# Unexpected but foreseeable mat settlements on Piedmont residuum

**Paul W. Mayne,** *Professor, Georgia Institute of Technology, USA*

**ABSTRACT***: A large mat foundation was constructed to support a 13-story dormitory on Piedmont residual silty soils in Atlanta, Georgia. Prior to construction, the geotechnical consultant of record estimated maximum expected settlements of the mat on the order of 1.8 inches (46 mm), while the building proceeded to deflect as much as 10 inches (250 mm) at the center and 5 inches (127 mm) at the corners near the end of construction. Details on the case history are reviewed by an outside observer and placed within the context of geotechnical practice. In addition to routine soil borings, the use of enhanced in-situ testing (in this case, flat dilatometer tests) in concert with elastic continuum solutions would have provided calculated values in line with the observed performance.*

**KEYWORDS:** foundations, displacements, elasticity, in-situ tests, mats, rafts, settlements.

## INTRODUCTION

In preparation for the 1996 Summer Olympics, the construction of many office and residential buildings were underway in downtown Atlanta in order to accommodate a large number of visiting athletes, participants, and spectators. A site located south of the Olympic Village was to receive four mid-rise buildings that were eventually turned over to Georgia State University (GSU) for use as dormitories. The site was previously occupied by small one- and two-story residential and commercial buildings that were demolished prior to the new construction. Based on the results of soil test borings with standard penetration tests (SPT), the consultant recommended different systems for each of these buildings. Buildings A and D were supported on deep foundations, while Building C was placed on spread footings and Building B was situated on a large mat foundation. For the shallow foundations, unexpectedly large settlements were observed during construction with up to 5.5 inches (140 mm) for Building C and up to 10 inches (250 mm) for Building B. The latter project is discussed in this paper.

## PREFACE

From the onset, the author must clearly make note that he was not hired as a geotechnical consultant, nor as a professional engineer or as a forensic expert after the discovery of these issues. All of the subsequent evaluations and conclusions are made herein by the author as an outside observer. In some cases, the author found access to geotechnical data and reports directly relevant to the project and has used this information to describe and document this case history. Therefore, there are most likely additional details and data that exist, yet these have not been made available by those directly involved in the actual project or in later studies. The author is in a unique position to openly discuss this story because he has not had any official ties or role in the technical or legal aspects.

## GEOLOGY

The Piedmont geology is an important physiographic region along the eastern region of North America and includes major cities such as Atlanta, Baltimore, Charlotte, Columbia, Philadelphia, Raleigh, Richmond, Washington/DC, Wilmington, and Winston-Salem. The surficial exposure shows as a lenticular body approximately 1200 km long and 200 km wide that ranges from Alabama to Pennsylvania (Mayne, 1999). The southern portion of the Piedmont is shown in Figure 1. The Piedmont actually reaches much further eastward, yet underlies the vast and more recent Atlantic Coastal Plain. For example, in the Savannah GA area, the Piedmont is found some 800 to 1000 m below ground surface.

Mayne, P. W. (2005), *Unexpected but foreseeable mat settlements on Piedmont residuum* , International Journal of Geoengineering Case histories, http://casehistories.geoengineer.org , Vol.1, Issue 1, p.5-17.

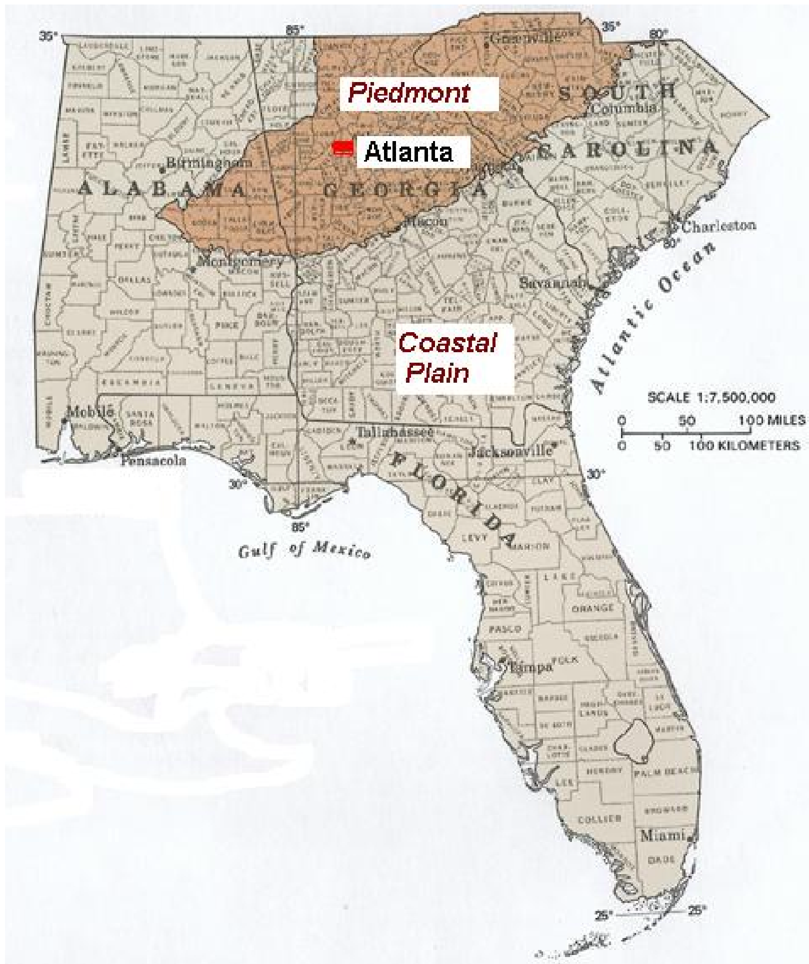


*Figure 1. Surficial exposure of the Southern Piedmont region*

The name Piedmont means "foot-of-the-mountains" and reflects remnants of an ancient mountain range that has since been extensively weathered, decomposed, and eroded to form rolling terrain and hillsides. The predominant metamorphic rocks are schist and gneiss of PreCambrian (Z-time) with granitic intrusives of the Paleozoic age (Chew, 1993) with localized occurrences of phyllite, quartzite, slate, greenstone, diabase, quartzite, and soapstone.

Variable weathering, temperature, drainage, and topography have reduced the rocks inplace to form overburden residual soils that range from thin silty clay topsoil to fine sandy silts and silty fine sands that grade with depth back into saprolite and partially-weathered rocks (Sowers & Richardson, 1983). The relict structure of the parent rock can be left within the derived soil materials with evidence of bonding or dissolved bond features, as well as cracks and fissures from the original fractured rock mass. The degree of weathering varies laterally and vertically, in such a way that rock may be encountered at depths of between 10 to 60 meters below ground surface, yet there are also common rock outcrops and occasional small remnant mountains up to 300 m high across the region (e.g., Kennesaw Mountain, Stone Mt). As a consequence, there is often concern in encountering intact rock along highway cuts, foundation excavations, and utility construction. Foundation systems can range from shallow footings and mats to driven piles or drilled and bored piles, depending upon the thickness and consistency of the overburden soils, saprolite, partially-weathered rock, and the overall depth to parent rock.

Residual soils transition into a saprolite and disintegrated or partially-weathered rock with depth until refusal is encountered at the parent rock interface (Martin, 1977; Sowers & Richardson, 1983), as depicted in Figure 2. The very shallow surface soils ($z < 1$ m) can weather severely to fine-grained sandy or silty clay, thus in the southern Piedmont, it has been called "Georgia red clay" by locals because of its agrarian use and red-tan color. Generally, the true clay fraction (< 0.002 mm) of Piedmont soils is often small (generally 5 to 20 %), thus "clay" is generally a misnomer.

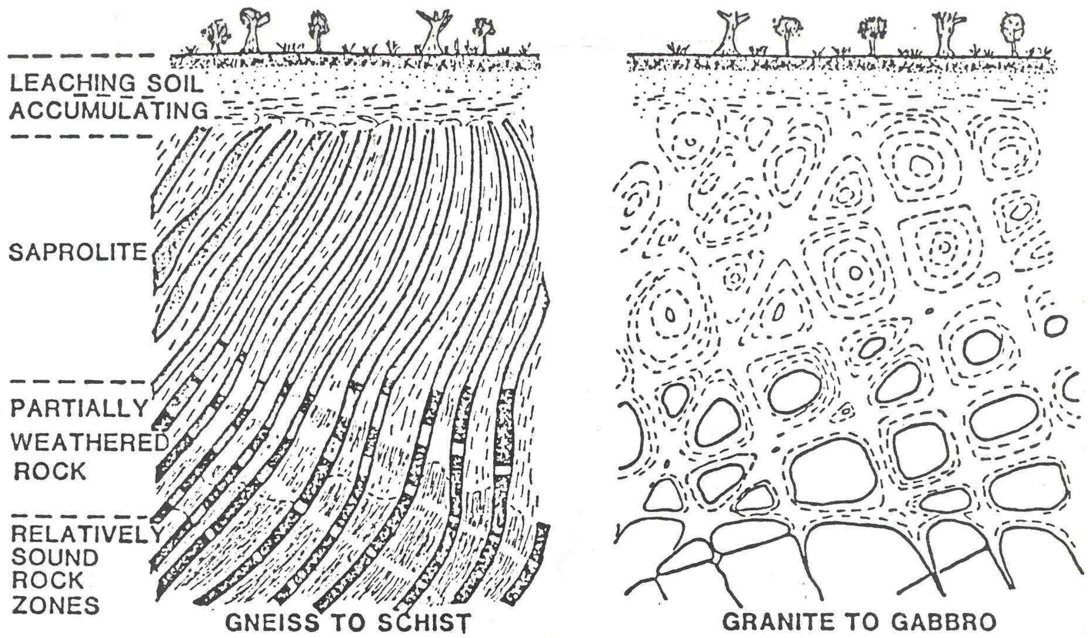


*Figure 2. Weathering profile in residuum derived from Piedmont metamorphic and igneous rock types (after Sowers & Richardson, 1983).*

The Piedmont residual soils are not particularly well-categorized by the Unified Soil Classification System (USCS) that was developed on the basis of extensive testing and empirical groupings of sedimentary soil deposits. With the USCS, a vertical profile in the Piedmont seems variable with alternating random strata of silty fine sands (SM) and fine sandy silts (ML). The material is actually somewhat uniform, because the mean grain size is close to the 0.075-mm criterion (No. 200 sieve size) used to define the percent fines (PF) content. In fact, the Piedmont residuum acts more as a dual soil type (SM-ML), exhibiting aspects of both fine-grained soils (undrained) when loading is fast, and features of granular soils (drained) when subject to slower rates of loading (Mayne et al. 2000).

Because of their unique aspects, a number of research programs have documented the characteristics and performance of Piedmont residuum as "non-textbook" type geomaterials (e.g., Borden, et al. 1996; Finke, et al. 2001; Hoyos & Macari, 1999; Mayne & Harris, 1993; St. John, et al. 1969; Schneider et al. 1999; Sowers 1994).

## BUILDING AND SITE DETAILS

The dormitory buildings were constructed on a previously occupied site located at the southwest quadrangle at the intersection of Route 29 (North Avenue) with combined interstates I-75 and I-85 ("The Connector") in downtown Atlanta. Figure 3 shows the site location of the development. Building Dorm B was situated at the southeastern corner of the property and underlain by approximately 2 to 5 m of fill and alluvial soils that were also derived from the ubiquitous residual silts and sands of the Piedmont geology in the Atlanta area. A small surcharge fill of about 5 m high was placed at the building area in an attempt to preload the soils, however, the imposed stresses amounted to only one-half to two-thirds of the eventual building loading, thus reflecting activities along the soil recompression-swelling line in the conventional consolidation framework.

Dormitory B was constructed on a reinforced concrete mat foundation having a breadth of 104 m (340 feet), width of 18 m (60 feet), and thickness of 1.1 m (3.5 feet). Designs were based on a combined total live and dead loading of approximately 200 kPa (2 tsf), although the settlements noted herein are based on the actual dead load plus reduced live loading evaluated to be about 150 kPa (1.5 tsf). Construction of the 13-story structure commenced and took about 1 year for completion. The completed structure is shown in Figure 4. During the final stages of construction, the contractor had finished placing the brick facing and noticed that the lowest floors exhibited a concave camber in the bricks, indicative of settlement. A resurvey across the top of the mat area by controlled elevation measurements showed considerable deflections and prompted the hiring of an independent engineering consulting firm to re-evaluate the situation.

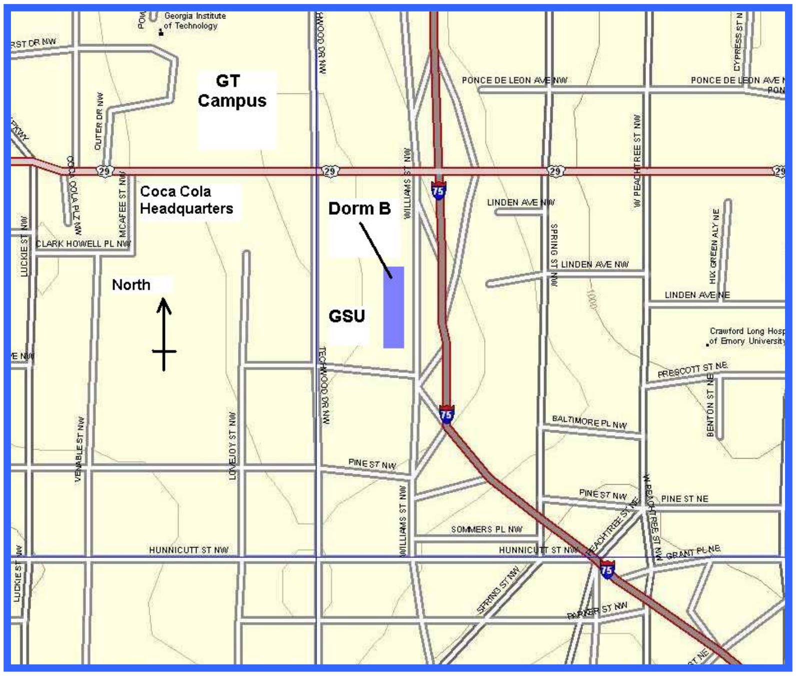


*Figure 3. Site Location of Dormitory Building B, Atlanta, GA*

*Figure 4. Completed Dormitory Building B in GSU Complex*

In addition to conducting a review of the approximate 20 soil boring records that were available, the independent engineering firm conducted flat plate dilatometer tests (DMT) in general accordance with ASTM D 6635 guidelines (Marchetti, 1980; 1997; Lutenegger, 1988). In addition, two undisturbed shelby tube samples were procured and used in one-dimensional consolidation tests. Notably, oedometer test results in Piedmont soils are rather difficult to interpret, however, because of stress relief, sample disturbance effects, pronounced curvature, and the lack of a clear definition of yield (e.g., Wesley, 1994; Mayne & Harris, 1993).

## SUBSURFACE CONDITIONS

Soil borings confirmed the presence of shallow fill and alluvial soils overlying natural residual fine sandy silts (ML) and silty fine sands (SM) with an ambient groundwater table approximately 6 to 7 meters (20 to 23 feet) deep. These materials exhibited N-values in the range of 5 to 25 blows per 0.3 meters, generally increasing with depth, as shown in Figure 5. A thin zone of partially weathered rock (PWR) exists at the site (generally 1 to 3 meters thick) and SPT refusal was generally encountered at depths ranging from 12 m (40 feet) to 24 m (80 feet) below existing grade. Presumably, the SPT refusal reflects the top of unweathered Piedmont rocks. At a nearby Georgia Tech site northwest of this property, rock coring in five borings confirmed the presence of granitic gneiss bedrock with 38 < RQDs < 45 percent (Mayne & Harris, 1993).

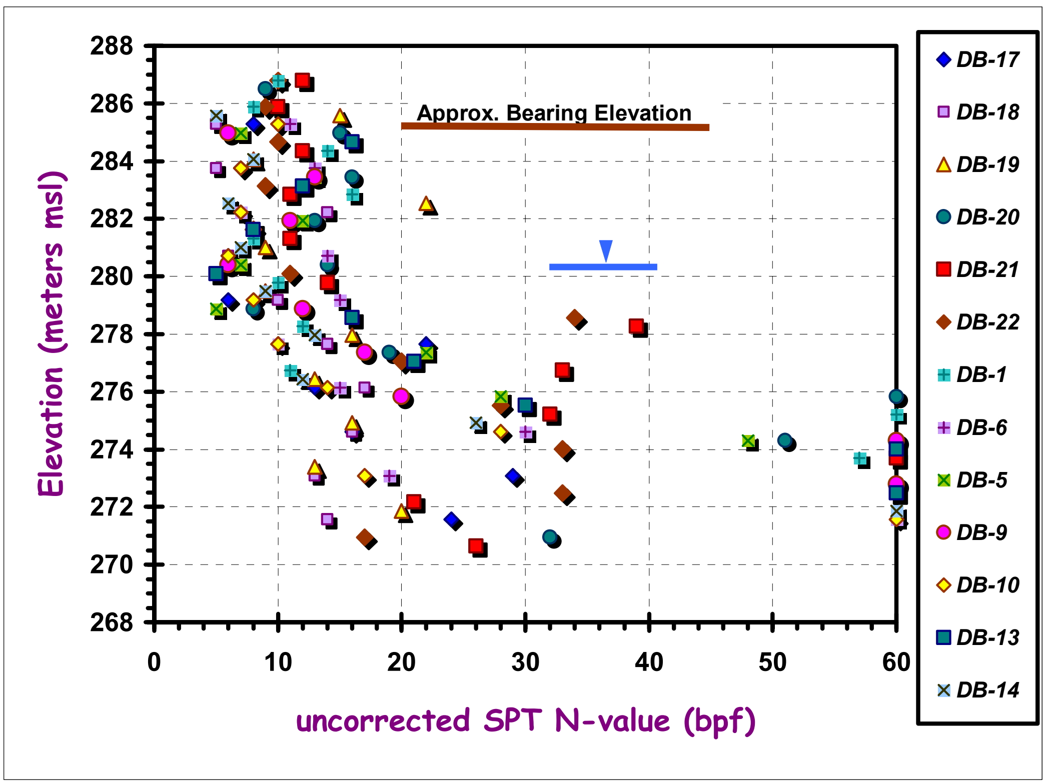


*Figure 5. Summary of SPT Resistances at GSU Complex, Atlanta, Georgia.*

For Building Dorm B, it appears that not all borings on the site were taken into the PWR or refusal, but perhaps were terminated at pre-selected test depths. In light of budget considerations, it is possible many of the borings extended only to 12 m depths (40 feet) in order to save costs. Calculated deflections based on elastic continuum theory depend strongly on the thickness of the compressible layer (i.e., depth to PWR or rock), as well as the dimensions of the mat foundation, and applied level of loading (Poulos & Davis, 1974).

Results of flat dilatometer tests (DMT) are useful for settlement computations involving shallow foundations in soil types ranging from sand to silt to clay (Marchetti, et al., 2001). The DMT obtains measurements of contact pressure ($p_0$) and expansion pressure ($p_1$) that are processed to derive three parameters: material index ($I_D$), dilatometer modulus ($E_D$), and horizontal stress index ($K_D$). Details on the calculations of these values are given elsewhere (e.g., Marchetti, 1980;

Lutenegger, 1988). Figure 6 shows the profile of one of two DMTs conducted at the GSU site by the independent consultant. The results are comparable to DMT data from both the nearby Georgia Tech research site (Harris & Mayne, 1994) and the Opelika National Geotechnical Experimentation Site (NGES) off I-85 in Alabama (Brown & Vinson, 1998; Mayne, et al. 2000).

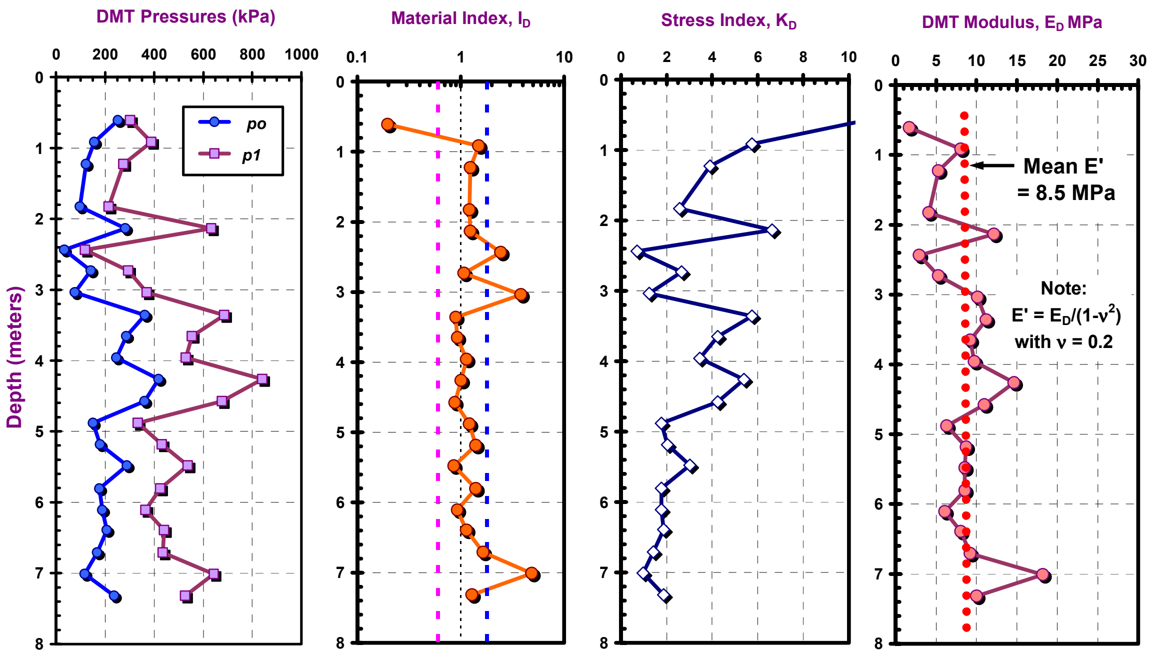


*Figure 6. Results of DMT Sounding at GSU Complex, Atlanta, GA.*

The material index $I_D$ correctly falls within the category for silty soils, with some variation into the silty sands and sandy silt regimes. If a simple homogeneous layer (modulus constant with depth) is adopted, then the mean value of dilatometer modulus averages $E_D$ = 8.5 MPa (85 tsf). Note that since $E' = E_D/(1-\nu^2)$ was derived from elastic theory (Marchetti, 1980), in fact for the Piedmont with an adopted value of Poisson ratio $\nu = 0.2$, a first approximation is that $E' \approx E_D$. Beneath a 2-m crust, the Opelika NGES site exhibited a homogeneous condition (Brown & Vinson, 1998; Martin & Mayne, 1998), yet the Georgia Tech site trend was more indicative of a Gibson-type soil with modulus increasing linearly with depth (Harris & Mayne, 1994). A Piedmont site in North Carolina showed a DMT profile somewhat intermediate to these cases (Wang & Borden, 1996).

## MEASURED SETTLEMENTS

The measured settlements of the Dorm B mat are shown in contour plots presented in Figure 7. A maximum settlement of 250 mm (9.8 inches) was observed near the center and the four corners showed settlements of between 100 mm (4.0 inches) and 140 mm (5.5 inches). The fact that the cornerpoint deflections are approximately one-half the centerpoint value is indicative of a flexible foundation (e.g., Fraser & Wardle, 1976). An examination of the structure by various engineering firms revealed no imminent concerns for collapse or serious distress in the building. However, long-term maintenance of utilities, doors, elevators, windows, and architectural features (e.g., brick facing replacement, concrete cracking, glass and mortar repair) was anticipated during occupation and utilization of the building in future years.

The official excuse offered by the geotechnical firm of record was that data were incorrectly inputted into a computer program by an entry-level engineer. The engineer intended to model the subsurface situation as twelve layers each 5 feet (1.5 m) to represent the soil to bedrock conditions (total depth H = 60 feet), but apparently not fully understanding the program, entered 12 layers of 1-foot each (0.3 m). This would explain the calculated displacements off by a factor of approximately fivefold (5 times 1.8 inches = 9 inches).

An alternate story told to the author is that the original firm used a subgrade reaction method to evaluate the mat foundation displacements, misinterpreting these calculated deflections as "settlements". In this scenario, empirical correlations that were utilized for assessing the moduli of subgrade reaction were not adjusted for the large mat size, which can be very significant (e.g., Horvath, 1983). One unsubstantiated story is that the original geotechnical firm used a correlation between raw measured N-value and subgrade modulus ($k_s$) that was intended for wheel loading on pavements (i.e., NAVFAC DM-7 (1982). Also, without energy measurements taken during the SPT or in a requisite calibration of the drill rig and crew, the energy-corrected $N_{60}$ values were not known, thus the penetration resistances are not necessarily standardized to values used in practice (e.g., Skempton, 1986).

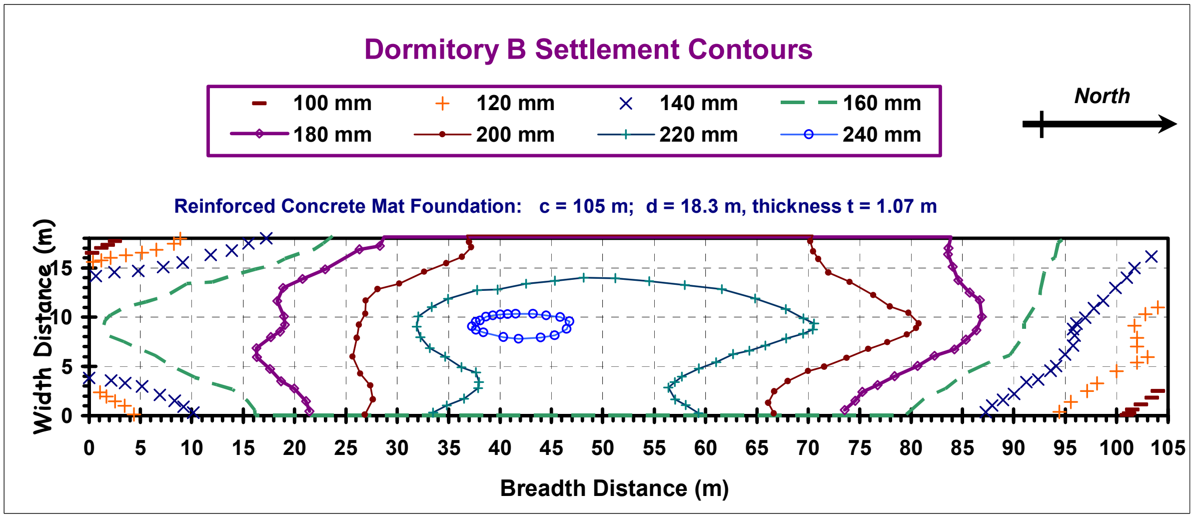


*Figure 7. Measured Settlement Contours of the Dormitory Mat Foundation.*

Perhaps in certain circumstances involving mats with simple loading and subsurface conditions, a subgrade reaction approach can be considered for displacement calculations related to primary settlements. In general, the subgrade reaction approach is more useful for evaluating relative bending deflections across the mat, thus for assessing the amount of needed steel reinforcement based on moment diagrams during floor by floor construction. Calculations of total displacements due to drained primary consolidation are best handled by either the classical evaluation of elastic stress distributions beneath surface foundations coupled with e-log$\sigma_v$' data, or elasticity theory using displacement influence factors and appropriate moduli.

In any event, with either scenario 1 or 2, the full judgment, wisdom, and oversight of a senior engineer appears to have been missing from the "Class A" type predictions of settlements that were issued in the original geotechnical report.

## CLASS "C" SETTLEMENT CALCULATIONS

The residuum of the Piedmont generally consolidates fairly rapidly (Finke & Mayne, 1999; Finke, et al. 1999, 2001; Mayne & Brown, 2002). Therefore, the measured settlements of Building Dorm B may be attributed to those of drained primary consolidation. While geotechnical practice in the USA often relies on the empirical e-log$\sigma_v$' graphs for the analysis of primary consolidation settlements, these data are from one-dimensional oedometer tests and the deflections of full-scale foundations are, in fact, three dimensional. The traditional e-log $\sigma_v$' graphs are actually a subset and synonymous with elastic continuum solutions (Fellenius, 1996). If the consolidation test data are plotted in terms of applied stress ($\sigma_v$') vs. vertical strain [$\varepsilon_v = \Delta e/(1+e_0)$, the slope is the constrained modulus ($D' = \Delta\sigma_v'/\Delta\varepsilon_v$).

From elasticity theory, the inter-relationship between constrained modulus (D') and elastic Young's modulus (E') is simply:

$$D' = E' \cdot \frac{(1-\upsilon')}{(1+\upsilon')(1-2\upsilon')} \tag{1}$$

For realistic values of Poisson's ratio, the values are quite similar. In fact, D' = E' at $\nu'=0$. Using local internal strain measurements, recent research has shown the operational value of Poisson's ratio is generally quite small at working loads levels such that $0.1 \leq \nu' \leq 0.2$ (e.g., Tatsuoka, et al. 1994; LoPresti, et al. 1995), and consequently D' ranges from 1.05 E' to 1.11 E'.

In terms of displacement influence factors, the form of the equation for centerpoint settlement ($\rho_c$) beneath a shallow foundation using elastic continuum theory is:

$$\rho_c = \frac{q \cdot d \cdot I_h \cdot (1-\nu^2)}{E_{s0}} \tag{2}$$

where q = applied foundation stress, d = foundation width, $E_{s0}$ = equivalent elastic soil modulus beneath the foundation base, and $I_h$ = displacement influence factor that accounts for the simplified subsurface geometry, layering, foundation rigidity, layer thickness, and soil stiffness variation with depth.

Using the numerical procedure described by Mayne & Poulos (1999), it is a simple matter to obtain the values of $I_h$ for a rectangular foundation resting on a homogeneous medium of finite thickness (h), assuming a smooth soil-rock interface. For a rectangle of long dimension c (length) and small dimension d (width), the vertical stress change at the centerpoint is given by (Harr, 1966; Das 2002):

$$\Delta\sigma_z = \frac{2}{\pi}\frac{mn}{\sqrt{1+m^2+n^2}}\frac{1+m^2+2n^2}{(1+n^2)(m^2+n^2)} + \frac{2}{\pi}\sin^{-1}\left(\frac{m}{\sqrt{m^2+n^2}\sqrt{1+n^2}}\right) \tag{3}$$

$$where \quad m = c/d \quad and \quad n = 2z/d$$

The integration of the equivalent uniaxial strains with depth gives the solutions presented in Figure 8. These compare well with the published select values given in tables by Harr (1966). It can be taken that the effects of embedment and foundation rigidity for this case study are relatively small (Mayne & Poulos, 1999). As a first approximation, the cornerpoint settlements for a flexible rectangular foundation under uniform loading are one-half the centerpoint values (Poulos & Davis, 1974). Additional considerations and complexities on the effects of relative foundation flexibility and rigidity for rectangular foundations can be found in Fraser & Wardle (1976) and Horikoshi & Randolph (1997).

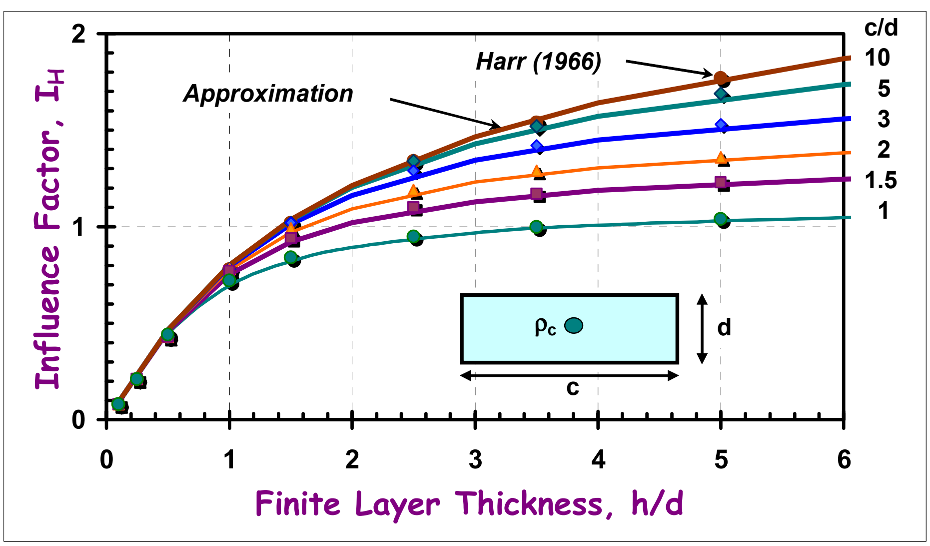


*Figure 8. Displacement Influence Factors for Flexible Rectangular Foundation Resting on Finite Homogeneous Layer and Smooth Subsurface Interface ($\nu$ = 0). Approximate numerical integration solution (after Mayne & Poulos, 1999) compared with closed-form elastic solution (Harr 1966)*

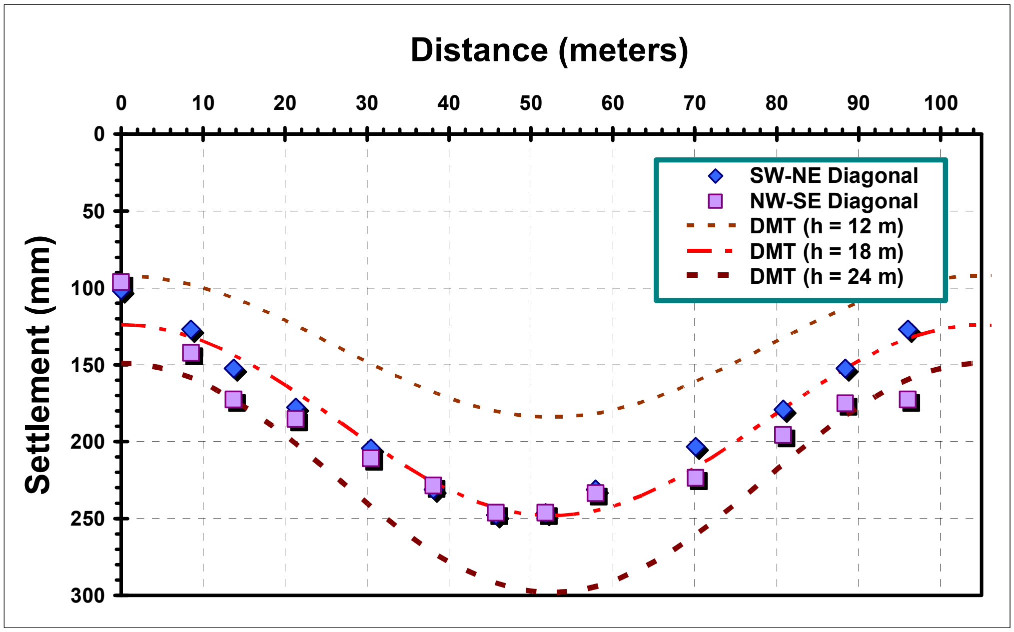


*Figure 9. DMT-Calculated versus Measured Settlements for Dorm B Mat Foundation over Selected Finite Compressible Layer Thicknesses. Note: Homogeneous soil modulus $E_s$ = 8.5 MPa and q = 150 kPa.*

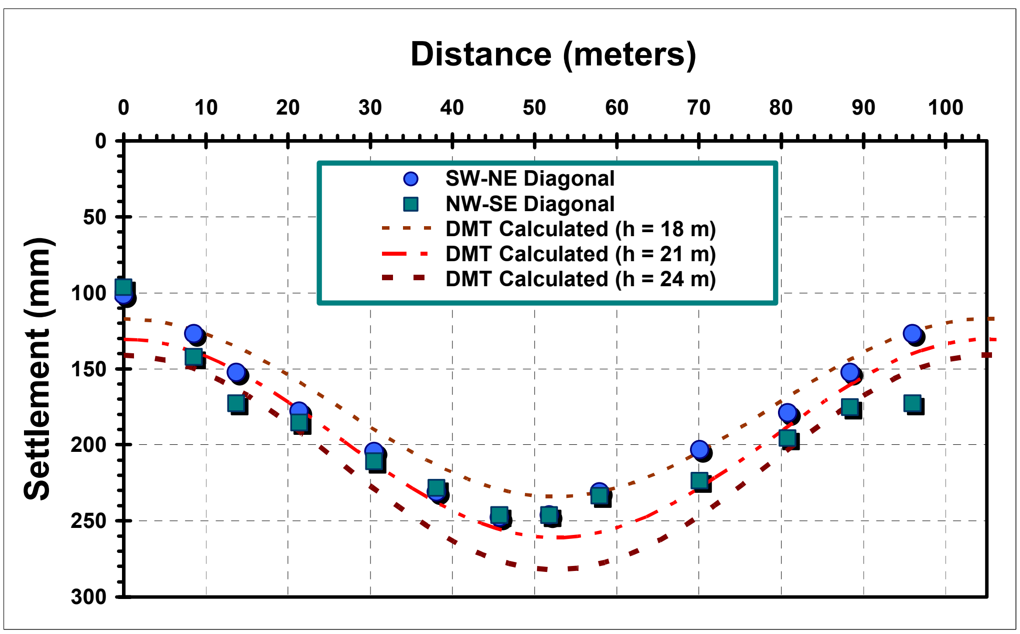


*Figure 10. Calculated versus Measured Settlements Using DMT Constrained Modulus Values and Finite Layer Thicknesses. Note: Applied stress q = 200 kPa.*

The calculated settlements depend upon the actual depth to rock and/or partially-weathered rock, thus Figure 9 shows the influence of variable thicknesses of the compressible residual soil mantle. A homogeneous stiffness of $E_s' = E_{DMT}$ = 8.5 MPa (85 tsf) was adopted for all calculations. A Poisson's ratio ν' = 0.2 was taken representative of drained conditions (Mayne & Poulos, 1999). Relatively good agreement is observed for the case where h = 18 m (60 feet) is taken representative as the depth to PWR/rock and a perfectly flexible mat (q = 150 kPa) considered in the analysis. Alternative interpretations could consider the trend of SPT data (Figure 5) indicating a Gibson-type profile of stiffness with depth and a finite layer h = 12 m. Apparently, many of the soil boring records stopped before encountering either auger or SPT refusal. In addition, the applied mat loading may be as high as q = 200 kPa, as noted earlier. Regardless of the various specific assumptions made herein, a centerpoint deflection of between 200 mm to 300 mm should have been anticipated.


*Figure 11. View of Dorm B looking west from combined interstates I-75/I-85.*

**DISCUSSION**

A number of flaws are notable in the original geotechnical study that should have been caught during a senior review. In the author's opinion, these include:

1. Assignment of pre-set boring depths, without review of the actual conditions encountered. The large mat size required borings to be terminated in rock or partially-weathered rock in order to properly define the thickness of the compressible soil layer. Many borings in the Dorm B area (particularly the eastern side) did not extend sufficiently deep to fully define the actual thickness of the residuum over rock.
2. Over-reliance on SPT-N values for use in settlement computations. The manner and method of conducting the SPTs was not noted in the reports and documents that were available to the author, however, it is well-known that the SPT suffers extensively from energy inefficiency during operations (e.g., Butler, et al. 1998). Even in the case of automatic hammers, the different systems (Mobile, Acker, CME, Failing, Dietrich, Layne) give measured efficiencies of between 60 to 100%. In the case of donut and safety hammers, the efficiencies can be from 30 to 88%.
3. The application of a preload in the silty fine sands and fine sandy silts of the Piedmont that exhibit such quick drainage may seem fruitless, yet the applied surcharge loading was only 100± kPa and was well below the applied building mat stress of 150 to 200 kPa, and was thus of no real value. Moreover, the load applications resulted in stresses that were likely less than the natural preconsolidation stress, since overconsolidation ratios (OCRs) due to groundwater fluctuations, desiccation, and aging effects are generally between 1.5 to 4.0 in such an old formation (Mayne & Brown, 2003). In fact, using the Marchetti (1980) correlations, the DMT data infer a range of OCRs from 2 to 3 at this site.
4. Use of consolidation tests for settlement computations in the Piedmont geology is unreliable because of sample disturbance, possible swelling due to mica, and the high fine sand content (Mayne & Harris, 1993; Wesley, 1994), as well as the presence of stones and sampling problems (Martin, 1977) and partial saturation (Wang & Borden, 1996). Although results from only two tests are known to the author, additional results may exist, yet may be of only minor value. The two tests reviewed had no unload-reload cycle, thus the derived D' of 2.5 and 7.5 MPa are probably low.
5. While the use of subgrade modulus for evaluating the relative mat deflections, contortions, and moment distributions is reasonable, the calculation of foundation settlements due to primary consolidation based on subgrade reaction modulus is not fundamentally-based.
6. The mix of spread footings, deep foundations, and structural mats for support of four buildings in close proximity to the GSU complex should have given rise for concern in expected foundation performance, in that all four buildings are of

comparable size in height and footprint area. Building C (not discussed here) was situated on spread footings that settled as much as 140 mm (5.5 inches), which is quite beyond the normal performance mark used in design (e.g., Wahls, 1994).

7. The use of a computer program should be checked by simple hand calculations.

In the Piedmont geology, the dilatometer modulus $E_D$ has been shown useful when input directly into elastic continuum solutions for evaluating the magnitude of foundation settlements due to primary consolidation (Mayne & Frost, 1988; Mayne et al. 1999). This is similar yet different than the procedure recommended using a DMT constrained modulus approach (e.g., Schmertmann, 1986), where: $D' = M' = R_M*E_D$ in that the empirical factor $R_M$ is a function of both $I_D$ and $K_D$ with an established range $0.85 < R_M < 2.5$, thus generally increasing the magnitude of the elastic modulus $E_D$ (Marchetti, et al. 2001). Both methods use elastic stress distributions (i.e. Boussinesq theory), yet the displacement influence approach is able to account for foundation rigidity, whereas the DMT-M' approach does not actually address this difference and likely the solutions for flexible uniform loading are used in practice, even for rigid footings. As the use of consolidation test data coupled with Boussinesq theory often overpredicts foundation settlements in the Piedmont, it has been common to adopt a Westergaard elastic solution to reduce the imposed stress magnitudes, yet this too results in overcomputed settlements (Willmer, et al., 1982; Barksdale, et al., 1982). Since the $K_D$ is between 2 to 6 (mean = 3.3) at the GSU site, the derived $R_M$ for the Piedmont sandy silts at this site averages only about 1.3. The deflections have been recalculated using a representative D' = M' = 12 MPa and q = 200 kPa for the mat, as shown in Figure 10. Here, a representative depth to rock of 20 m gives the best fit to the measured settlements. The alternative M' method would have still warned the designers of excessive displacements, if only it had been used.

## CONCLUSIONS

A case study involving a mat foundation on Piedmont residual soils is presented to illustrate large centerpoint settlements of up to 250 mm (9.8 inches) that were unexpected by the geotechnical firm in design who had calculated approximately 46 mm (1.8 in.) maximum deflections. Simple elastic continuum solutions with input moduli derived from flat dilatometer tests and finite layer thicknesses showed excellent agreement with measured settlement contours. An over-reliance on SPT-N data from soil borings, coupled with the poor choice of a subgrade reaction model for analysis and other bad judgments, resulted in settlement calculations that were very much incorrect. In the author's opinion, the building should have been supported on deep foundations consisting of either drilled shafts or a mat resting on a group of augered pilings.

## REFERENCES


Barksdale, R.D., Bachus, R.C., and Calnan, M.B. (1982). Settlements of a tower on residual soil. Engineering & Construction in Tropical & Residual Soils. ASCE Conference Proceedings, Honolulu, 647-664.

Borden, R.H., Shao, L, & Gupta, A. (1996). Dynamic properties of Piedmont residual soils. ASCE Journal of Geotechnical & Geoenvironmental Engineering 122 (10), 813-821.

Brown, D.A. & Vinson, J. (1998). Comparison of strength and stiffness parameters for a Piedmont residual soil, Geotechnical Site Characterization, Vol. 2, Balkema, Rotterdam, 1229-1234.

Butler, J.J., Caliendo, J.A., and Goble, G.G. (1998). Comparison of SPT energy measurement methods. Geotechnical Site Characterization, Vol. 2, Balkema, Rotterdam, 901-905.

Chew, V.C. (1993, 1988). Underfoot: a geologic guide to the Appalachian Trail, Second Edition, Appalachian Trail Conference, Harpers Ferry, 237 p.

Das, B.M. (2002). Principles of Geotechnical Engineering: Fifth Edition, Brooks - Cole, Thomson Learning Inc., Pacific Grove, CA., 248-250.

Fellenius, B.H. (1996). Basics of Foundation Design: A Geotechnical Textbook. BiTech Publishers, Richmond, B.C., 134 pages.

Finke, K.A. & Mayne, P.W. (1999). Piezocone tests in US Atlantic Piedmont residual soils. Proceedings, XI Pan American Conference on Soil Mechanics & Geotechnical Engineering, Vol. 1, Foz do Iguassu, Brazil, 329-334.

Finke, K.A., Mayne, P.W., & Klopp, R.A. (1999). Characteristic piezocone response in Piedmont residual soils. Behavioral Characteristics of Residual Soils, GSP No. 92, ASCE, Reston, VA, 1-11.

Finke, K.A., Mayne, P.W. & Klopp, R.A. (2001). Piezocone penetration testing in Atlantic Piedmont residuum. ASCE Journal of Geotechnical & Geoenvironmental Engineering 127 (1), 48-54.

Fraser, R.A. and Wardle, L.J. (1976). Numerical analysis of rectangular rafts on layered foundations. Geotechnique 26 (4), 613-630.

Harr, M.E. (1966). Foundations of Theoretical Soil Mechanics. McGraw-Hill, New York, 77-78.
Harris, D.E., & Mayne, P.W. (1994). Axial compression behavior of two drilled shafts in Piedmont residual soils, Proceedings, International Conference on Design & Construction of Deep Foundations, Vol. 2, Federal Highway Administration, Washington, D.C., 352-367.
Horikoshi, K. and Randolph, M.F. (1997). On the definition of raft-soil stiffness ratio for rectangular rafts. Geotechnique 47 (5), 1055-1061.
Horvath, J.S. (1983). Modulus of subgrade reaction: new perspective. Journal of Geotechnical Engineering 109 (12), 1591-1596.
Hoyos, L.R., Jr. & Macari, E.J. (1999). Influence of in-situ factors on dynamic response of Piedmont residual soils. ASCE Journal of Geotechnical and Geo-environmental Engineering 125 (4), 271-279.
LoPresti, D.C.F., Pallara.O. and Puci, I. (1995). A modified commercial triaxial testing system for small strain measurements. ASTM Geotechnical Testing J. 18 (1), 15-31.
Lutenegger, A.J. (1988). Current status of the Marchetti dilatometer test. Penetration Testing 1988, Vol. 1, Balkema, Rotterdam, 137-155.
Marchetti, S. (1980). In-situ tests by flat dilatometer. ASCE Journal of Geotechnical Engineering 107 (GT3), 832-837.
Marchetti, S., Monaco, P., Totani, G. & Calabrese, M. (2001). The flat dilatometer test (DMT) in soil investigations. A report by the ISSMGE Committee TC 16. Proceedings, International Conference on In-Situ Measurement of Soil Properties & Case Histories, Bali, Indonesia, 95-131.
Martin, G.K. & Mayne, P.W. (1998). Seismic flat dilatometer tests in Piedmont residual soils, Geotechnical Site Characterization, Vol. 2, Balkema Rotterdam, 837-843.
Martin, R.E. (1977). Estimating foundation settlements in residual soils, ASCE Journal of the Geotechnical Engineering Division (ASCE) 103 (GT3), 197-212.
Mayne, P.W. (1999). Site characterization aspects of Piedmont residual soils in eastern US. Proceedings, 14th International Conference on Soil Mechanics & Foundation Engineering, Vol. 4, Balkema, Rotterdam, 2191-2195.
Mayne, P.W. and Brown, D.A. (2003). Site characterization of Piedmont residuum in North America. Characterization & Engineering Properties of Natural Soils, Vol. 2, Singapore Workshop Proceedings, Balkema Publishers, Rotterdam, 1323-1339.
Mayne, P.W., Brown, D.A., Vinson, J., Schneider, J.A. and Finke, K.A. (2000). Site characterization of Piedmont residual soils at Opelika, Alabama. National Geotechnical Experimentation Sites, GSP 93, ASCE, Reston, Virginia, 160-185.
Mayne, P.W. & Frost, D.D. (1988). Dilatometer experience in Washington, D.C. Transportation Research Record 1169, National Academy Press, Washington, D.C., 16-23.
Mayne, P.W. & Harris, D.E. (1993). Axial load-displacement behavior of drilled shaft foundations in Piedmont residuum. Research Report No. E-20-X19 to Federal Highway Administration prepared by Georgia Tech Research Corporation, Atlanta, 162 pages (available from the Association of Drilled Shaft Contractors, Dallas).
Mayne, P.W., Martin, G.K. and Schneider, J.A. (1999). Flat dilatometer modulus applied to drilled shaft foundations in Piedmont residuum. Behavioral Characteristics of Residual Soils (GSP 92), ASCE, Reston/VA, 101-112.
Mayne, P.W. and Poulos, H.G. (1999). Approximate displacement influence factors for elastic shallow foundations. ASCE Journal of Geotechnical & Geoenvironmental Engineering 125 (6), 453-460.
Naval Facilities Engineering Command (NAVFAC, 1982). Design Manual: Soil Mechanics, Foundations, & Earth Structures. Manual DM-7, Department of the U.S. Navy, Alexandria, Virginia.
Poulos, H.G. & Davis, E.H. (1974). Elastic Solutions for Soil & Rock Mechanics, Wiley & Sons, New York. Now available from Univ. of Sydney Press.
Schmertmann, J.H. (1986). Dilatometer to compute foundation settlement. Use of In-Situ Tests in Geotechnical Engineering, GSP 6, ASCE, Reston/VA, 303-321.
Schneider, J.A., Hoyos, L., Mayne, P.W., Macari, E.J., & Rix, G.J. (1999). Field and lab measurements of dynamic shear modulus of Piedmont residual soils. Behavioral Characteristics of Residual Soils (GSP 92), ASCE, Reston, 12-25.
Skempton, A.W. (1986). Standard penetration test procedures and the effects in sands of overburden stress, relative density, particle size, ageing, and overconsolidation. Geotechnique 36 (3), 425-447.
Sowers, G.F. (1994). Residual soil settlement related to the weathering profile. Vertical and Horizontal Deformations of Foundations and Embankments, GSP No. 40, Vol. 2, ASCE, Reston, Virginia, 1689-1702.
Sowers, G.F. & Richardson, T.L. (1985). Residual soils of the Piedmont and Blue Ridge, Transportation Research Record No. 919, National Academy Press, Washington, D.C., 10-16.
St. John, B.J., Sowers, G.F. & Weaver, C.E. (1969). Proceedings, 7th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, Mexico, City, 591-597.

Tatsuoka, F., Teachavorasinskun, S., Dong, J., Kohata, Y. and Sato, T. (1994). Importance of measuring local strains in cyclic triaxial tests on granular materials. Dynamic Geotechnical Testing II, STP 1213, ASTM, West Conshohocken/PA, 288-302.

Wahls, H.E. (1994). Tolerable deformations. Vertical & Horizontal Deformations of Foundations & Embankments, Vol. 2, GSP 40, ASCE, Reston/VA, 1611-1628.

Wang, C.E. & Borden, R.H. (1996). Deformation characteristics of Piedmont residual soils. ASCE Journal of Geotechnical Engineering 122 (10), 822-830.

Wesley, L.D. (1994). The use of consolidometer tests to estimate settlement in residual soil. Proceedings, 13th International Conference on Soil Mechanics & Foundation Engineering, Vol. 2, New Delhi, 929-934.

Willmer, J.L., Futrell, G.E., and Langfelder, J. (1982). Settlement predictions in Piedmont residual soil. Engineering & Construction in Tropical & Residual Soils. ASCE Conference, Honolulu, 629-646.

## The International Journal of Geoengineering Case Histories (IJGCH) is funded by:

Email us at journal@geoengineer.org if your company wishes to fund the International Journal of Geoengineering Case Histories.