Reducing Settlement Risks In Residual Piedmont Soils Using Rammed Aggregate Pier Elements

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ABSTRACT

Residual Piedmont soils are difficult for geotechnical engineers. They exhibit large vertical and horizontal variability that is attributable to variations in parent rocks and uneven weathering. This variability results in differential settlements below shallow spread footings. *Rammed Aggregate Pier™* elements are used to increase the composite stiffness of Piedmont soils and are tailored to accommodate soil variability. The results of 31 load tests performed on aggregate pier elements installed in native and fill soils of the Piedmont province are presented. Test results indicate that bulging is seldom of concern and that the response of the piers depends on the ability of the matrix soil to resist shear stresses along the pier perimeters and, that after the full perimeter shear resistance is mobilized, compressive stresses are observed at the bottoms of the elements. Aggregate pier elastic modulus values vary between 115 MPa (1,200 tsf) and 270 MPa (2,800 tsf) with stiffness ratios of piers relative to matrix soils ranging between 5 and 60. The increase in stiffness and reduction in variability of foundation soils reinforced with aggregate piers allows for control of foundation settlement and reduces the risk of excessive differential movements.

INTRODUCTION

The residual soils of the Piedmont physiographic province are challenging to geotechnical engineers because of the difficulty defining soil, rock, and transition

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zones in the subsurface profile and because of the variable engineering properties of these materials. For design of foundations in Piedmont soils, current geotechnical practice relies on in-situ tests correlated to engineering parameter values. Regardless of the methods used to characterize the engineering properties, the highly variable nature of residual soil materials presents risks for both design and construction. Whether for shallow or deep foundations, suitable bearing surfaces are difficult to define, as is the construction effort required to reach bearing elevations. A cost-effective solution is needed to increase performance by providing consistent and verifiable behavior with a low risk for budget overruns.

In recent years, *Rammed Aggregate Pier* elements have been increasingly used as a cost-effective solution to accommodate soil variability and to increase the overall strength and stiffness of residual Piedmont soils. The piers are designed using classical geotechnical engineering principles. They are used as part of a stabilized mass of soil with relatively well-defined engineering properties. Aggregate piers are presently being used to support footings ranging in column load from less than 200 kN (50 kips) to as large as 13,300 kN (3,000 kips). The piers are also being used to reinforce soils for landslide control, embankment support and retaining wall support.

This paper presents the results of 31 load tests performed for aggregate piers constructed in the Piedmont province. Many of the tests were performed with basal telltales in order that settlement of both the tops and bottoms of the piers could be monitored. The piers were constructed in native soils and in fill soils of Piedmont origin. This work is of particular significance because it provides a practical and comprehensive means to establish design parameter values used for a rapidly growing soil reinforcement method in the Piedmont province

BACKGROUND – PIEDMONT PHYSIOGRAPHIC PROVINCE

The Piedmont physiographic province extends from southeastern Pennsylvania to eastern Alabama, as shown in Figure 1, and is characterized by metamorphic and igneous rocks formed during the formation of the Appalachian Mountains (Smith 1987, Pavich 1996). Surficial soil conditions in the Piedmont province generally consist of residuum and saprolite derived from the mechanically and chemically weathered parent rock. The residual soils are typically characterized as silty sand (SM) and sandy silt (ML). The surficial soils vary from loose to very dense, depending on localized weathering. With depth, the residual soils transition into saprolite, partially weathered rock, and weathered rock. The transition from soil-like to weathered rock consistency is not uniform, however, and may be quite variable over short horizontal distances. A schematic of a typical Piedmont province subsurface soil profile is presented in Figure 2.

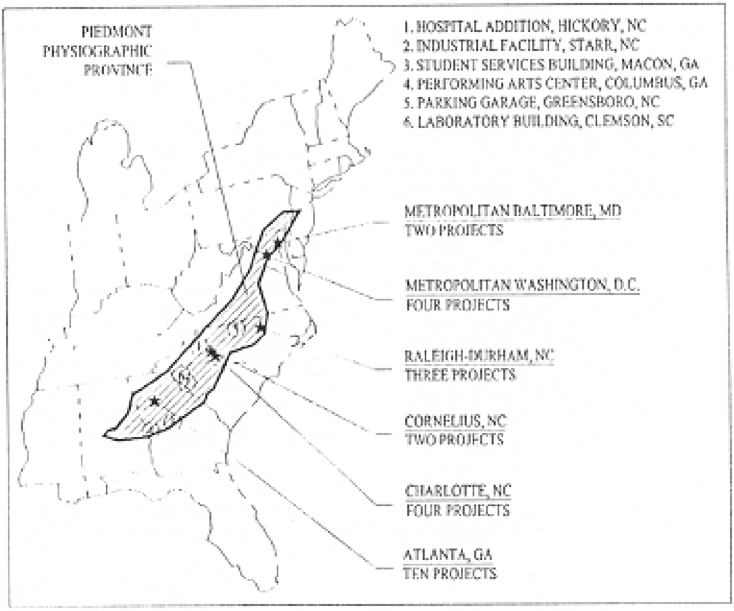


Figure 1. Piedmont province and load test locations

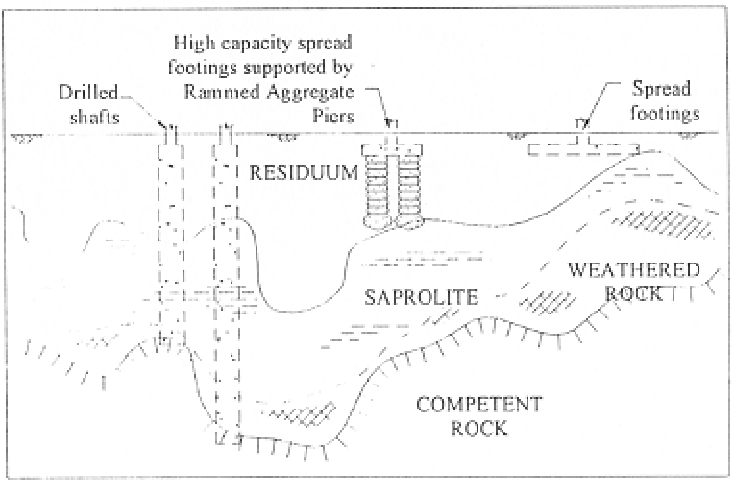


Figure 2. Typical subsurface profile

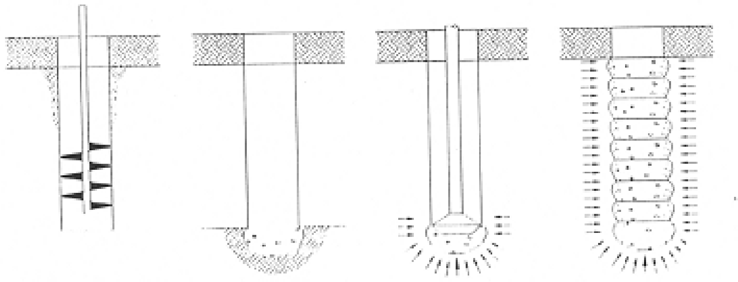
Foundation engineering and construction in the Piedmont province is made difficult by the wide variability in the soil horizons. Excavations for shallow spread footings often encounter residual materials with variable shear strength and compressibility. This condition leads to concerns regarding differential settlement. The construction of deep foundations such as drilled shafts designed for end-bearing in rock often leads to cost overruns because of the variabilities in shaft lengths and in rock drilling quantities. It is difficult to predict these variabilities prior to construction. Foundation engineering and construction in fill materials comprised of Piedmont soils is equally difficult because of the variability inherent in undocumented fill deposits and because of the high costs often associated in the removal and replacement of these materials if they extend to significant depths.

Rammed Aggregate Pier elements are used to strengthen, stiffen, and reduce the inherent variabilities within the native and fill soils of Piedmont province origin. The aggregate piers do not typically extend to a hard layer and are not intended to be end-bearing elements. Extensive load tests and field construction experience have shown that aggregate piers improve the overall stiffness of the subsurface soils at depths in which footing-induced stresses are the highest. The patented system is designed to improve the subsurface soil conditions and allow the use of high bearing pressure shallow spread footings for foundation support and to limit long-term foundation settlements to the design criterion.

CONSTRUCTION

Rammed Aggregate Pier construction is shown in Figure 3. The piers are installed by drilling 610 mm (24 inch) to 915 mm (36 inch) diameter holes to depths ranging between 2.1 m and 6.1 m (7 feet and 20 feet) below footing bottoms, placing controlled lifts of aggregate stone within the cavities, and compacting the aggregate using a specially designed high-energy beveled impact tamper. The first lift consists of clean stone and is rammed into the soil to form a bottom bulb below the excavated shaft. The bottom bulb effectively extends the design length of the aggregate pier element by one pier diameter. The piers are completed by placing additional 0.3 m (one-foot) thick lifts of aggregate over the bottom bulb and densifying the aggregate with the beveled tamper. During densification, the beveled shape of the tamper forces stone laterally into the sidewall of the excavated cavity. This action increases the lateral stress in the matrix soil thus providing additional stiffening and increased normal stress perpendicular to the perimeter shearing surface.

Aggregate piers are typically designed and installed to cover approximately 30% to 40% of the gross area of overlying footing elements. High bearing pressure spread footings, with allowable composite bearing pressures typically ranging between 240 kN/m^2 (5,000 psf) and 430 kN/m^2 (9,000 psf), are then constructed directly over the aggregate piers.



A. Make cavity. B. Place stone at bottom of cavity. C. Ram stone to form bottom bulb. D. Place and ram thin lifts to form undulated-sided shaft.

Figure 3. Construction process

DESIGN CONSIDERATIONS

Aggregate piers increase the bearing capacity of the reinforced matrix soil and significantly reduce foundation settlements. Foundation settlements are calculated using a two-step procedure: the compression of the zone of matrix soil reinforced by the aggregate piers (upper zone) is first estimated and then added to calculated compression of the zone of soil that is located below the tips of the piers (lower zone).

Upper zone calculation procedures are based on a spring analogy described in the literature (Lawton and Fox 1994, Lawton et al. 1994, Wissmann and Fox 2000, Wissmann et al. 2000). The procedure includes the assumption that the footing is rigid relative to the foundation materials. The stress that is applied to the tops of the aggregate piers (q_g) depends on the average footing-bottom stress (q), the stiffness ratio between the aggregate pier and surrounding matrix soil (R_s), and the ratio of the cross-section area of the aggregate piers to the footing bottom area (R_a):

$$q_g = q \; [R_s / (R_a R_s + 1 - R_a)] \; . \tag{1}$$

The stiffness ratio, R_s, is defined as the ratio of the aggregate pier stiffness modulus (k_g) and the matrix soil stiffness modulus (k_m), where stiffness modulus is defined as the quotient of applied stress to measured deflection at a specified top of pier stress. Stiffness modulus is expressed in units of force per length cubed (F/L^3). Settlement in the upper zone soils (s_{uz}) is simply computed as the quotient of stress applied at the top of the aggregate pier to the aggregate pier stiffness modulus:

$$s_{uz} = q_g / k_g \; . \tag{2}$$

Estimates of settlement in the lower zone materials, below the bottom of the aggregate pier bulb, are computed using conventional geotechnical settlement analysis procedures combined with soil elastic modulus values interpreted from the results of Pressuremeter and Standard Penetration Tests (Martin 1987). The analysis includes the assumption that the lower zone stress distribution induced by the footing may be estimated using solutions for a footing supported by an elastic half-space, ignoring the presence of the stiff aggregate pier elements.

MODULUS LOAD TESTS

To verify the assumed modulus values used for aggregate pier design, full-scale aggregate pier modulus tests are conducted prior to construction.

Test Procedures

The tests are performed by placing circular steel plates over the full cross-sectional area of an installed aggregate pier element and then applying pressure in gradual increments. A load test frame is shown in Figure 4. The maximum applied stress normally required for the test corresponds to 150% of the design stress computed at the top of the aggregate pier elements (Equation 1).

Load test piers often incorporate a telltale in the bottom of the pier. The telltale consists of a horizontal steel plate that is attached to two vertical bars, each extending along the sidewall of the cavity to the top of the pier. The telltale is placed in the pier after the bottom bulb is constructed and prior to the construction of the pier shaft. The bars are sleeved with PVC pipes to reduce friction between the bars and the adjacent aggregate. The telltales are used to monitor vertical deflections at the base of the piers during applications of stress at the tops of the piers.

Stress-Deformation Mechanics

As can be seen in Figure 5, loading initiates a linear response both at the top and at the bottom of the piers. At any given deflection, shown by the dashed horizontal lines, the top plate pressure represents the total load. The bottom plate movement at the same level of stress is an indication of the portion of the load that is taken in end bearing at the top of the enlarged bulb at the end of the pier. The vertical separation between the two lines is significant because it is an indication of the load response that is transferred to the soil by side friction along the shaft.

Type B (bulging) behavioral mode. An inflection point for the top plate but not for the telltale near the bottom, as shown in Figure 5A, indicates pier bulging. A steeper linear stress deflection response of the top plate is observed beyond the inflection point as bulging continues. Even though it acts to increase settlement, the bulging action is considered to be an advantage of aggregate piers over more rigid materials, by acting as a cushion that will help to distribute the load among a pier group instead of concentrating it on the most resistant pier within the group. The

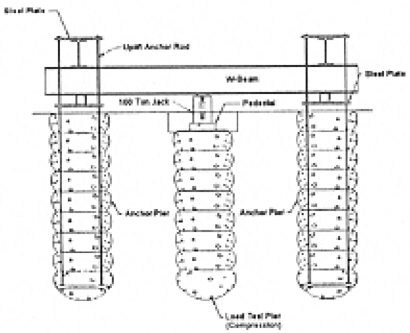


Figure 4. Load test frame

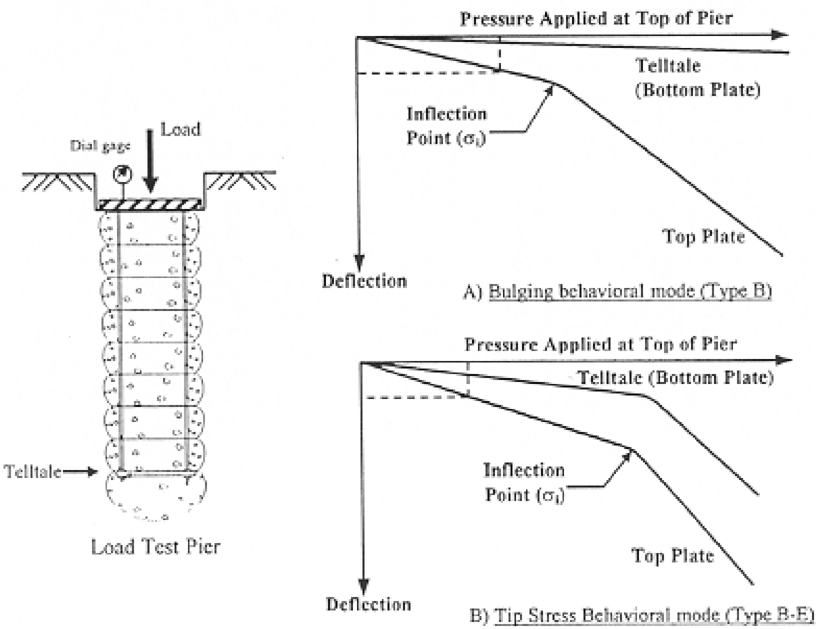


Figure 5. Typical modulus load test results

improved load distribution is particularly important in Piedmont soils where the depth to rock is not uniform, and one element in a group might bear on rock while others bear on soil.

Type B-E (bulging-end bearing) behavioral mode. The inflection of the top of the pier response sometimes corresponds to an inflection point in the bottom of pier (telltale) response. This behavior, shown in Figure 5B, indicates an initiation of an increase in load taken on the bottom bulb. The load transferred to the bottom bulb will act first to compress the bulb and then be distributed by end bearing to soil underneath the bulb. Type B-E behavior is more characteristic of short piers because of the limited shaft length for development of side friction. As in the case of Type B behavior, any increase in vertical separation in this zone indicates an increase in side friction that probably is a result of continued bulging. The perimeter side shearing capacity of the piers (Q_s) is calculated as the product of the average frictional resistance offered by the matrix soils and the perimeter area (A_s) of the element (Lawton and Fox 1994):

$$Q_s = \sigma_v' K_p \tan\phi' A_s \,, \tag{3}$$

where σ_v' is the average effective vertical stress within the matrix soils surrounding the element, K_p is the matrix soil Rankine passive earth pressure coefficient, and ϕ' is the matrix soil angle of internal friction. The Rankine passive earth pressure coefficient is implemented for analysis because of the high lateral earth pressures that are induced during pier construction (Wissmann and Fox, 2000). The stress at the top of the pier at the inflection point (σ_i) for piers exhibiting Type B-E behavior is computed by dividing the perimeter shearing capacity (Q_s) by the element cross-sectional area (A_g):

$$\sigma_i = Q_s / A_g \,. \tag{4}$$

Test Results

Table 1 summarizes the results of 31 modulus load tests conducted within native and fill soils of Piedmont origin. Table 1 summarizes test pier geometries, load test results, if an inflection point is noted in the test curve, the interpreted mechanism inherent in deformations past the inflection point test results, and average SPT N-values obtained prior to pier installation

For all tests, top of pier stresses measured at the inflection point range between 310 kN/m^2 (6,450 psf) and 1,300 kN/m^2 (26,600 psf) and top of pier deflections range between 2.8 mm (0.1 inch) and 23 mm (0.9 inch) at the inflection point. As shown in Figure 6, inflection point stresses generally increase with increasing matrix soil SPT N-value confirming the concepts embodied in existing design methods (Fox and Cowell, 1998). For tests in which an inflection point is not reached, the data presented in Figure 6 represent maximum applied stress; these

Table 1. Modulus load test results

Project (1)	Project Location (2)	Pier Diameter, d (mm, [in]) (3)	Depth to Top of Pier, Df (m, [ft]) (4)	Pier Shaft Length, Hs (m, [ft]) (5)	Inflection Point Observed in Plot? (6)	Interpreted Deformation Mechanism (7)	Inflection Point Stress, σ, (kPa, [psf]) (8)	Top of Pier Deflection at Inflection Pt. (mm, [in]) (9)	Stiffness Modulus,[1] kg (MN/m3, [pci]) (10)	ASTM Soil Classif. (11)	Avg. SPT N-value (12)
Native Soils of the Piedmont Province											
Hospital Addition	Hickory, NC	914 [36]	0.6 [2]	2.4 [8]	No	n/a	1,267.8 [26,478]	12.1 [0.48]	105.1 [387]	ML	7
Parking Garage	Charlotte, NC	914 [36]	0.8 [2.5]	2.1 [7]	Yes	Tip deflect.	308.8 [6,450]	2.7 [0.11]	115.4 [425]	ML	5
Building and Parking Garage	Atlanta, GA	762 [30]	0.9 [3]	3.0 [10]	Yes	-	1,103.6 [23,050]	22.8 [0.90]	48.3 [178]	ML	8
Building and Parking Garage	Atlanta, GA	762 [30]	0.9 [3]	1.8 [6]	Yes	Tip deflect.	847.5 [17,700]	13.2 [0.52]	64.1 [236]	ML	8
Office Building	Raleigh, NC	914 [36]	0.3 [1]	3.7 [12]	No	n/a	1,115.7 [23,303]	11.8 [0.47]	94.2 [347]	ML,SM	16
Office Building	Raleigh, NC	762 [30]	0.6 [2]	2.1 [7]	No	n/a	1,275.5 [26,640]	15.3 [0.60]	83.6 [308]	ML,SM	8
Office Building	McLean, VA	762 [30]	0.6 [2]	2.4 [8]	Yes	-	1,067.7 [22,300]	12.9 [0.51]	83.1 [306]	ML	10
Parking Garage	Charlotte, NC	762 [30]	0.3 [1]	2.4 [8]	Yes	Tip deflect.	727.8 [15,200]	8.7 [0.34]	83.9 [309]	CL,ML	10
Morehead Parking Garage	Charlotte, NC	762 [30]	0.6 [2]	3.4 [11]	Yes	Bulging	1,153.9 [24,100]	11.1 [0.44]	103.7 [382]	CL,ML	10
Industrial Facility	Starr, NC	762 [30]	1.8 [6]	2.4 [8]	No	n/a	1,533.8 [32,035]	12.1 [0.48]	126.5 [466]	ML,SM	30
Office Building	Raleigh, NC	762 [30]	0.8 [2.5]	2.4 [8]	Yes	Tip deflect.	1,292.8 [27,000]	16.0 [0.63]	80.9 [298]	ML,SM	14
College Building	Macon, GA	762 [30]	0.3 [1]	2.4 [8]	No	n/a	1,067.7 [22,300]	16.1 [0.63]	66.5 [245]	SM	4
Performing Arts Center	Columbus, GA	762 [30]	0.3 [1]	3.0 [10]	Yes	-	790.0 [16,500]	9.2 [0.36]	86.3 [318]	SP,SM	8
Office Building	Atlanta, GA	762 [30]	0.3 [1]	3.0 [10]	No	n/a	1,168.3 [24,400]	10.2 [0.40]	115.1 [424]	ML,SM	10
Fill Soils of Piedmont Origin											
Apartment Building	Atlanta, GA	914 [36]	0.6 [2]	3.8 [12.5]	Yes	Bulging	598.5 [12,500]	12.0 [0.47]	49.9 [184]	ML	5
Apartment Building	Atlanta, GA	914 [36]	0.6 [2]	4.7 [15.5]	Yes	Bulging	462.0 [9,650]	9.2 [0.36]	50.2 [185]	ML	6
Parking Garage	Atlanta, GA	914 [36]	0.9 [3]	3.4 [11]	Yes	-	694.3 [14,500]	11.8 [0.47]	58.6 [216]	ML	7
Apartment Building	Atlanta, GA	762 [30]	0.8 [2.5]	3.7 [12]	Yes	Bulging	620.0 [12,950]	17.6 [0.69]	35.3 [130]	ML	6
Apartment Building	Atlanta, GA	762 [30]	0.8 [2.5]	2.1 [7]	Yes	Bulging	636.8 [13,300]	17.0 [0.67]	37.5 [138]	ML	5
Hotel	Charlotte, NC	762 [30]	0.3 [1]	2.4 [8]	No	n/a	633.5 [13,230]	6.5 [0.26]	97.7 [360]	ML,SM	10
Condominiums	Montgomery, MD	762 [30]	0.8 [2.5]	3.0 [10]	No	n/a	1,101.2 [23,000]	15.9 [0.63]	69.2 [255]	SM,ML,CL	11
Condominiums	Cornelius, NC	610 [24]	0.5 [1.5]	2.1 [7]	No	n/a	817.0 [17,064]	8.7 [0.34]	93.6 [345]	ML	5
Storage Building	Elliott City, MD	762 [30]	0.3 [1]	2.7 [9]	Yes	Bulging	646.4 [13,500]	11.5 [0.45]	56.2 [207]	ML	-
Condominiums	Cornelius, NC	610 [24]	0.3 [1]	2.4 [8]	No	n/a	911.3 [19,033]	7.7 [0.30]	118.6 [437]	MH	5
Parking Garage	Tyson's Corner, VA	762 [30]	0.3 [1]	2.1 [7]	Yes	Tip deflect.	1,311.9 [27,400]	12.8 [0.50]	102.6 [378]	ML	9
Office Building	Washington D.C.	762 [30]	0.6 [2]	2.4 [8]	No	n/a	1,259.7 [26,310]	15.7 [0.62]	80.3 [296]	ML	3
Office Building	Columbia, MD	762 [30]	0 [0]	2.4 [8]	Yes	Tip deflect.	746.9 [15,600]	7.6 [0.30]	98.0 [361]	ML	6
Parking Garage	Greensboro, NC	762 [30]	0.8 [2.5]	2.4 [8]	Yes	Tip deflect.	957.6 [20,000]	15.3 [0.60]	62.7 [231]	ML,SM	15
Medical Building	Atlanta, GA	762 [30]	0.3 [1]	1.8 [6]	Yes	Tip deflect.	502.7 [10,500]	7.6 [0.30]	66.0 [243]	ML,SM	6
Biotechnology Building	Clemson, SC	762 [30]	0.8 [2.5]	3.0 [10]	Yes	-	957.6 [20,000]	13.9 [0.55]	68.7 [253]	ML,SM	10
Commercial Development	Atlanta, GA	762 [30]	0.3 [1]	2.1 [7]	Yes	Tip deflect.	957.6 [20,000]	4.8 [0.18]	209.3 [771]	ML,SM	6

[1]Stiffness modulus is defined as ratio of top of pier stress to top of pier deflection measured at the inflection point. For tests that do not exhibit an inflection point, the stiffness modulus is the ratio of top of pier stress to top of pier deflection at the maximum applied stress.

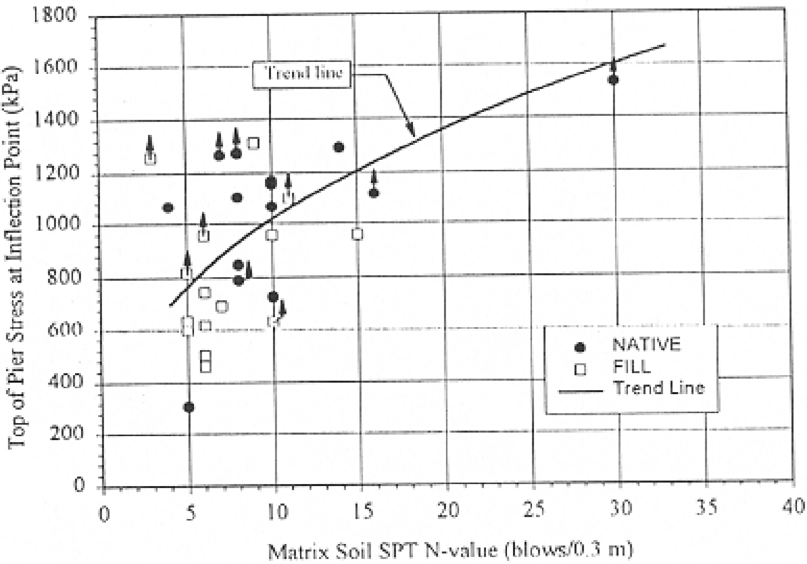


Figure 6. Top of pier stress at point of inflection

points are identified with an upward-pointing arrow indicating that additional stress could be applied prior to reaching the inflection point. Pier stiffness modulus values range between 35 MN/m^3 (130 pci) and 209 MN/m^3 (771 pci) at the inflection point for all tests. As shown in Figure 7, pier stiffness modulus values generally increase with increasing matrix soil SPT N-value.

The mobilization of tip resistance is identified as the controlling mode of deformation in nine of the tests and is thought to be the controlling mode of deformation in five additional tests. Bulging is observed as the controlling deformation mechanism in only six of the tests, five of which were conducted in loose and variable fill soils.

Discussion

For tests in which the mobilization of tip resistance was identified as the controlling mode of deformation, the inflection point stress may be computed using Equations 3 and 4. Figure 8 presents a comparison of estimated inflection point stress and measured inflection point stress for an assumed soil unit weight of 18.9 kN/m^3 (120 pcf) and an assumed angle of internal friction of 32 degrees. The reasonably good agreement between computed and measured stress values suggests that the computational method provides a good approximation of field conditions.

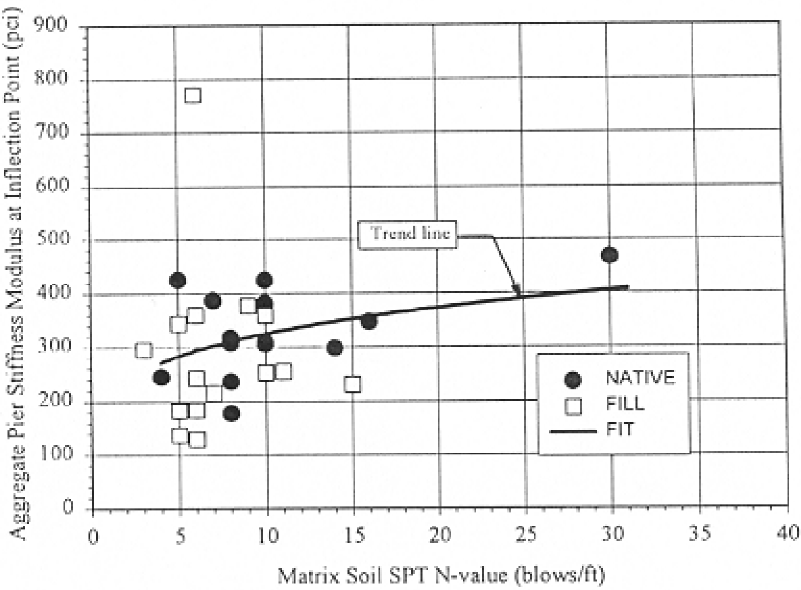


Figure 7. Stiffness modulus values at point of inflection

The results of similar computations using an angle of internal friction of 28 degrees indicates that all of the computed inflection point stresses are lower than those measured.

For tests in which the mobilization of tip resistance is identified as the controlling mode of deformation, an equivalent elastic modulus value (E) may be established from the test results by multiplying the pier stiffness modulus (k_g) at the inflection point by the pier shaft length. Equivalent elastic modulus values ranging between 115 MPa (1,200 tsf) and 270 MPa (2,800 tsf), with an average elastic modulus value of about 182 MPa (1,900 tsf), are computed from the results of tests that exhibit an inflection response attributed to tip stress mobilization and for tests in which tip stress mobilization is thought to control pier response.

REDUCTION OF FOUNDATION SETTLEMENT RISK

The design of shallow spread footings in Piedmont soils is made difficult by the wide variability in soil compressibility. Foundation settlement estimates are typically made using classical geotechnical analysis techniques that incorporate the concept of load spreading with depth and rely on empirical correlations to establish foundation soil elastic modulus (E) values. A widely used relationship that correlates matrix soil elastic modulus to matrix soil SPT N-value is presented in the bottom portion of Figure 9 (after Martin 1987). The plotted relationship was developed by multiplying Pressuremeter modulus values by 1.67 in accordance with Martin's recommendations. The data indicate that Piedmont soil elastic modulus values generally increase with increasing N-value but exhibit considerable variability over the entire range of N-values.

The equivalent elastic modulus values for the pier elements are also represented in Figure 9. The average of these values is 5 to 60 times greater than the recommended matrix soil modulus values, for SPT N-values ranging between 3 and 50. Table 2 presents a comparison of the average equivalent elastic modulus values for the aggregate pier elements and the recommended elastic modulus values for matrix Piedmont soil.

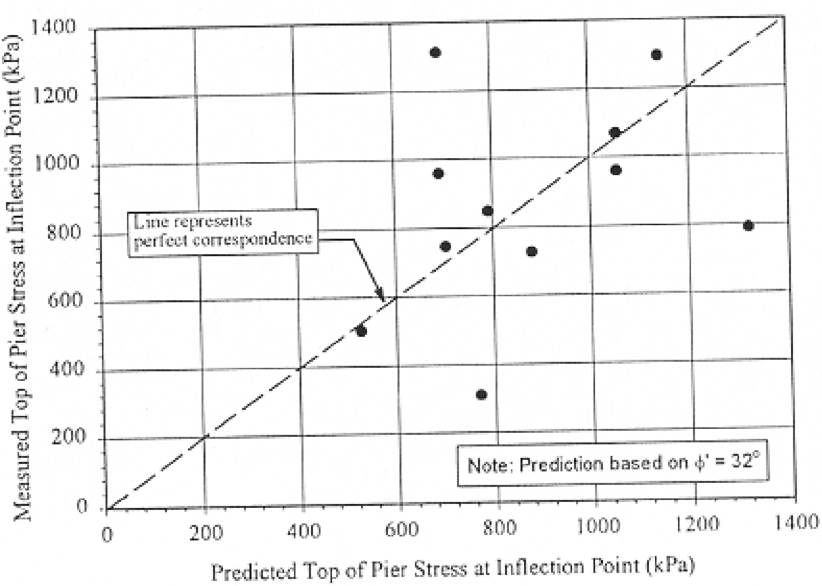


Figure 8. Predicted inflection point stress for aggregate piers that exhibit tip stress mechanism

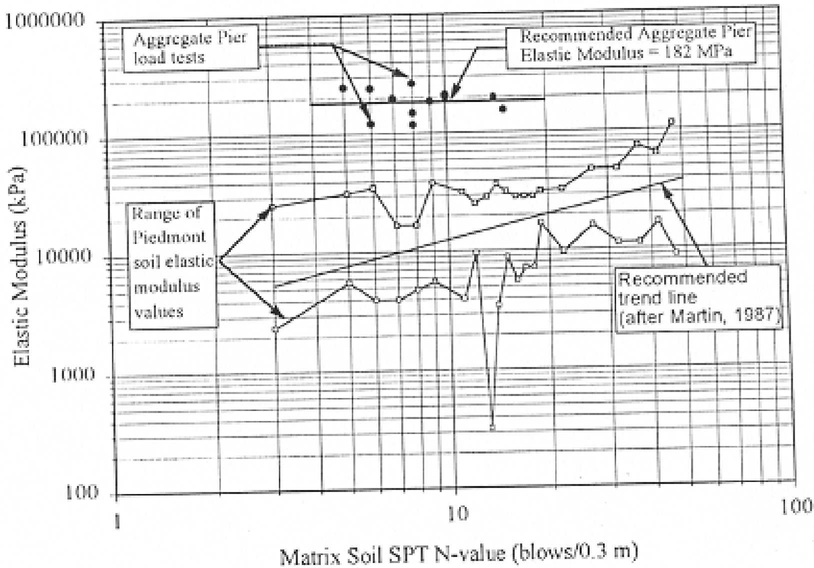


Figure 9. Comparison of elastic modulus values

Table 2: Comparison of elastic modulus values

SPT N-value (1)	Recommended matrix soil elastic modulus value[1] (MPa [tsf]) (2)	Average aggregate pier equivalent elastic modulus value (MPa [tsf]) (3)	Ratio of elastic modulus values (4)
3	2.9 [30]	182 [1,900]	63
8	10 [110]	182 [1,900]	18
50	38 [400]	182 [1,900]	5

[1]After Martin 1987.

The ratios presented in Table 2 are consistent with previous findings for tests conducted in soft lakebed deposits (Lawton 1999) and may be implemented as R_s within Equation 1 to estimate upper zone settlements below shallow spread footings. For typical design conditions and soil conditions exhibiting a stiffness ratio of 18, the settlement of footings supported by unreinforced Piedmont soils may be calculated to be 6 times greater than the settlement of equivalent footings supported by aggregate piers, confirming the small settlements measured for constructed buildings (Fox and Cowell 1998, Wissmann et al., 2000).

As shown graphically in Figure 9, the variability of the aggregate pier equivalent elastic modulus values is much lower than the variability in the matrix soil elastic modulus values. The ratio between the highest and lowest aggregate pier elastic modulus value is about 2 for N-values ranging between 5 and 14. The ratio between the highest and the lowest Piedmont soil elastic modules value is 11 for the same range of N-values, even after the lowest data point is discarded. The variability in the elastic modulus for unreinforced Piedmont soil is more than 5 times greater than that for the aggregate piers installed in Piedmont soils. The wide range in the response of Piedmont soils is a result of the variability inherent in the weathered soil profile and results in the potential for large differential settlements between adjacent footings.

SUMMARY AND CONCLUSIONS

The residual soils of the Piedmont physiographic province pose challenges for geotechnical engineers because of the difficulty defining soil, rock, and transition zones in the subsurface profile and because of the widely variable engineering properties associated with these materials. *Rammed Aggregate Pier* elements are increasingly being used as a cost-effective solution to reduce the variability and to increase the overall strength and stiffness of residual Piedmont soils. This paper presents the results of 31 load tests performed for aggregate piers constructed in the Piedmont province. The test results indicate the following.

1. Top of pier inflection point stresses and pier stiffness modulus values generally increase with matrix soil N-value.

2. The behavior of *Rammed Aggregate Pier* elements at high stress levels is controlled by two mechanisms: the potential for additional bulging and the potential for the mobilization of tip stresses. The latter mechanism dominates the behavior of aggregate piers in Piedmont soils.

3. Equivalent elastic modulus values ranging between 115 MPa (1,200 tsf) and 270 MPa (2,800 tsf) are computed for aggregate piers installed in Piedmont soils. These values are 5 to 60 times larger than matrix soil elastic modulus values, depending on matrix soil N-value, indicating proportionate decreases in settlement. By incorporating these high modulus elements within Piedmont soils, settlement of spread footings can be controlled within close tolerances and the need for deep foundations can often be eliminated.

4. The use of aggregate piers reduces the variability of the elastic modulus used for settlement calculations. The variability of aggregate pier equivalent elastic modulus values is five times smaller than that for matrix Piedmont soils. This reduction in variability allows for increased confidence in mitigating differential settlements.

To date more than 70 major structures are supported by *Rammed Aggregate Pier* elements in the area of the Piedmont Province; aggregate piers nationwide support more than 300 structures.

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