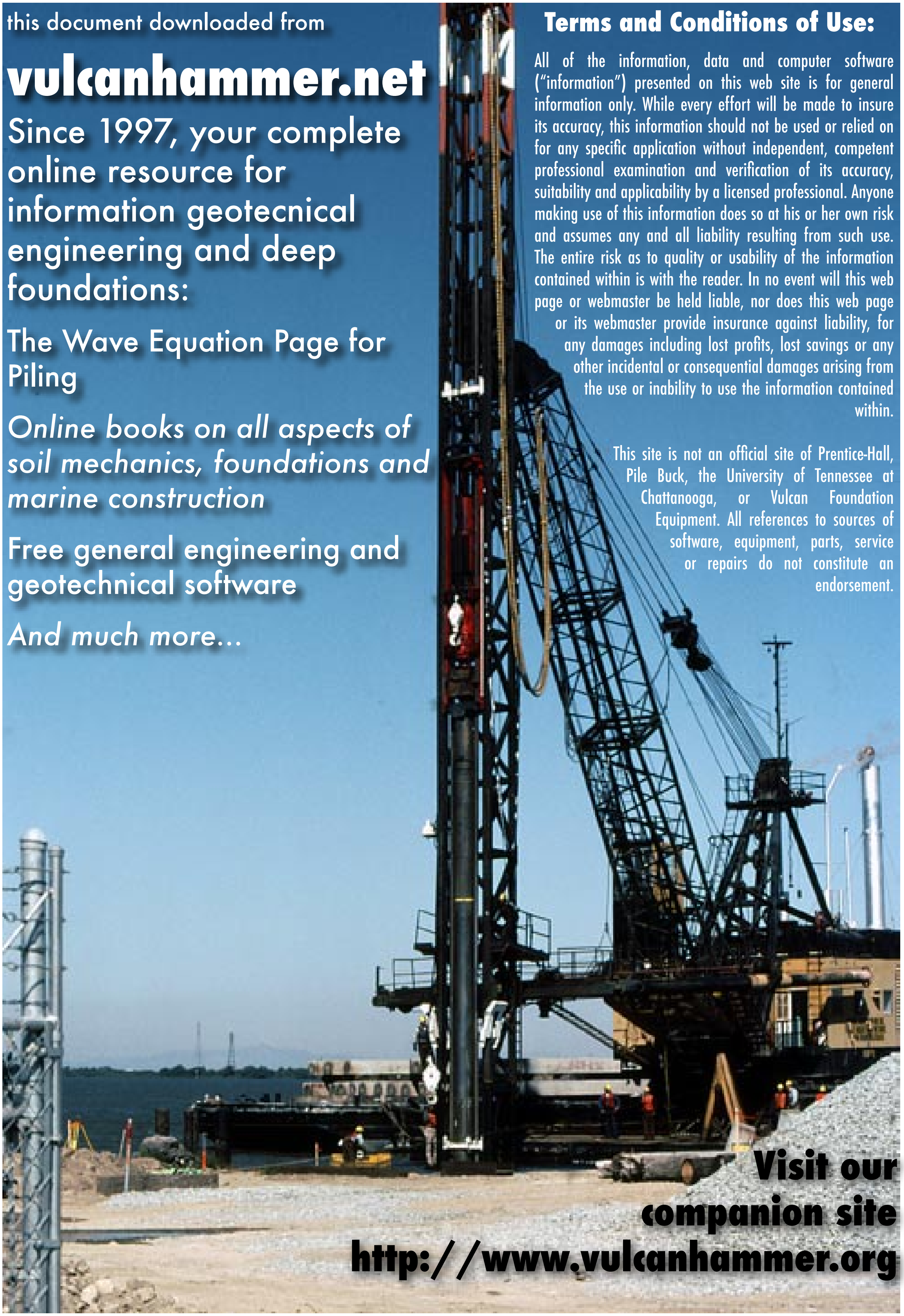

Geotechnical Engineering within the Piedmont Physiographic Province

by

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INTRODUCTION

The purpose of the study described in this report is to compile and synthesize information on the geology and engineering properties of Piedmont residual soils, and on geotechnical engineering design methods appropriate for use in these soils. Considerable information was found on geology, classification, sampling, and testing of Piedmont residual soils. Less information has been published concerning geotechnical engineering design methods and the performance of foundations in Piedmont residual soils. Guidelines for anticipating excavatability, for estimating settlements of shallow foundations, and for estimating capacities of drilled shafts have been published, and are summarized and illustrated here. However, many subjects of interest, such as effects of pile driving, behavior of driven piles, and use of ground improvement techniques, have not been treated as extensively in the published literature as might be anticipated, given the size of the Piedmont region and the amount of engineered construction in recent years. It seems likely that a great deal of information regarding geotechnical engineering in the Piedmont has been accumulated, which would be of great value to the profession if published.

GEOLOGY

The Piedmont physiographic province is located in the eastern United States and extends from Alabama into Georgia, the Carolinas, Virginia, Maryland, Pennsylvania and southeastern New Jersey. As shown in Figure 1, the Piedmont province underlies several major cities, including Atlanta, Charlotte, Raleigh, Richmond, Washington-DC, Baltimore, and Philadelphia. It is bounded to the west by the Blue Ridge and Appalachian physiographic provinces and to the east by the Coastal Plain physiographic province. The Piedmont is characterized by relatively low relief and rolling topography, with elevations ranging from 400 to 1200 feet. Drainage of the Piedmont is generally directed south and southeast towards the Atlantic Ocean (Goldberg and Butler, 1989).

The Piedmont province is underlain by metamorphic rock formations, generally consisting of gneisses and schists of Precambrian age. Parallel banding, resulting from the segregation of minerals during metamorphism, characterize these crystalline rocks. The bands usually appear contorted or twisted, although they remain parallel and generally dip in a consistent direction. The metamorphic formations include various intrusive igneous rocks, such as granite and diabase. These igneous intrusions vary in size from large masses (hundreds of feet wide) to narrow bands (several inches wide). The igneous rocks are considerably younger than the metamorphic rocks, although the exact age is unknown (Sowers, 1954).

The Piedmont has experienced various episodes of heat, pressure, and structural deformation. Heat and pressure created varying degrees of metamorphism, while the structural deformations and folding produced joints and foliations. Joint set orientations are described as uniform in some areas and random in others. Faults exist across the region. The fault lengths range from tens of feet to miles, and displacements range from a few feet to thousands of feet (Sowers and Richardson, 1983).

Figure 1. Piedmont physiographic province (from Mayne, 1997).

RESIDUAL SOIL FORMATION AND THE WEATHERING PROFILE

Residual soils are products of physical and chemical weathering of the underlying bedrock. Depending on the degree of weathering, the soil can retain much of the fabric, or structural features, of the parent rock. Weathering generally decreases with depth; however, there is generally no well-defined boundary between soil and rock. Typical weathering profiles for metamorphic and igneous environments are shown in Figure 2.

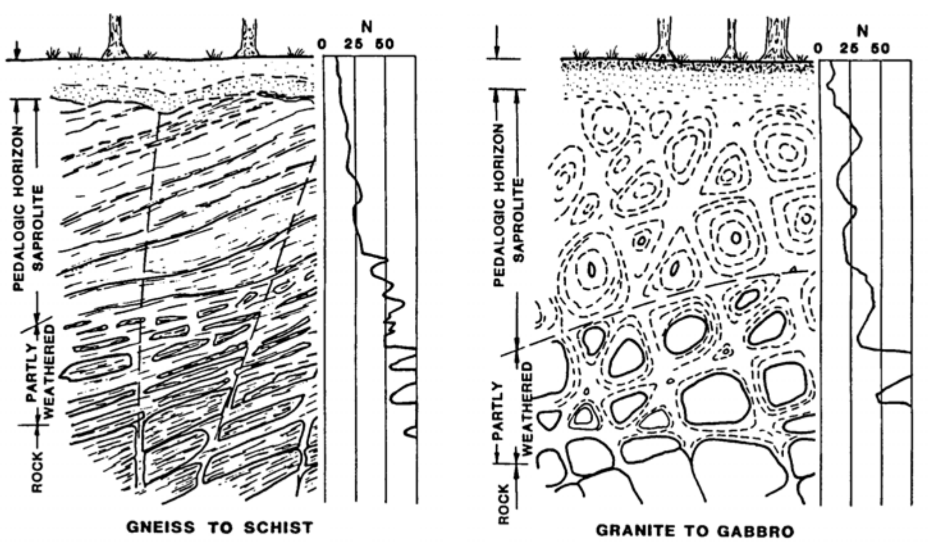

Figure 2. Weathering profiles (from Sowers, 1994).

Several weathering profile or classification conventions exist for residual soils in the Piedmont. Sowers (1963), Deere and Patton (1971), Law/MARTA-Metropolitan Atlanta Rapid Transit Authority (from Richardson and White, 1980), and Schnabel Engineering Associates (from Martin, 1977), have developed various classification systems based on weathering. Table 1 shows the various classification systems and weathering zones. Zone boundaries are determined based on Standard Penetration Test (SPT) N-values, rock core recovery, and rock quality designation (RQD) values.

Table 1. Classification systems of weathering profiles (from Wilson and Martin, 1996).

<table>
<tr><th>Sowers (1963)</th><th colspan="2">Deere & Patton (1971)</th><th>Law/MARTA (Richardson & White, 1980)</th><th>Schnabel Engineering Associates (from Martin, 1977)</th></tr>
<tr><td rowspan="2">Soil
N=5-50</td><td rowspan="3">I
Residual Soil</td><td>IA
A Horizon</td><td rowspan="2">Upper Horizon
No Residual Structure</td><td rowspan="3">Residual Soil
N < 60</td></tr>
<tr><td>IB
B Horizon</td></tr>
<tr><td>Saprolite
N=5-50</td><td>IC
C Horizon</td><td>Saprolite</td></tr>
<tr><td rowspan="2">Partially Weathered Rock - Alternate Hard & Soft Seams
N>50</td><td rowspan="2">II
Weathered Rock</td><td>IIA
Transition From Residual Soil to Partially Weathered Rock</td><td>Partially Weathered Rock
N>100
Core Recovery<50%</td><td>Disintegrated or partially weathered rock
N≥60</td></tr>
<tr><td>IIB
Partly Weathered Rock</td><td>Rock Core Recovery>50%
RQD<50%</td><td rowspan="2">Rock
N≥100/2”
Core For Confirmation</td></tr>
<tr><td>Rock
RQD>75%</td><td colspan="2">III
Unweathered Rock
RQD>75%</td><td>Sound Rock
RQD>50%
Core Recovery>85%</td></tr>
<tr><td colspan="5">RQD = Rock Quality Designation
N=Standard Penetration Test N-Value (blows/foot)</td></tr>
</table>

For example, the Sowers (1963) weathering profile consists of four zones. The upper zone consists of completely weathered material, or soil, with SPT N-values between 5 and 50 blows/foot. A second intermediate zone is termed 'saprolite.' This material retains the relict structure of the parent rock, although its strength resembles that of soil. Pavich (1996) states that the saprolite zone comprises more than 75% of the material overlying bedrock in the Piedmont province. The third zone is partially weathered rock with alternating seams of saprolite and weathered rock. SPT N-values would be greater than 50 blows/foot in this zone. And finally, unweathered rock, or bedrock, exists below the partly weathered rock. This material requires rock coring and is characterized by RQD values of 75% or more. As Martin (2001) notes, the most challenging delineation in the weathering profile, particularly when designing deep foundations, is to accurately determine the transition from partially weathered rock to bedrock. This transition is usually noted at SPT refusal, however SPT refusal can be interpreted several different ways. ASTM D 1586 suggests refusal is reached if less than 6-inches of penetration is made after 50 blows, or if no penetration is made after 10 blows. Many engineers, including Martin, who practice in the Piedmont prefer testing beyond ASTM's refusal criterion, determining refusal near 100 blows/2", with confirmation of bedrock required by coring.

ENGINEERING CLASSIFICATION

According to Sowers and Richardson (1983), conventional soil classification systems (i.e. Unified Soil Classification System) can only be applied to the completely weathered soil zones in residual soils. Surficial and completely weathered residual soils typically classify as silty sands (SM), sandy silts (ML), lean clays (CL), and fat clays (CH). Fines range from 30 to 60 percent and mica is generally present (Wilson and Martin, 1996).

For the saprolite and partly weathered rock zones, conventional classification systems are not applicable because index tests do not account for soil structure and fabric. Sowers (1985) suggests that analyses developed for fractured rock would better characterize the behavior of these zones. It is also suggested that the void ratio and mica content of the material are more useful in identifying behavior problems rather than typical index tests.

It can be expected that void ratios vary considerably depending on the degree of weathering. As shown in Table 2, Sowers and Richardson (1983) suggest a range of values for each zone of the Sowers (1963) classification system.

Table 2. Void ratio through the weathering profile (from Sowers and Richardson, 1983).

Zone	Void Ratio Range
Soil in which minerals have leached out (topsoil & organics)	0.6 - 1
Soil in which minerals have accumulated	0.4 - 0.8
Saprolite	0.7 – 3
Partially weathered rock	0.1 - 0.5
Rock	0.02 or less

Note that the largest variation of void ratio was found in the saprolite zone, where the material has the strength of a soil, but the relict structure of the underlying bedrock. Mica content also has a significant influence on void ratio in this zone.

ENGINEERING PROPERTIES

Permeability, compressibility, and shear strength of the residual material within the Piedmont region reflect the complexity of the weathering profile. Many residual soils are non-homogeneous and anisotropic; therefore, engineering properties may change depending on direction and may vary considerably across a project site (Sowers, 1954). With sedimentary soils, horizontal effective stress can often be related to vertical effective stress by a constant proportion, namely the at-rest earth pressure coefficient, k_0. However this is not the case with residual soils, and Sowers (1985) cites two main reasons for this: (1) tectonic stresses may only be partially released during weathering; and (2) void ratio increases during weathering and at shallow depths, where the resisting gravity force is less than the resistance of the soil mass, vertical expansion will result in a horizontal effective stress greater than the vertical effective stress. Sowers (1985) also notes that in some cases, the stress field can change due to erosion if the soil is exposed to the elements, as in the case of a steep hillside.

Permeability

As is the case with void ratio, it can be expected that the permeability will vary considerably depending on the degree of weathering. As shown in Table 3, Sowers and Richardson (1983) suggest a range of values for each zone of the Sowers (1963) classification system.

Table 3. Permeability through the weathering profile (from Sowers and Richardson, 1983).

Zone	Permeability Range
Soil in which minerals have leached out (topsoil & organics)	10^{-3} to 10^{-5} cm/sec, isotropic
Soil in which minerals have Accumulated	10^{-5} to 10^{-7} cm/sec, isotropic
Saprolite	10^{-4} to 10^{-6} cm/sec, anisotropic
Partially weathered rock	10^{-1} to 10^{-5} cm/sec, anisotropic
Rock	Impervious

Sowers and Richardson (1983) indicate that flow in the partially weathered rock zone is anisotropic, with permeability parallel to the foliations typically 10 times greater than that of the permeability perpendicular to the foliations. However, as with any rock structure, fractures will often transmit more water than the intact materials. It is strongly advised that laboratory testing for permeability be supplemented with field testing where practical.

Matheson (1996) also conducted permeability tests, however only on material in the saprolite zone. He reported a permeability range of 0.1 to 5 ft/day ($2x10^{-3}$ to $4x10^{-5}$ cm/sec), which is close to the range estimated by Sowers and Richardson (1983).

Compressibility

According to Sowers and Richardson (1983), "a partly saturated saprolite exhibits significant initial consolidation, well-defined primary consolidation, and usually significant continuing secondary consolidation" (p14). It is generally acknowledged that consolidation within the Piedmont region typically occurs relatively quickly, partially due to the materials' high permeability. Sowers and Richardson (1983) further note that between ¼ and ½ of the ultimate settlement of a structure will occur during construction followed by a year or two of hydrodynamic consolidation, with the rate of compression decreasing with time.

Residual soils typically exhibit an apparent preconsolidation stress, possibly due to weathering related volume changes, residual bonds between particles, and possible residual lateral tectonic stresses associated with formation uplift and folding (Sowers, 1994). Sowers further notes that this apparent preconsolidation varies erratically with no discernable relation to past or present stress, but typically ranges between 1 and 5 ksf.

Barksdale et al. (1982) report that residual soils having undergone the least amount of weathering appear to be preconsolidated the most, and typically the softer, more weathered, residual soils of the southeast appear to be preconsolidated between 2 and 4 ksf.

It should also be noted that the presence of mica in residual soils increases its compressibility (Feist, 1992). Percentages of mica in the Piedmont region profiles typically vary from 5 to 25 percent (Martin, 1977).

Shear Strength

The shear strength of residual soil is controlled by the presence of relict rock structure and fissures in the material. Mayne (1992) notes "a major difficulty occurs in the interpretation of engineering properties from in-situ tests of these materials since they behave strictly neither as clay nor sand...they exhibit certain aspects that are characteristic of both cohesive and cohesionless soils" (p91).

Sowers and Richardson (1983) evaluated both total and effective strength parameters for partially saturated and saturated residual soils located in Atlanta. Results indicated a higher apparent cohesion for the partially saturated samples, which is believed to be due to capillary tension. Sowers (1963) found that true cohesion is less than the apparent cohesion. The measured cohesion is the result of residual unweathered bonds as well as semi-soluble precipitation bonding produced during the weathering process. It should be noted however, that despite its presence, cohesion is often ignored in engineering design.

Dynamic Properties

The dynamic properties of Piedmont residual soils appear to be at the forefront of current research. Borden et al. (1996), Wang and Borden (1996), and Schneider et al. (1999) have all advanced the understanding of the response of residual soils in the Piedmont to dynamic forces. Borden et al. (1996) performed dynamic laboratory tests on residual soils including resonant column and torsional shear tests, and reported that normalized shear modulus and damping values were in the range of those reported for transported sands, silts and clays. As might be expected, the characteristics of the Piedmont soil did not follow typical clay or sand behavior, but rather fell somewhere between the two. "In general, the normalized shear modulus decreased and damping increased at a rate faster than that for clays but slower than that exhibited by sands" (p821). Schneider et al. (1999) also ran laboratory tests, and reported that the lab results compared well with results of several in-situ tests (cross-hole, seismic flat dilatometer, seismic piezocone, and surface wave tests) performed. From this, it was concluded that a strain-based correction factor is not necessary in Piedmont residual silts.

GEOTECHNICAL INVESTIGATION, SAMPLING, AND TESTING

Geotechnical investigations, sampling, and testing in the Piedmont region can be challenging due to the variable subsurface conditions and complex weathering profile. Conventional subsurface investigation methods have been used with success in the Piedmont, including auger drilling, wash drilling, test trenches, etc. Sampling of residual soils, on the other hand, has been a source of concern in geotechnical practice. Due to the inherent fabric and relict parent structure of some residual soils, sampling disturbance can have a profound impact on laboratory testing results.

Sampling Methods

Sampling techniques in the Piedmont residual soils include the following:

- Split spoon sampler. Used in conjunction with the SPT test (ASTM D1586), the split spoon sampler is the most common sampling tool. Two and three-inch diameter samplers are most common.

- Denison sampler. A Denison sampler (ASTM D3550), as shown in Figure 3, is typically 4 to 6 inches in diameter and is used for sampling hard saprolite zones (greater than 20 blows per foot). According to Sowers (1985), a Denison sampler provides relatively good quality samples from the saprolite and occasionally partially weathered rock zones, although if there are alternating hard and soft layers, samples from the soft layers can easily be lost.

- Thin-walled (Shelby) sampler. The Shelby tube (ASTM D1587) can be used in soils that are completely weathered. Rock fragments, such as those in the saprolite zone, will hinder the sampler.

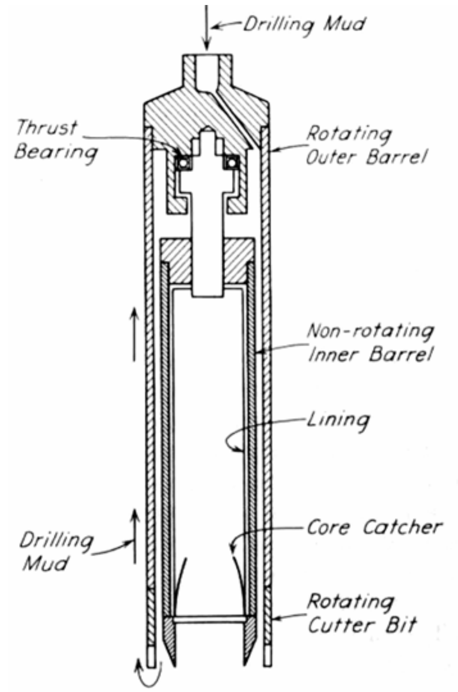

Figure 3. Example of Denison sampler (from Terzaghi et al., 1996).

In-situ Testing

In-situ testing techniques include the following:

- Standard Penetration Test (SPT). The SPT test (ASTM D 1586) is the most common in-situ test used for residual soils. Advantages include: (1) it provides representative (disturbed) samples for visual classification and lab tests, such as water content, gradation, and Atterberg limits; (2) it offers the possibility of continuous testing and sampling throughout the subsurface; (3) it can penetrate the entire weathering profile down to the soil-rock boundary; (4) it is relatively inexpensive to perform; and (5) SPT N-values are widely correlated, so they can be used to design shallow and deep foundations alike.

  A few detractions of the SPT test in the Piedmont include: (1) the dynamic penetration action remolds the soil and possibly destroys relict rock structure; (2) depending on the frequency of sampling, pinpointing different strata can be difficult; and (3) low or inaccurate SPT N-values, not reflective of the undisturbed in-situ properties of the soil, may lead to over-conservative design.

  Kelley and Lutenegger (1999) report the use of the SPT-T (torque) tests coupled with DCPT (driven cone penetration) tests as a quick and relatively inexpensive way to characterize the subsurface conditions in the Piedmont. Martin (2001) also notes SPT-T tests can be used as a screening method before lab testing to measure strength and compressibility.

- Cone Penetration Test (CPT). Although it has been used to characterize residual soil profiles for years, there is not much documented history of specific uses of the CPT (ASTM D3441/D5778) in the Piedmont. Part of the reason, as Mayne (1992) notes, is that rock fragments (commonplace in the saprolite zone) may prevent full-depth penetration to intact material. It is also worth noting that standard CPT probes cannot penetrate the partially weathered rock zone. When it is used, the CPT is often supplemental to other in-situ test methods performed at a site. The data gathered from a CPT test can be used to develop a soil modulus profile, which is an essential parameter for settlement predictions. Hezagy et al. (1997) report how statistical methods can be used to interpolate between soundings to minimize uncertainties in subsurface conditions, and Martin and Mayne (1998) and Finke (1998) review seismic piezocone testing in the Piedmont.

- Dilatometer (DMT). Like the CPT, the dilatometer may be hindered by the presence of rock fragments, however Mayne (1992) notes the dilatometer blade may be driven through some obstructions. Mayne and Frost (1988) found that estimates of overconsolidation ratio and soil modulus compared reasonably well with laboratory results and back-calculated field performance results. Likewise, Mayne et al. (1999) report a correlation between soil modulus obtained using a flat dilatometer and the settlement of drilled shafts in the Piedmont. Martin and Mayne (1998) also report experimenting with a seismic flat dilatometer in the Piedmont, and found it produced a reasonable profile when compared with other in-situ seismic tests.

- Pressuremeter (PMT). The PMT (ASTM D4719) can be used successfully in loose to very dense residual soils to obtain a soil modulus, which can be used to calculate settlement of shallow foundations. Barksdale et al. (1986) found the pressuremeter useful in evaluating the stiffness of thinly stratified dense residual soil and partially weathered rock. Lambe and Riad (1990) found that the pressuremeter test was difficult to perform in areas where rock fragments exist. Because many engineers may not be familiar with the pressuremeter test despite its usefulness in completely weathered soil zones, Appendix A describes the PMT testing procedure, explains how to interpret the data, and runs through an example problem.

- Geophysical Methods. Mayne and Harris (1993), Mayne (1997), Mayne and Dumas (1997), and Brown and Vinson (1998) report using Spectral Analysis of Surface Waves (SASW) to characterize the subsurface conditions in the Piedmont. By measuring the velocities of several waves of different wavelengths, a profile of material properties can be developed (Mayne and Harris, 1993). SASW is useful for determining groundwater levels and alternating hard and soft layers, and if nothing else, can indicate transition zones between materials of varying stiffness.

It is also possible to use seismic crosshole testing (ASTM D4428), electrical resistivity (ASTM D6431), seismic refraction (ASTM D5777), and ground penetrating radar (ASTM D6432) to help delineate the boundaries of the weathering profile in residual soils. However, because of the difficulty associated with interpreting the results, they have not been frequently used in practice (White & Richardson, 1987).

- <u>Permeability</u>. Matheson (1996) reported that the measurement of hydraulic conductivity of saprolites is difficult. Small-scale borehole ‘slug’ or ‘falling head’ tests are typically used, whereas aquifer-pumping tests are seldom used. Laboratory and small-scale borehole tests generally underestimate the larger scale hydraulic conductivity that controls flow during construction. Vepraskas et al. (1996) reviewed several ways in which the low permeability transition zone between soil and saprolite can be identified for wastewater discharge.

Laboratory Testing

Conventional laboratory tests have been performed on Piedmont residual soils with mixed results. Careful sampling, trimming and testing of samples is essential to minimize sample disturbance and preserve the in-situ soil structure.

In a study on landslides and slope stability in North Carolina, Lambe and Riad (1990) performed laboratory tests on Piedmont residual soils. Shear strength tests included direct shear, triaxial compression, and ring shear. Based on their experience, Lambe and Riad favored the use of the direct shear test over the triaxial test in measuring shear strength for the following reasons: (1) direct shear samples are easier to trim; (2) direct shear results showed less variability than triaxial results, so fewer tests were required; and (3) three strength envelopes (peak, remolded, and residual strengths) can be determined from the direct shear test, as opposed to one envelope from triaxial tests. However, shear strengths determined from direct shear probably do not represent the field strength due to the fact the small samples have fewer foliation planes and fissures. For residual shear strength, it was determined that direct shear typically gave higher values than ring shear. Ring shear tests provide a simple and relatively fast means for determining a lower bound for strength.

Lambe and Heartz (1988) also performed consolidated drained triaxial testing on Piedmont residual soils. Undisturbed samples were obtained using Shelby tubes and block samples. Specimens generally classified as micaceous silts (MH) and were characterized by foliations (or layers) dipping as much as 60 degrees from vertical. "To minimize breakage along weakness planes, samples were confined during trimming by a 12.5-cm-long and a 3.8-cm-inner diameter cylindrical steel tube having a 0.3-cm-long and 3.6-cm-inner-diameter cutting shoe" (p313). During application of isotropic consolidation pressures less than the pre-stress pressure, some samples tilted, having consolidated more perpendicular than parallel to the plane of layering. Three modes of failure were experienced during shear: (1) symmetrical bulging mode; (2) failure along a plane of weakness without tilting of the top of the sample; and (3) failure showing tilting of the top cap. Bending indicates the sample experienced non-uniform stresses and strains.

Sowers (1954) found that shear strengths tested by direct shear tests and quick (undrained) triaxial tests were similar unless definite planes of weaknesses, such as bands of mica, were present. In addition, comparisons made between quick (undrained) triaxial tests and slow (drained) triaxial tests show that the difference in shear strengths measured from the two tests was negligible for micaceous silty sands (e=1.4). Sowers also noted that remolding had little to no effect on the effective friction angle, which indicates that internal friction is not greatly dependant on soil structure.

EXCAVATABILITY

Excavation is a major concern in residual soils since the material and its properties vary so much within the weathering profile. Engineering design at or near the soil-rock boundary requires adequate knowledge of the geology, including rock strength properties, strike and dip, joint spacing, etc. Discrepancies between estimated and actual excavation quantities can often lead to claims and litigation.

The definition of the soil/rock boundary is a difficult challenge for engineers and contractors. Smith et al. (1991) reviewed various definitions of the soil/rock boundary for different engineering applications in the Piedmont, and concluded that different definitions are required for different applications. To demonstrate this, Smith et al. presented three case histories (an excavation for a basement, an excavation for a cut-off trench for a dam, and an excavation for a drilled shaft) along with three different definitions of the soil/rock boundary.

White and Richardson (1987) conducted a survey of geotechnical engineers and contractors regarding the methods of investigating excavatability of Piedmont residual soils. Highlights of their findings include:

- The most widely used investigative method was the Standard Penetration Test (SPT). Augering without SPT sampling and percussion drilling are also used as qualitative approaches to assessing the condition of bedrock. Consultants recommended the use of seismic refraction as a supplement to borings and/or test trenches. For the most direct means of assessing excavatability, contractors and consultants recommended performing a test excavation section at a particular site.

- Borings, spaced at 100 feet, were recommended for equi-dimensional excavations (i.e. structures). A spacing of 300 to 600 ft. was considered reasonable for roads or utility excavation.

- Consultants generally interpreted excavatability based on SPT N-values, seismic compression wave velocity, and rock core recovery. Ranges and typical values used to determine excavatability are presented in Table 5. It is important to realize, as Smith (2001) points out, that the SPT N-values denoting the excavation boundaries in Table 4 are near the limit of the SPT test (i.e. 100 blows/4") for producing meaningful data.

- Consultants and contractors estimated that subsurface investigation costs are typically 1 to 2 percent of excavation costs.

Table 4. Excavation techniques based on in-situ testing (from White and Richardson, 1987).

Material	**SPT N-value**		**Compression Wave Velocity (fps)**		**Excavation Technique**
	Range	Typical	Range	Typical	
Soil/Partially Weathered Rock Boundary	40-100/6"	80-100	2,500-4,500	3,500	Boundary between conventional means and ripping
Partially Weathered Rock/Solid Rock Boundary	100-refusal	100/4"	4,000-8,000	6,000	Boundary between ripping and blasting

Given the variability of the typical soil profile in the Piedmont region, preparing contract and bidding documents can be a challenge for the geotechnical engineer. Kulhawy et al. (1991) report that all too often, ambiguous and inappropriate terms are used to describe subsurface materials and, in particular, rock. The best way to prevent discrepancies, according to Kulhawy et al., is to use direct and explicit language when describing the material to be excavated. Smith et al. (1991) and Smith (2001) go one step further, in that they recommend specifications in all excavation work in the Piedmont be bid as "unclassified" with the following provisions: (1) provide exploration data, borings, laboratory data, etc.; (2) request the Contractor to base his bid on the aforementioned baseline data (plus any other data he chooses to acquire), as well as his own experience, equipment, personnel, and schedule; and (3) include a differing site condition clause that clearly indicates the contractor's right to rely on the data provided.

A guideline on using a "differing site condition" clause in contract documents has been prepared by the Federal Highway Administration (1996). The document also provides recommendations related to disclosure and presentation of subsurface information and contract document language.

DESIGN CONSIDERATIONS

Although there is a considerable amount of literature on the geology and soils of the Piedmont province, much of it focuses on classification, sampling, and testing. Despite the numerous metropolitan areas within the province (Washington-DC, Charlotte, Atlanta, etc.) there is a sparse amount of information relating to the specifics of design in the native residual soils. And the little that has published relating to engineering design and performance focuses on either prediction of settlement of shallow foundations or drilled shaft design and/or performance. For these reasons, settlement of shallow foundations and design of drilled shafts are the only subjects discussed in detail in the ensuing section. Other aspects of design in the Piedmont are less thoroughly documented in the literature. Some useful references on other aspects are listed in Table 5.

Table 5. Other sources of information for geotechnical design subjects in the Piedmont region.

Subject	Source(s) of Information
De-watering	Wirth and Ziegler (1982) Peterson, Brand, Roldam, and Sommerfield (1999)
Environmental contamination Detection and/or remediation	Corley, Martin, and Macklin (1999) Kirtland, Aelion, and Widdowson (2001)
Embankment dams	Wilson and Martin (1996)
Ground anchors	Weber (1982)
Lateral/uplift loading on piles/drilled shafts	Watson (1970) Law Engineering (1998) Anderson, Grajales, Townsend, and Brown (1999) Lutenegger and Adams (1999)
Mini-piles	Sanders, Hussin, and Hull (1999)
Pressure Injected Footings	Neely, Waitkus, and Schnabel (1987)
Reinforced Earth	Elias and Swanson (1983)
Soil Nailing	Sigourney (1996)
Slope Stability	Deere and Patton (1971) Lambe (1996) Peterson, Brand, Roldam, and Sommerfield (1999)
Stone Columns	Tice and Hussin (1999) Wissman, Moser, and Pando (2001)
Surcharging	DeMello, Ceppolina, and DeOliveira (1984)
Waste Disposal Fills	Sowers (1973)

Settlement of Shallow Foundations

Residual soils are the product of in-place weathering of igneous and metamorphic rocks, and behave differently than deposited soils. They tend to be non-homogeneous and anisotropic. The design approach to foundation systems on residual soils is typically very conservative, primarily due to the fact that the behavior of these residual soils is not completely understood and is difficult to predict.

Methods to estimate settlement:
Settlement is dependent on applied load and the deformation characteristics of the soil, which can be measured using a variety of methods. The more commonplace methods include: (1) in-situ testing such as the Menard Pressuremeter (PMT), dilatometer (DMT), and cone penetration test (CPT); (2) correlations based on standard penetration test (SPT) N-values; (3) one-dimensional consolidation test results; (4) data from plate load testing; (5) back-calculations based on elastic theory; and (6) stress path testing in the laboratory (Feist 1992).

After determining the soil parameters, one or more of the following methodologies can be used to estimate settlement. Descriptions and example calculations for most of the methodologies can be found in Appendix B.

Schmertmann strain influence method
Originally developed by Schmertmann (1970), and later modified by Schmertmann et al. (1978) to estimate settlement in sands, this method can be applied to residual soils because it is based on strain influence factors and soil modulus. Values of soil modulus, E_s, can be correlated from the data from one of three following in-situ tests:

- Cone Penetrometer Test (CPT): Schmertmann (1978) developed a straightforward correlation between tip resistance and soil modulus.

- Pressuremeter Test (PMT): The soil modulus of deformation, E_s, may be determined from pressuremeter test data. The test data yields E_{PMT}, which, Martin (1977) concluded is equivalent to the soil modulus, E_s, based on back-analysis of building settlements in the Piedmont.

- Standard Penetration Test (SPT): Soil modulus may also be determined from correlations with SPT N-values. Martin (1987) relates E_{PMT} (which by the previous paragraph equals E_s) to N-values recorded throughout the Piedmont. However, this correlation has been found to overestimate settlements, and Martin (1987) suggests a correction that reduces the predicted settlement by 40% when using the SPT correlation.

The method is explained in detail in Appendix B, and the writers have prepared a spreadsheet program to predict settlement based on Schmertmann's strain influence method. The program is on the floppy disk included with this report, and an explanation of input into the program and an example calculation are included in Appendix B.

Martin (1977, 1987), Barksdale et al. (1986), Borden and Sullivan (1988), and Wilson (1988) have all documented the applicability of this method in the Piedmont.

<u>Modified Meyerhof SPT method</u>

In this method, formalized by Duncan and Buchignani (1976), settlement is predicted using a correlation with bearing pressure, SPT N-values and footing width. Barksdale et al. (1986), Harshman (1989), and Wilson (1988) provide details of the use of this method in the Piedmont.

<u>Peck, Hanson, and Thornburn SPT method</u>

For quick preliminary estimates of settlements, Barksdale et al. (1986) recommend the use of the original 1948 Terzaghi and Peck curves as reported in Peck, Hanson, and Thornburn (1953) for sands, which use uncorrected SPT N-values. Although the curves were originally developed to relate footing width and SPT N-value to allowable bearing pressure for settlement less than or equal to 1", they can also be used to estimate settlement. Barksdale et al. (1986) and Willmer et al. (1982) both report using this method in the Piedmont to estimate settlement.

<u>One-dimensional consolidation tests</u>

Conventional one-dimensional consolidation tests have been shown to overpredict settlements in residual soils. Sowers (1994) reports that "because the measured settlements on saprolites from gneiss and schist based on laboratory tests on undisturbed samples are consistently from ⅔ to ¾ of those computed from conventional laboratory tests on representative undisturbed samples in the various horizons with significant void ratios, a correction factor of about 0.8 is applied to the computed values by many geotechnical engineers in the Southeastern USA" (p1698).

Martin (1977) goes so far as to suggest "consolidation testing is impractical for most residual soils unless undisturbed block samples can be obtained from large diameter shafts" (p200). Obtaining good-quality samples of material with SPT N-values greater than 15 and/or material containing rock fragments is difficult. Use of this procedure to estimate settlement in the Piedmont is discussed by Sowers and Glenn (1965), Willmer et al. (1982), Barksdale et al. (1986), Borden and Sullivan (1988), and Wilson (1988).

Menard PMT method

This method, originally developed by Menard and Rousseau (1962) and modified by Baguelin et al. (1978), estimates settlement based on E_{PMT}, bearing pressure, footing shape and soil type. Although the method is applicable in a wide range of soil types, including residual, it is most often used in France where it was developed. Use is not widespread in the United States. Although the method was employed for comparisons in this report, due to its complexity and empirical nature, it is not described in detail in Appendix B. Details are provided in Menard and Rousseau (1962), Baguelin et al. (1978), Barksdale et al. (1986), Gambin and Rousseau (1988), and Wilson (1988).

Plate Load Testing

Plate load tests are generally reserved for large projects where bearing capacity and settlement are crucial. Feist (1992) provides a discussion on the test, including advantages, disadvantages, and methods of interpretation.

Stress Path Testing

Limited stress path testing has been performed on residual soils, although Barksdale et al. (1982) believe that the stress path method is theoretically the best approach. The method is not widely used in geotechnical practice since few firms have the capability to conduct the test, and the test is relatively expensive.

Conclusions regarding reliability of methods:

Approximately 25 different case studies of settlements of shallow foundations in the Piedmont were evaluated to determine the accuracy of the aforementioned methods. Reliability was evaluated for 5 methods (the Schmertmann strain influence method, the Menard PMT method, the modified Meyerhof SPT method, the Peck, Hanson, and Thornburn SPT method, and the one-dimensional consolidation method) by comparing measured settlements (S_m) and predicted settlements (S_c) as described by Duncan (2000). A summary of the evaluations, which include coefficient of variation and bias, is shown in Table 6. More information on the reliability of each method can be found in Appendix B.

The comparison of methods shown in Table 6 and Appendix B reveal that almost all of the methods analyzed overpredict the settlement of shallow foundations in the Piedmont, as indicated by the fact that their average values of S_c/S_m were greater than 1.0. The closer the value of bias is to 1.0, and the smaller the coefficient of variation (standard deviation divided by average), the more accurate the method. In addition to being somewhat difficult and cumbersome to use, the Menard PMT method was also found to often predict settlements smaller than the measured settlements.

Table 6. Coefficient of variation and bias of several settlement estimation methods.

Estimation method	**Number of Comparisons**	**Coefficient of Variation**	**Bias $(S_c/S_m)_{avg}$**
Schmertmann strain influence method			
with PMT test data	10	0.39	1.09
with SPT test data	23	0.75	1.79
with SPT test data and Martin's correction	23	0.35	1.07
Modified Meyerhof SPT method	19	0.70	1.41
Peck, Hanson, and Thornburn SPT method	13	1.80	2.95
One-dimensional consolidation test method	27	1.17	1.74
Menard PMT method (using equations by Baguelin et al., 1978)	9	0.33	0.75
S_c = calculated settlement S_m = measured settlement			

Drilled Shafts

Drilled shafts are a popular foundation type for heavily loaded structures in the Piedmont. However, there is no well-established design methodology for drilled shafts, and in some instances, local regulations inhibit cost-effective design and construction (Gardner, 1987). Mayne and Harris (1993) noted the skin friction developed in these residual soils has been a particularly controversial issue, as well as the relative proportions of load transferred to the shaft and base. Local practice in Atlanta, GA dictates that side resistance of drilled shafts is to be ignored. This conservatism may be justified considering the lack of understanding of Piedmont residual soils, and the limited published research in the area. And it should also be noted that construction techniques can have an effect on side resistance, due to soils expanding or relaxing and possibly losing strength between excavation, casing removal, and concreting (Mayne and Harris, 1993).

Design methods

Mayne and Harris (1993) recommend a "hybrid α-β method" to calculate total shaft capacity. Side resistance is calculated using an effective stress method and tip resistance is determined using a total stress method. Drained conditions are assumed along the shaft since the shaft interface is believed to act as a drainage path. Undrained conditions are assumed to exist at the tip. Studies by Mayne and Harris (1993), Finke (1998), and Finke et al. (1999) have shown that pore pressures develop at the tip of shafts during loading, as compared to the sides of the shaft, where virtually no pore pressures develop.

O'Neill et al. (1996) suggested two amendments to Mayne and Harris' original method: (1) a reduction in friction angle if the shaft is excavated using a slurry; and (2) an upper limit on SPT N-values. Given that the method is not yet widely known, Appendix C is devoted to explaining the method in detail, and shows an example calculation.

The ADSC Drilled Shaft Manual by O'Neill and Reese (1999) proposes to use the hybrid α-β method, only for "intermediate geomaterials," defined as materials having SPT N-values greater than 50 blows/foot. They recommend that cohesionless materials with SPT N-values less than 50 blows/foot be analyzed using conventional methods for granular soils.

As previously mentioned, a literature review did not result in much published data on drilled shaft projects in the Piedmont region, despite fairly frequent use of drilled shafts in a number of major metropolitan areas. In fact, only five documented drilled shaft case histories, complete with subsurface profile and load test results, were found. An additional two case histories were found in unpublished data. Based on this limited amount of information, it is not possible at this time to discern which of the proposed design methods best models actual performance. Table 7 provides the load test results of the available case histories, along with the capacity calculations using the original hybrid α-β method (Mayne and Harris, 1993), the modified hybrid α-β method (by O'Neill et al., 1996), the ADSC method (1999), and two other well-known methods, the Meyerhof method (1976), and the Reese and Wright method (1977). Descriptions of the Meyerhof and the Reese and Wright methods can be found in Ooi et al. (1991). Details of the case histories as well as calculations for the capacities listed in Table 7 are provided in Appendix D.

Table 7. Comparison of drilled shaft case histories in the Piedmont region.

DRILLED SHAFT CASE HISTORY SUMMARY

	Calculated Capacities										Measured Capacity (at failure)		Interpretation of Measured Capacity
	Meyerhof method (1976)		Reese and Wright method (1977)		Original Mayne and Harris method (1993)		Modified Mayne and Harris method (1996)		ADSC method (1999)				
	Side Shear	End Bearing	Side Shear	End Bearing	Side Shear	End Bearing	Side Shear	End Bearing	Side Shear	End Bearing	Side Shear	End Bearing	
CASE #1 (Dia = 48", L = 45') Museum of Nature and Science, Raleigh, NC	132	988	388	503	506	395	506	356	579	356	>220	130	Writers interpreted failure at settlement = 5% of shaft dia.
CASE #2 ADSC/ASCE Test Site, Atlanta, GA													Failure of shaft C-1 is reported as max. load applied; Failure of shaft C-2 is the average of 9 estimated failure load criterion. See Appendix D for further explanations.
Shaft C-1 (Dia = 30", L = 72')	190	891	394	196	825	419	752	151	984	151	650	350	
Shaft C-2 (Dia = 30", L = 55')	65	106	191	118	296	65	296	65	514	106	300	50	
CASE #3 Georgia Tech, Atlanta, GA													Failure load reported at point at which a line drawn tangent to the initial section of the load-displacement curve intersects a line drawn tangent to the final portion of the curve. See Appendix D for comments on failure load reported for shafts 3 and 5.
Shaft 1 (Dia = 18", L = 15')	19	51	56	32	56	14	56	14	33	29	58	17	
Shaft 2 (Dia = 18", L = 15')	19	51	56	32	56	14	56	14	33	29	81	3	
Shaft 3 (Dia = 18", L = 22')	28	45	82	32	88	15	88	15	66	29	76	0	
Shaft 4 (Dia = 18", L = 22')	28	45	82	32	88	15	88	15	66	29	92	23	
Shaft 5 (Dia = 18", L = 20')	25	---	75	---	79	---	79	---	56	---	41	---	
Shaft 6 (Dia = 18", L = 20')	25	---	75	---	79	---	79	---	56	---	58	---	
CASE #4 (Dia = 36", L = 63') Coweta County, GA	198	985	385	283	944	397	455	193	733	193	320	130	Writers interpreted failure at settlement = 5% of shaft dia.
CASE #5 (Dia = 36", L = 13.5') Virginia Center, Vienna, VA	103	---	211	---	325	---	325	---	325	---	494	---	Failure reported at full mobilization of skin friction ('plunging' failure).
CASE #6 (Dia = 36/42", L = 62') Buncombe County, NC	293	657	665	283	1197	311	1010	198	970	198	>680	230	Writers interpreted failure at settlement = 5% of shaft dia.
CASE #7 (Dia = 60", L = 59') Springfield Interchange, VA	591	1329	1496	785	3016	1272	1965	555	1938	555	>215	475	Writers interpreted failure at settlement = 5% of shaft dia. See Appendix D for details.

Note all units are in tons.

Conclusions regarding design methods for drilled shafts:
The comparisons of computed and measured capacities summarized in Table 7 show that computed capacities are greater than the measured capacities in some cases, smaller in others. Based on the comparisons in Table 7, it is not possible to select a design procedure that is accurate or even reliably conservative for all conditions that may be encountered in the Piedmont. The writers recommend that designers review the detailed information in Appendix D, and employ design procedures consistent with their own evaluation of the evidence available.

One fact that emerges clearly from the data in Table 7 and Appendix D is that there is no case in which side shear is zero. Therefore, it can be firmly concluded that ignoring the contribution of side shear to the axial load capacities of drilled shafts is excessively conservative.

Other aspects of drilled shaft behavior:
The previous methods and case histories only address axial compression of drilled shafts in the Piedmont. Some uplift and lateral load tests have been performed on drilled shafts in the Piedmont, but information on these aspects of performance is limited. Table 5 lists the published cases found during this investigation.

Gardner (1987) suggested that load-deformation behavior can be modeled by elastic and load transfer analyses. Mayne and Harris (1993) concluded that elastic continuum methods as presented by Poulos and Davis (1980) and Randolph and Wroth (1979) are suitable for use in the Piedmont.

Excavation and construction:
As with general excavation work, defining excavation limits and preparing contract documents for drilled shaft projects is a challenge for the geotechnical engineer. Schwartz (1987) presents recommendations to minimize potential discrepancies during construction and litigation. Recommendations include proper investigation techniques, specification preparation, pre-qualification of contractors, and monitoring requirements for drilled shaft construction. Smith (2001) also discusses excavatability issues associated with drilled shafts in the Piedmont

O'Neill and Reese (1999) reviewed drilled shaft construction procedures in detail. The methods discussed are the dry, wet, and cased methods.

The dry method is applicable if the water table is below the depth of interest and the borehole will stand open for a period of time. A short piece of casing may be used near the surface if the surface soils are weak.

The wet method, or slurry-displacement method, uses prepared slurry to maintain borehole stability throughout the entire depth of excavation. It may be necessary to use the wet method when the borehole is unable to support itself, or when groundwater may cause stability problems. There are two procedures for removing the cuttings with the wet method: (1) the static process; and (2) the circulation process. In the static process, a drilling tool removes the cuttings, while in the circulation process, the cutting are brought to the surface by circulating the slurry.

Some experts believe that the circulation process produces a cleaner borehole than the static process, however conclusive evidence to support the claim was not uncovered in our literature review. In either case, upon completion of the excavation, the rebar is placed and tremied concrete displaces the slurry.

The cased method is employed if caving or excessive deformation is expected to occur during excavation, or if the water table is near the ground surface. The casing can be dropped, tapped, pushed, or vibrated into the ground. It is usually installed ahead of the excavation, but some rigs (full-depth casing rigs) excavate and install the casing simultaneously. As with a pile-driving job, driving the casing into place prior to excavation may densify the adjacent soils, possibly causing settlement of the surrounding ground.

As previously mentioned, it has been argued that the method by which the drilled shaft excavation is made (i.e. dry, wet, or cased) has an effect on the capacity of the drilled shaft. Majano et al. (1994) theorizes that shafts drilled by the wet and/or cased methods will have reduced capacity compared to those drilled by the dry method.

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