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FLAT DILATOMETER MODULUS APPLIED TO DRILLED SHAFT FOUNDATIONS IN PIEDMONT RESIDUUM

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ABSTRACT

The axial load-displacement response of drilled shaft foundations in Piedmont residual soils can be assessed using an elastic continuum approach. Equivalent elastic moduli measured by flat plate dilatometer tests are shown to provide reasonable deflections at "working loads" when evaluating settlements for axial compression loading. Load transfer to the base is also evaluated. Two case studies involving field load testing of end-bearing and friction-type drilled shafts in Atlanta, Georgia are presented.

INTRODUCTION

The name Piedmont means "foot-of-the-mountains", yet the Paleozoic mountains are long gone, having been subjected to very long periods of erosion and weathering. The existing overburden consists of residual soil that was derived from the in-place decomposition of the parent metamorphic and igneous rocks that grades to saprolite and partially-weathered rock (Sowers & Richardson, 1983). Primary rock types include schist, gneiss, and granite, although localized regions include phyllite, slate, soapstone, greenstone, & diabase. The depth to rock varies because of differential weathering.

Drilled shafts are used extensively in Piedmont residuum for foundation support of large building columns, bridge piers, and towers. The geology is important in that many important cities and their expanding suburbs are located in the Piedmont, including Philadelphia, Baltimore, Washington-D.C., Richmond, Charlotte, Raleigh, and Atlanta.

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In the southern Piedmont, the overburden has been termed "Georgia Red Clay", based on local experiences with shallow topsoils affected by rainfall and their agricultural associations. In contrast, geotechnical categorization by the Unified Soils Classification System (USCS) place the residual materials as sandy silts (ML) and silty sands (SM). Thus, the contradictory notions of "clay versus sand" have resulted in confusion as to whether the Piedmont soils behave in an undrained or drained manner, as well in deciding which geotechnical tests are appropriate for defining the engineering parameters in analysis. In this paper, the use of an equivalent elastic moduli obtained from the flat plate dilatometer test (DMT) is shown to provide reasonable values in Piedmont soils for use in settlement calculations involving drilled shafts.

CHARACTERISTICS OF ATLANTA TEST SITE

A full-scale load test program on the axial behavior of drilled shaft foundations in Piedmont geology was conducted at a test site on the west side of the Georgia Tech campus in northwest Atlanta. Axial compression tests were performed on two similar drilled shafts with one end-bearing on rock and the other a friction-type shaft supported entirely within the residual soil matrix. These tests were conducted by the Association of Drilled Shaft Contractors (ADSC), Federal Highway Administration (FHWA), and local ASCE Atlanta Geotechnical Section, in conjunction with the geosystems group at the Georgia Institute of Technology. The testing was performed just prior to dormitory construction for the 1996 Summer Olympics.

A fairly extensive site characterization program was conducted over the test area, approximately 30 by 30 meters in plan. Field testing included standard penetration, electric cone penetration, flat dilatometer, pressuremeter, and geophysical methods in the soils, and diamond coring of the gneissic schist (Harris & Mayne, 1994). The in-situ results were complemented by series of laboratory index, consolidation, and triaxial testing. The groundwater table lies about 16 meters deep at this location.

Beneath a thin 3-m layer of silty fill, the results of nine soil test borings with standard penetration testing (SPT) showed N-values gradually increasing from 8 blows at 3 meters depth to around 25 blows at 16 meters depth (Figure 1). Then, the N-values increase dramatically to over 100+ until refusal is met at depths of about 22 meters. Rock coring at five boring locations confirmed gneiss and schist bedrock with fair rock quality designations (RQD) between 29 and 38 percent.

The very high N-values measured at depths from 16 to 22 meters implicated either the presence of a very hard or dense material, termed saprolite, or alternatively, an abundance of rock fragments. However, grain size analyses of all 113 drive samples revealed that the constituents of the residual soils and the less-weathered saprolite were identical. Figure 2 shows that, despite the dramatic increases in N-values, all samples contained the same basic makeup: 67 percent fine sand, 25 percent silt, and 8 % clay fraction. Representative soil indices included: mean grain size, D_{50} = 0.14 mm; fines content (< 75 microns) = 33 %; clay fraction (< 2 microns) = 8 %; liquid limit (LL) =

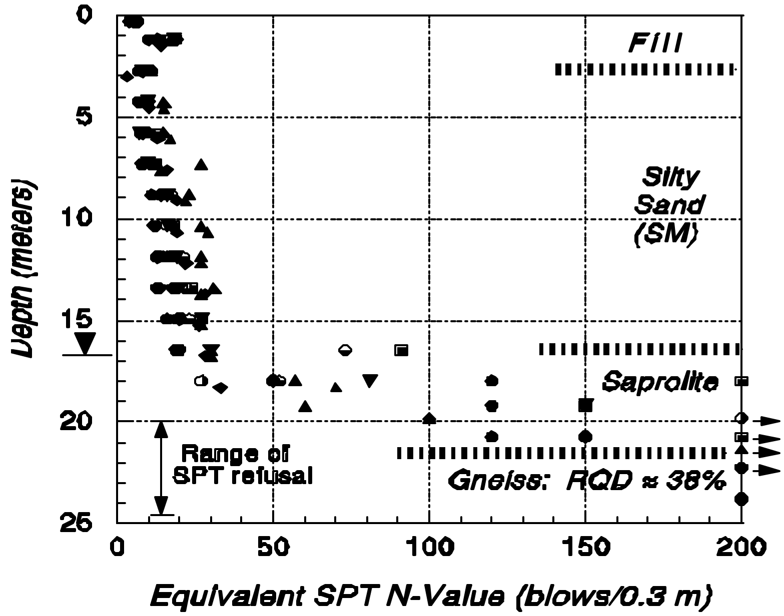


Fig. 1. Standard Penetration Tests in Piedmont Residuum at Atlanta Test Site.

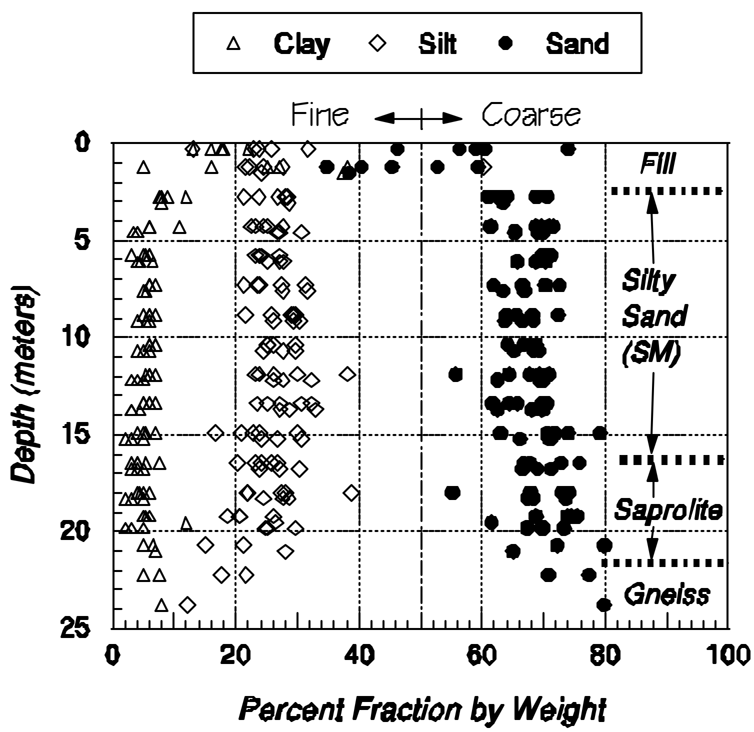
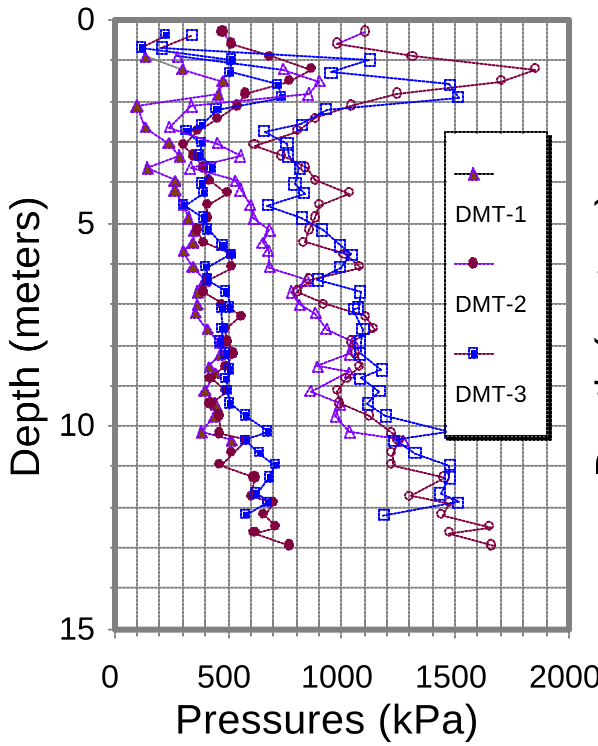


Figure 2. Summary of Grain Size Distributions from Drive Samples.



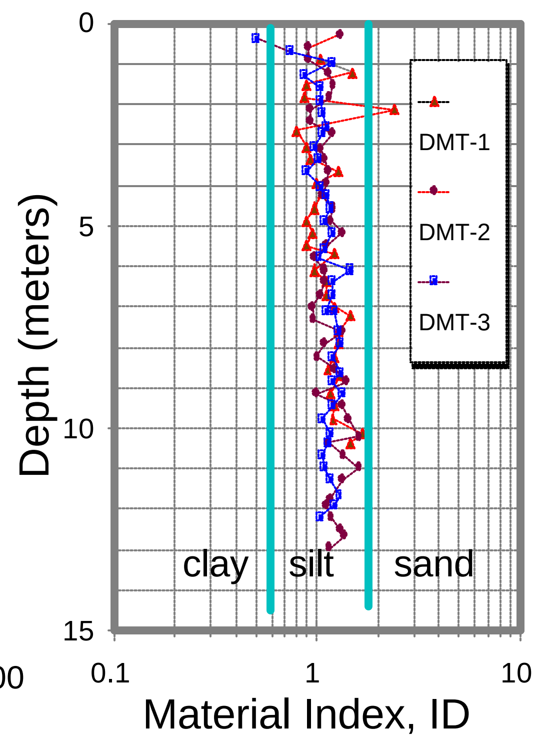


Fig. 3. Contact and Expansion Pressures

Fig. 4. Profile of Material Index

37 %; plasticity index (PI) = 11 % to nonplastic; and inplace void ratio (e_o) = 0.72. In accordance with the Unified Soils Classification System, the Piedmont categorizes as "SM" at this site.

Clearly, the "SM" designation is at odds with the term "Georgia Red Clay". With such a significant fines fraction, the Unified Soils Classification provides a dubious description rating for Piedmont soils and thus fails to indicate its behavioral aspects. When the rate of applied loading is sufficiently fast, as for many in-situ tests, these residual soils can behave in an undrained manner when saturated. Notably, however, during normal rates of building construction which are comparatively slow, the Piedmont essentially exhibits a drained response, and thus effective stress analyses are appropriate.

Three flat dilatometer tests (DMT) were performed at the site using the procedures outlined by Schmertmann (1986). Since the blade was hydraulically pushed using a standard drill rig, the forces were insufficient to fully penetrate the residual soil layer. Figure 3 shows the profiles of lift-off (contact) pressure (p_0) and expansion pressure (p_1) from three soundings performed at the test site. The variable nature of the upper fill is evident and the general repeatability and consistent readings in the natural silty sands are apparent from the results. Figure 4 shows the derived profile of material index (I_D) with depth, where $I_D = (p_1-p_0)/(p_0-u_0)$. The index $I_D < 0.6$ for clay soils, silts are noted with $0.6 < I_D < 1.8$, and sands have I_D

> 1.8 (Marchetti, 1980). The profile in Fig. 4 suggests the presence of sandy silt. The dilatometer modulus (E_D) is determined as:

$$E_D = 34.7\ (p_1 - p_0) \qquad (1)$$

where consistent units are used for p_0, p_1, and E_D. At this site, the profile of dilatometer modulus (E_D) is seen to increase with depth (z), as evidenced by Figure 5. This trend can be represented by a Gibson-type format:

$$E_D\ (\text{MPa}) = 6.6 + 1.9\ z\ (\text{meters}) \qquad (2)$$

as obtained from regression analysis of the DMT data for z > 3 meters. Notably, the parameter E_D was originally derived on the basis of elastic theory (Marchetti, 1980) and it is therefore of interest to pursue this as a means of quantifying the stiffness of the ground for foundation settlement analysis.

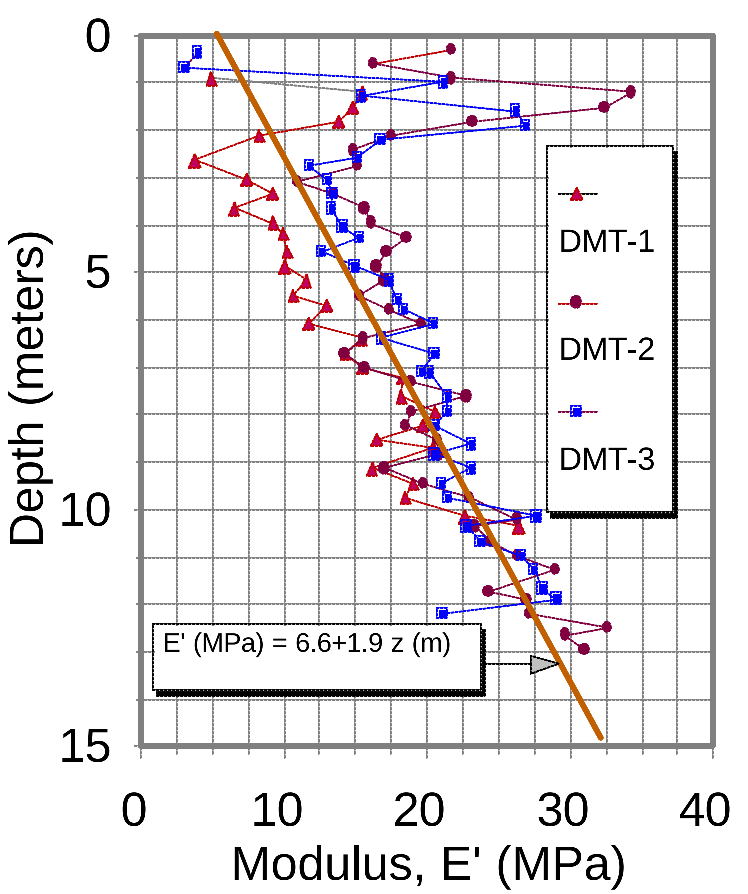


Figure 5. Derived Gibson-Type Profile of Equivalent Elastic Modulus.

AXIAL LOAD-DISPLACEMENT RESPONSE OF DEEP FOUNDATIONS

The axial load-displacement behavior of deep foundations may be represented by elastic continuum theory where solutions have been developed from boundary element formulations (Poulos & Davis, 1980), finite elements (Poulos, 1989), and approximate closed-form analytical solutions (Randolph & Wroth, 1978, 1979; Fleming et al. 1985). Continuum theory characterizes the soil stiffness by two elastic parameters: an equivalent elastic soil modulus (E_s) and Poisson's ratio ($<_s$). Two general cases are considered: (1) homogeneous case where E_s is constant with depth; and (2) a Gibson-type condition where E_s is linearly-increasing with depth. Both floating-type or end-bearing type piles can be considered. Figure 6 depicts the generalized stiffness profile that is capable of representing residual soil over a weathered rock stratigraphy.

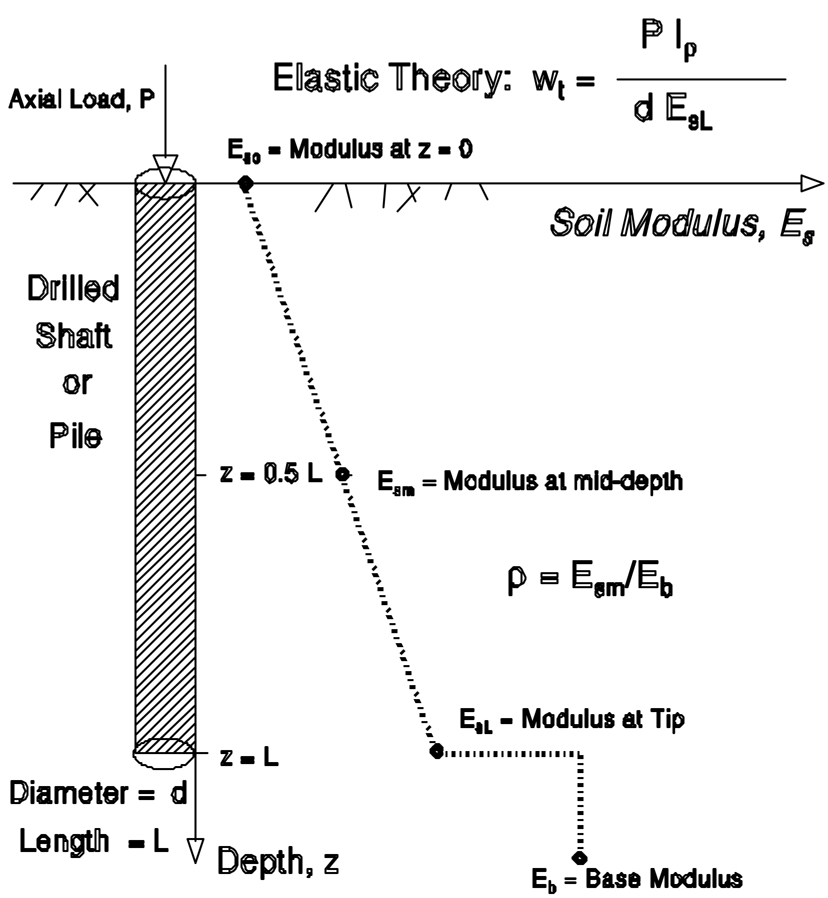


Figure 6. Moduli Definitions Used in Elastic Continuum Model.

The vertical displacement (w_t) of a pile foundation subjected to axial compression loading is expressed (Poulos, 1987, 1989):

$$w_t = P\, I_\rho /(E_{sL} d) \qquad (3)$$

where P = applied axial load at the top of the shaft, E_{sL} = soil modulus at the foundation base, d = foundation diameter, and I_ρ = influence factor. The factor I_ρ depends on the pile slenderness ratio (L/d), pile material, and relative soil-pile stiffness, as given in chart solutions, tables, or approximate closed-form. The latter is given in concise form (Randolph & Wroth, 1978, 1979; Poulos, 1987):

$$I_\rho = 4(1+\nu)\,\frac{\left[1 + \frac{1}{\pi\lambda}\,\frac{8}{(1-\nu_s)}\,\frac{\eta}{\xi}\,\frac{\tanh(\mu L)}{\mu L}\,\frac{L}{d}\right]}{\left[\frac{4}{(1-\nu_s)}\,\frac{\eta}{\xi} + \frac{4\pi\rho}{\zeta}\,\frac{\tanh(\mu L)}{\mu L}\,\frac{L}{d}\right]} \qquad (4)$$

where the following terms apply:

- d = shaft diameter.
- L = pile length.
- η = d_b/d = eta factor (d_b = diameter of base, so that η = 1 for straight shafts).
- ξ = E_{sL}/E_b = xi factor (ξ = 1 for floating pile).
- ρ = E_{sm}/E_{sL} = rho (ρ = 1 for uniform soil; ρ = 0.5 for simple Gibson soil).
- λ = $2(1+\nu_s)E_p/E_{sL}$ = lambda factor.
- ζ = $\ln\{[0.25 + (2.5\rho(1-\nu_s) - 0.25)\xi]\,(2L/d)\}$ = zeta factor.
- μL = $2(2/\zeta\lambda)^{0.5}\,(L/d)$ = mu factor.
- E_p = pile modulus (concrete plus reinforcing steel).
- E_{sL} = soil modulus value at pile shaft at level of foundation base (pile tip).
- E_{sm} = soil modulus value at mid-depth of pile shaft.
- E_b = soil modulus below foundation base (Note: $E_b = E_{sL}$ for floating pile).
- ν_s = Poisson's ratio of soil.

The fraction of the total load load transferred to the pile base is given by (Fleming et al. 1985):

$$P_b/P_t = \frac{\left[\frac{4}{(1-\nu_s)}\,\frac{\eta}{\xi}\,\frac{1}{\cosh(\mu L)}\right]}{\left[\frac{4}{(1-\nu_s)}\,\frac{\eta}{\xi} + \frac{4\pi\rho}{\zeta}\,\frac{\tanh(\mu L)}{\mu L}\,\frac{L}{d}\right]} \qquad (5)$$

which conveniently has the same denominator as eq (4) for spreadsheet use.

AXIAL LOAD TESTS

Two concrete test shafts were constructed with 0.76-m diameters (30 in) at the site. Test shaft C1 was an end-bearing type foundation with an embedded length of 21.3 meters (70 ft) and founded on the top of schist bedrock. Test shaft C2 was a friction-type shaft which was constructed with a length of 16.8 m (55 ft) and essentially floating within the residual soil matrix. Both shafts were provided with full-length reinforcing cages to allow load transfer measurements by vibrating wire strain gages. For each load test, a large steel beam was tied to reactions shafts that were constructed with 1.22-m diameters (48 in) and 22-m lengths (72 ft). Axial loads were applied stepwise using a hydraulic jacking system and redundant measurements of deflection were obtained using dial gages, levels, and wire setups.

For the end-bearing shaft (C1), a sequence of stepped loads attained a maximum applied axial force of 8.9 MN (1000 tons) to the top of the shaft, corresponding to the limits of the hydraulic jacking system. This was just shy of the anticipated capacity of 9.3 MN (1045 tons) given by the offset-line method. For the floating shaft (C2), the testing program provided a maximum load of P_t = 4.5 MN (506 tons) which fully-mobilized the side resistance component and induced a recorded settlement of 150 mm (6 in). Table 1 summarizes key information regarding these two load tests. Details concerning the axial capacity calculations involving drilled shafts in the Piedmont formation are given elsewhere (e.g., Gardner, 1987; Harris & Mayne, 1994).

Table 1. Summary of Test Shaft Geometries and Measured Load Test Values.

Test Shaft Number	Diam. d (m)	Length, L (meters)	Depth to Rock, h (m)	Max. Applied Load (MN)	Measured Deflection at P_{max} (mm)	Capacity by Offset Line (MN)
End-Bearing C1	0.76	21.3	20.7	8.9	26	9.3
Floating Type C2	0.76	16.7	20.1	4.5	150	3.1

The DMT modulus (E_D) can be considered to be a first-order value of the drained equivalent elastic modulus (E') for Piedmont soils. For the end-bearing shaft, the DMT data indicate that the equivalent modulus at mid-shaft is E_{sm} = 27 MPa (281 tsf) and at the base level E_{sL} = 46 MPa (479 tsf) , giving a modulus rate parameter D = 0.6 (Gibson-type profile). The equivalent modulus of the weathered gneiss below the foundation base was evaluated based on correlations between RQD and shear wave velocities (Mayne, 1998). A parametric range of E_b values taken from 100 to 500 MPa (1040 to 5200 tsf) showed that accurate moduli for the bedrock were not required for analysis, as this range only altered the calculated displacements up or down by about 10 percent. Figure 7 shows the comparison of predicted and measured response for E_b = 184 MPa (1920 tsf), or $E_b = 4\ E_{sL}$. Notably, the calculated proportions of load shared between the side and base components were in agreement with those measured by the vibrating wire strain gages.

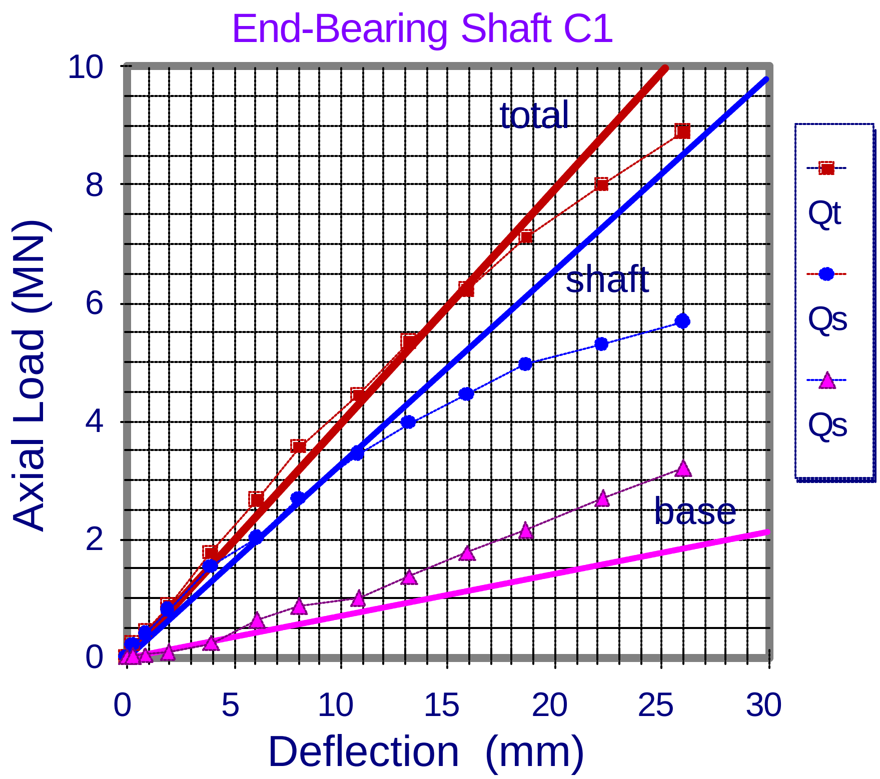


Figure 7. Load-Displacement Response of the End-Bearing Shaft C1.
Note: Measured (dots) and Predicted (lines).

For the floating shaft, the DMT data give an equivalent modulus at mid-shaft with E_{sm} = 22 MPa (230 tsf) and at the base level E_{sL} = 38 MPa (395 tsf) with a Gibson parameter D = 0.6 for the rate of modulus increase with depth. Since the bedrock lies only 3.4 meters (11 feet) beneath the base of this shaft, it also is necessary to reduce the value of the displacement influence factor I_D obtained from (4). This was accomplished using the depth correction factor (R_h = 0.6) on I_D for the case of h/L = 1.2, given by Poulos & Davis (1980). The measured and predicted axial load-displacement responses are shown in Figure 8. In this case, the measured response was quite a bit more nonlinear and therefore nonlinear approaches have been sought to better represent the behavior (e.g., Mayne & Dumas, 1997).

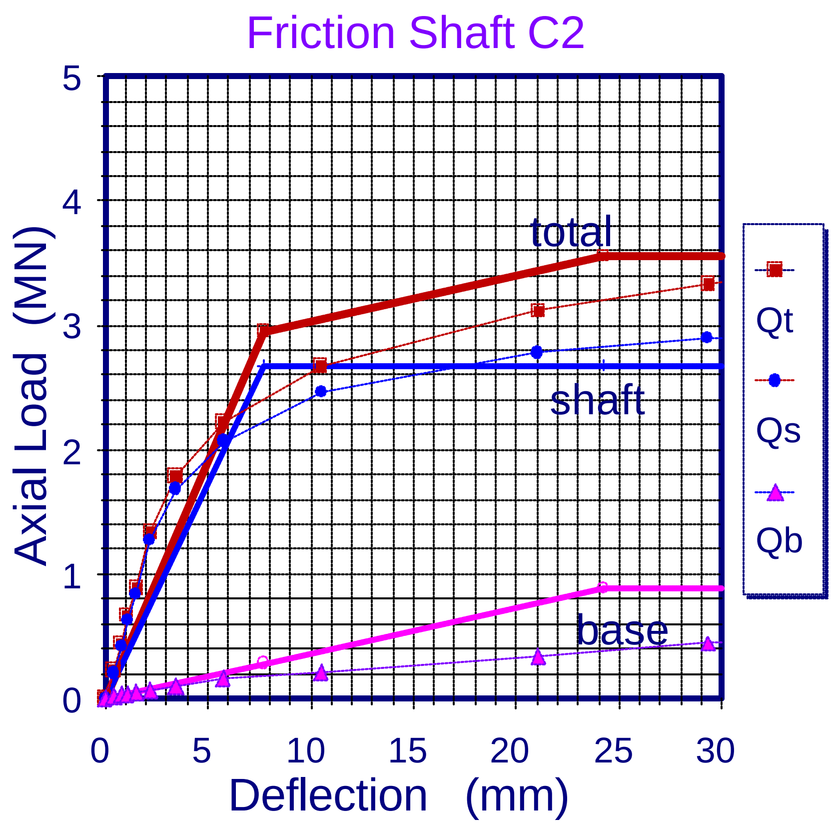


Figure 8. Load-Displacement Response of the Floating Shaft C2.
Note: Measured (dots) and Predicted (Lines).

CONCLUSIONS

The drained stiffness of Piedmont silts and sands at working loads can be reasonably characterized by equivalent elastic moduli obtained directly from flat dilatometer tests. Evaluations of the axial load-displacement response of drilled shafts in these residual soil materials are afforded by elastic continuum solutions, available in either chart or analytical closed-form expressions. Case studies involving two test shafts in Atlanta are used to illustrate the approach.

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