Evaluating O-cell response from seismic cone tests

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ABSTRACT
Elastic continuum solutions can be used to represent the responses of both upper and lower segments of drilled shafts subjected to Osterberg load testing. For site investigation purposes, the seismic piezocone test obtains a rather complete soil characterization since the penetration readings can be used to evaluate side and base capacities, while the shear wave velocity provides a fundamental stiffness for displacement analyses. A case study involving a three-staged O-cell test for the Cooper River Bridge in Charleston, South Carolina is presented to illustrate the approach.

RÉSUMÉ
Des solutions élastiques de continuum peuvent être employées pour représenter les réponses des segments supérieurs et inférieurs des axes forés soumis à l'essai de charge d'Osterberg. Pour la recherche d'emplacement, l'essai séismique de piezocone obtient une caractérisation plutôt complète de sol puisque les lectures de pénétration peuvent être employées pour évaluer des capacités latérales et basses, alors que la vitesse de vague de cisaillement fournit une rigidité fondamentale pour des analyses de déplacement. Une étude de cas impliquant un essai trois-par étapes d'O-cellule pour le pont de fleuve de tonnelier à Charleston, SC est présentée pour illustrer l'approche.

1 INTRODUCTION

The Osterberg load cell provides an alternative to conventional static load testing methods that rely on either large load frames or cumbersome reaction beams with anchor piles. The O-cell uses a minimal space in its configuration, essentially the same space taken by the drilled shaft foundation itself (Osterberg 1998, 2000). The original O-cell design consisted of an inflatable and sacrificial hydraulic jack that was situated at the base of the drilled shaft (O'Neill, et al. 1997). After the concreting and curing, the jack is pressurized to force the base downward (to measure end bearing resistance) while simultaneously pushing the shaft component upward (to measure side resistance). The shaft and jack are fully-instrumented to measure load-displacement response and axial load transfer during pressurization (Fellenius, 2001). After the load testing phase, the O-cell is grouted up to become part of the completed foundation.

In later design arrangements, the O-cell has been embedded at mid-section elevations with the shaft in order to juxtapose the opposing capacities for the downward segment versus the upperward segment. In fact, multiple levels of O-cells can be utilized to stage test different length segments of the drilled shaft foundation, as well as ascertain the base component. The results of these phased loading segments are combined to form a top-down equivalent load-displacement-capacity of the foundation for the axial compression mode.

2 ELASTIC CONTINUUM FRAMEWORK

Elastic continuum theory provides a simple and rational framework for representing the axial load-displacement behavior of piles (Poulos, 1989). The analytical model of Randolph & Wroth (1978, 1979) is particularly attractive as the solutions can be handled by manual calculation, spreadsheet (e.g., Excel or Quattro), mathematical software (e.g., Mathcad, Matlab), or compiled computer program (e.g., Piglet, Defpig). The normal solutions are presented for displacements and load transfer under axial compression loading (Fleming, et al. 1992). Yet, as the Randolph formulation was originally derived by simply combining a soil-supported shaft with a circular plate, the method lends itself to the evaluation of O-cell results where side and base resistances are measured directly.

The solution for a mid-level O-cell arrangement using rigid pile segments within the elastic continuum is given in Figure 1 (Mayne & Woeller, 2008). If desired, additional considerations can be given towards the consideration of pile compressibility, soil modulus increasing with depth, and belled bases (Fleming, et al. 1992).

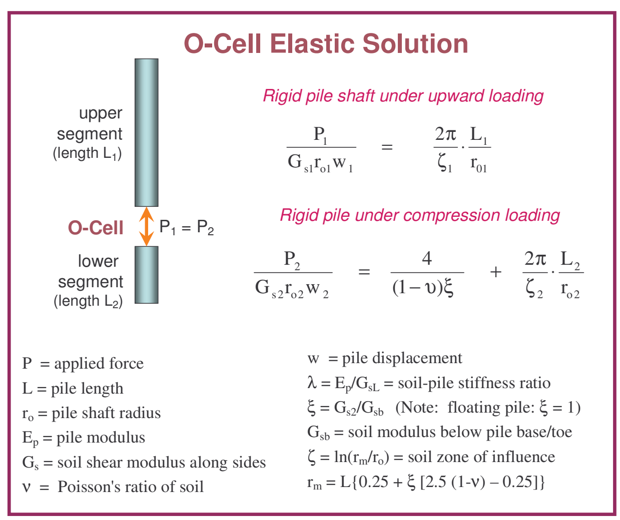

Figure 1. Elastic solution for mid-shaft O-cell position

3 COOPER RIVER BRIDGE CASE STUDY

The newly-completed Arthur Ravenel Bridge over the Cooper River in Charleston, SC was supported by large drilled shaft foundations having diameters of 2.5 to 3 m and embedded lengths of 50 to 60 m. The cable-stayed bridge connects the city of downtown Charleston with Drum Island and Mount Pleasant. The foundation soils in the area consists of the very stiff Cooper Marl lying at elevations generally below -15 to -20 m MSL that is used to support large bridges, buildings, port, and dock facilities (Camp, 2004). Above the marl, the overlying soils are comprised of soft Holocene clays, loose sands, silts, and other geomaterials from both marine, deltaic, and alluvial processes, all quite variable at location to location in the coastal areas.

3.1 Cooper Marl

The Cooper marl is a marine deposit consisting of stiff sandy calcareous clay of Oligocene age that has been overconsolidated by erosional processes as well as additional preconsolidation effects due to cementation. The stiff sandy clay has a fairly high calcite content on the order of 60 to 80 %.

Representative mean values of index water content parameters include: w_n = 48%, w_L = 78%, and PI = 38%. General characteristics of the Cooper Marl are reported by Camp et al. (2002) and summarized with ranges of indices in Figure 2. Typical SPT-N values in this material are in the range of 12 to 16 blows/0.3 m.

The marl is quite frictional in its behaviour. Separate series of isotropically-consolidated undrained triaxial compression tests (CIUC) on undisturbed samples of this calcareous clay by various geotechnical firms for both the new Arthur Ravenel Bridge in 2005 and 15-year older Mark Clark Bridge show consistently high effective stress friction angles $40^\circ \leq \phi' \leq 45^\circ$ for the Cooper Marl.

The Cooper marl is overconsolidated, having been preconsolidated by erosion and groundwater changes, as well as added structuration due to the calcium carbonate chemistry. A mean prestress $\Delta\sigma_v'$ = 480 kPa appears appropriate, or alternatively $\Delta\sigma_v'$ is now termed the overconsolidation difference: OCD = $\sigma_p' - \sigma_{vo}'$ (Locat, et al. 2003). This OCD has been shown reasonable for the stress history (Mayne, 2007a) when compared with consolidation data. The constant value of OCD gives a representative variation of preconsolidation stress (σ_p') that increases with depth, as well as a corresponding profile of overconsolidation ratio (OCR = σ_p'/σ_{vo}') which decreases with depth.

3.2 Load Testing Program

A fairly comprehensive load testing program of drilled shafts was performed at the site at the direction of the South Carolina Dept. of Transportation (Camp, 2004). This included axial and lateral O-cell arrangements to measure static capacities and displacement behavior. Series of Statnamics tests were also conducted, following the initial series of O-cell tests. Three test areas were established to represent general conditions in the Charleston city area, Drum Island, and Mount Pleasant.

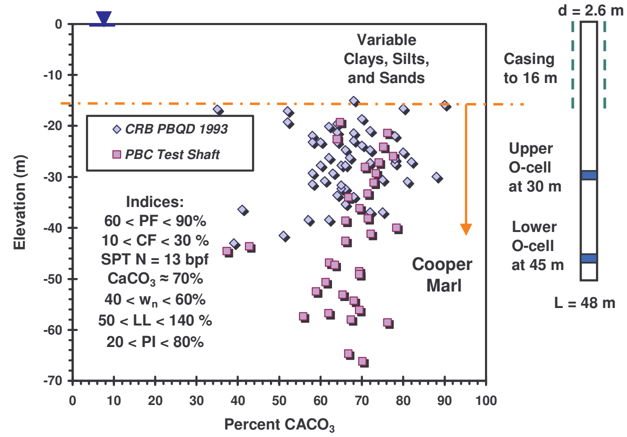

Figure 2. Cooper marl characteristics and MP-1 O-cell setup at Mount Pleasant test site.

Test setup for a representative drilled shaft load test MP-1 was established at the north end of the bridge in the Mt. Pleasant area. This bored pile had a constructed diameter d = 2.6 m and embedded length L = 48 m. The upper 16 m was cased with large diameter steel pipe to section off results strictly to the lower formation of the Cooper Marl. At MP-1, two levels of Osterberg hydraulic jacks were installed at approximate depths of 30 and 45 m to allow three-stage testing. The O-cell arrangement is depicted in Figure 2. The full-scale load testing of O-cells was performed by LoadTest of Gainesville, Florida.

The first stage of load testing involved pressurization of the lower O-cell resulting in a downward movement of the lower pile segment (d = 2.6 m; L = 2.53 m) while essentially no movement occurred in the upper shaft portions. The stage 1 involved base mobilization into the marl to evaluate end bearing resistance plus a small portion of side friction. The second stage involved pressurization of the upper O-cell with the lower cell ventilated (open). This stage 2 resulted in a downward motion of the mid-section shaft (d = 2.6 m; L = 14.0 m) with essentially no movements above the 30 m mark. Thus, the stage 2 is entirely shear dominated response due to mobilization of the side friction in the Cooper Marl. Finally, stage 3 was conducted by closing the lower O-cell and pressurizing the upper O-cell to move the top portion of the shaft (above elev. 30 m) upward. Essentially no displacements were recorded in the lower pile portions (below -30 m). The stage 3 data provided information on the shear resistance in the Cooper marl in the non-cased zone from -16 to -30 m interval. Results from all three stages of testing are presented in Figure 3.

In addition to test pile MP-1, a total of 14 O-cell tests were completed for this project. Full details are given by Camp (2004).

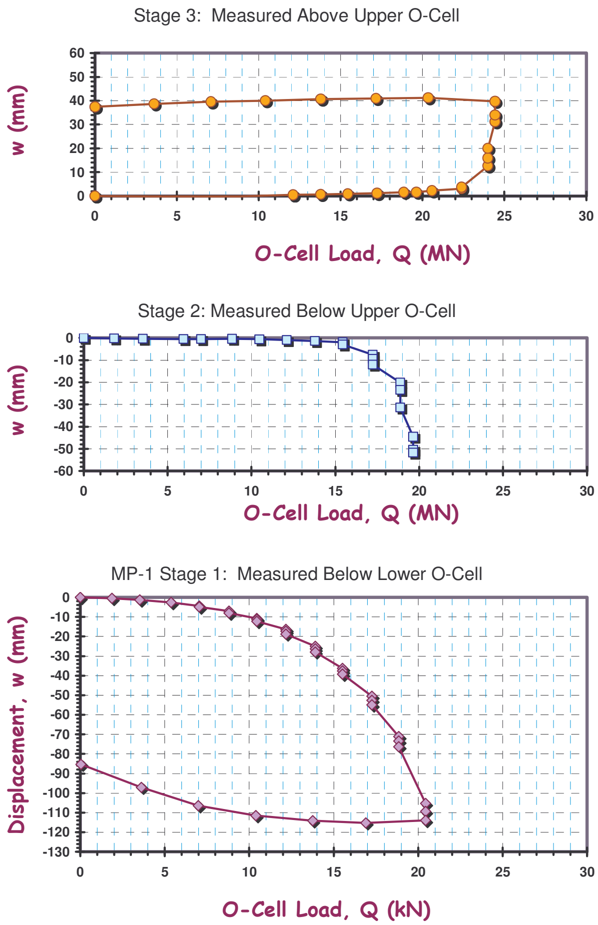

Fig. 3. Results from three-stages of loading on the MP-1 O-cell arrangement (two hydraulic jacks).

3.3 Seismic Piezocone Tests

The field site characterization program for the bridge over the Cooper River was carried out via a series of soil test borings with drilling, sampling, and standard penetration testing (SPT) and cone penetration tests (CPT). Required depths of exploration were generally 55 to 60 m. This took approximately 1.5 to 2 days for each rotary drilled boring, while only 3 to 4 hours for a given CPT.

In particular, the CPT tests were completed by ConeTec Investigations using a type 2 penetrometer having a porous filter element at the shoulder to measure penetration porewater pressures and a geophone arrangement to record downhole type shear wave measurements. The test is termed a seismic piezocone penetration test (SCPTu) and provides up to five readings on soil behaviour in a single sounding, including: cone tip resistance (q_t), sleeve friction (f_s), porewater pressures (u_2), time rate of decay of pressures to 50% consolidation (t_{50}), and shear wave velocity (V_s). For the bridge project, approximately 45 borings and 55 SCPTu soundings were completed.

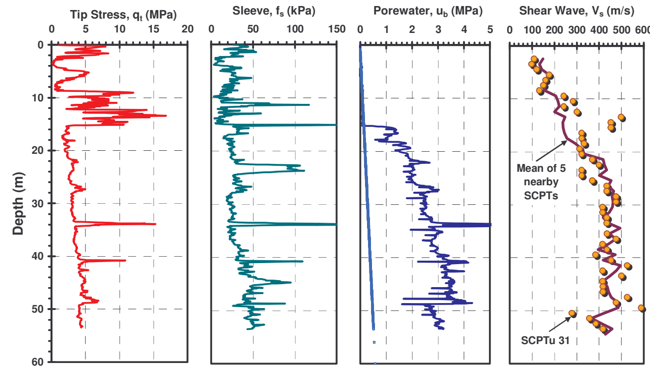

Figure 4. SCPTu-31 at the Mount Pleasant Shaft MP-1

At the specific MP-1 test location at Mount Pleasant, sounding SCPTu-31 was performed with the continuous penetrometer readings (q_t, f_s, and u_2) and the intermittent V_s data at one-meter intervals presented in Figure 4. The subsurface elevation indicating the top of the Cooper marl is quite evident at 20 m where the induced porewater pressures jump quite high.

3.4. Pile Side and Base Resistances

The penetrometer data can be utilized either directly or indirectly to assess the end bearing and side capacity of drilled shafts. Rational approaches (which are sometimes termed "indirect CPT methods) are the conventional procedures which use a static equilibrium for calculating unit side friction (f_p) and limit plasticity theory for evaluating base resistance (q_b) at the toe or pile tip (e.g., Kulhawy, et al. 1983). In the rational approach, the CPT data are first interpreted to provide soil engineering parameters (i.e., K_0, ϕ', s_u, OCR, etc.) that are then input into theoretical equations for f_p and q_b. In addition, a number of direct CPT methods have been developed that are applicable to either driven piles (e.g., Almeida, et al. 2001) or drilled shafts and bored piles (e.g., Poulos, 1989). Of recent vintage with the modern electronic piezocone, all three readings from the CPTu can be used for capacity determination (e.g., Eslami & Fellenius, 1997; Takesue, et al. 1998; Mayne 2007b).

For the end bearing resistance of piles in clays, limit plasticity solutions detail that:

$$q_b = N_c\, s_u \qquad [1]$$

where N_c = bearing factor (N_c = 9.33 for circular pile) and s_u = undrained shear strength. Presumably, the latter is a mode corresponding to direct simple shear (DSS), as the failure mechanism beneath the pile tip would extend in all directions away from the pile. In that case, the DSS strength can be given obtained from:

$$s_u = \tfrac{1}{2} \sin\phi' \cdot OCR^{\Lambda} \cdot \sigma_{vo}' \qquad [2]$$

where σ_{vo}' = effective overburden stress. The CPT data can provide the relevant OCR profile via a hybrid model derived from spherical cavity expansion and critical-state soil mechanics (Mayne, 2005), which in simplified form gives:

$$OCR = \tfrac{1}{3} Q \qquad [3]$$

where $Q = (q_t-\sigma_{vo})/\sigma_{vo}'$ = normalized cone tip resistance.

For the Cooper marl, it can be seen (Fig. 5) that the derived OCRs from the reference one-dimensional consolidation tests on undisturbed samples decrease from 4 at z = 20m to about 2 at z = 60 m. The CPT-interpreted profile is seen to be higher that the consolidation tests. Yet, it is well-recognized that sample disturbance effects tend to lower the apparent value of preconsolidation stresses obtained from laboratory tests (e.g., Lacasse et al. 1985).

The effective stress friction angle of soils can be evaluated by consideration of an effective stress limit plasticity solution developed at the Norwegian Institute of Technology, or NTH (Senneset, et al. 1989). A simplified form of this can be expressed (Mayne, 2005):

$$\phi' = 29.5\, B_q^{0.121}\, [0.256+0.336B_q + \log Q] \qquad [4]$$

where $B_q = (u_2-u_0)/(q_t-\sigma_{v0})$ = normalized excess porewater pressure and the following ranges apply: $20^\circ \le \phi' \le 45^\circ$ and $0.1 \le B_q \le 1.0$.

The effective ϕ' for Cooper marl is quite high, on the order of 44º, perhaps making it one of the most frictional clays worldwide (Diaz-Rodriguez, et al. 1992). Figure 6 shows comparable results in ϕ' obtained by lab triaxial series and the CPTu method given by [4].

Combining results from [1] to [4] provides the rational calculated end bearing resistance (indirect CPT) in the Cooper marl. Using SCPT-31, Figure 7 shows that the q_b profile is seen to be in good agreement with the backfigured end bearing values from the available O-cell load tests in this formation.

From considerations of static equilibrium, the side resistance (f_p) can be expressed in terms of the lateral stress coefficient (K_0) and interface friction between the pile surface and the surrounding soil. As a first approximation, this "beta" method approach gives:

$$f_p \approx K_0\, \sigma_{vo}' \tan\phi' \qquad [5]$$

In consideration of pile material type and installation (Kulhawy, et al. 1983), the expression in modified to:

$$f_p \approx C_m\, C_k\, K_0\, \sigma_{vo}' \tan\phi' \qquad [6]$$

where C_m = interface factor (= 1 for drilled shafts, 0.9 for driven concrete piles, 0.8 for timber, and 0.7 for rusty steel piles) and C_k = installation factor (= 1.1 for driven piles and 0.9 for bored piles). The at-rest coefficient can be estimated as:

$$K_0 = (1-\sin\phi')\, OCR^{\sin\phi'} \qquad [7]$$

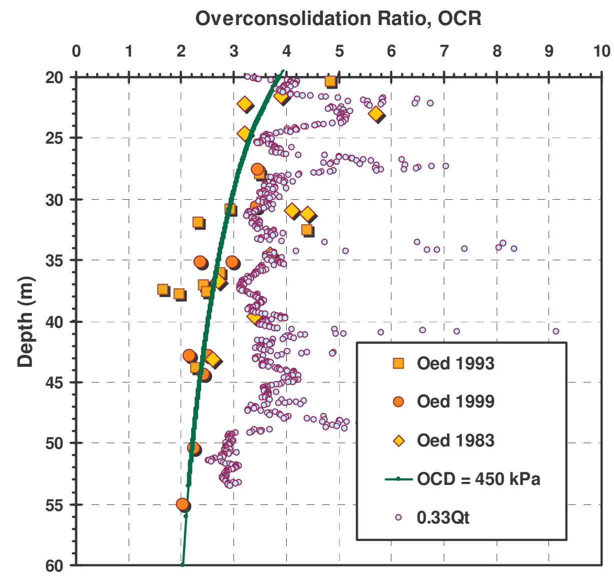

Figure 5. OCR profile in Cooper marl formation.

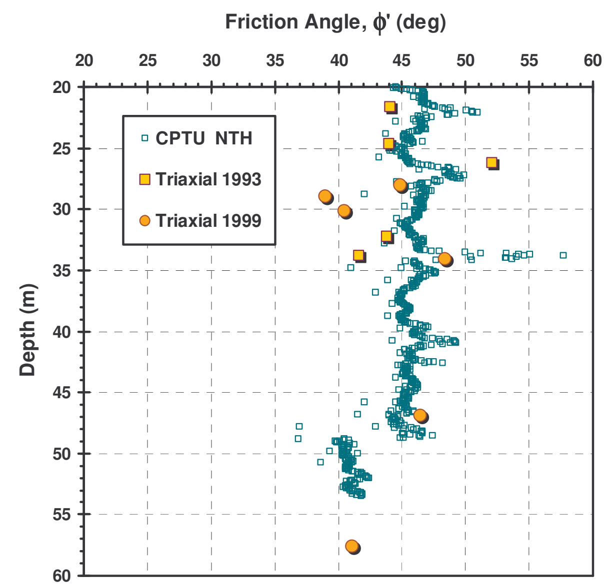

Figure. 6. Effective friction angle of Cooper marl.

Combining [3] to [7] provides the interpreted unit pile side resistance for drilled shaft foundations in clay from CPT. As evidence by Figure 8, the CPTu-derived fp values are in reasonable agreement with the backcalculated resistances from the O-cell load tests.

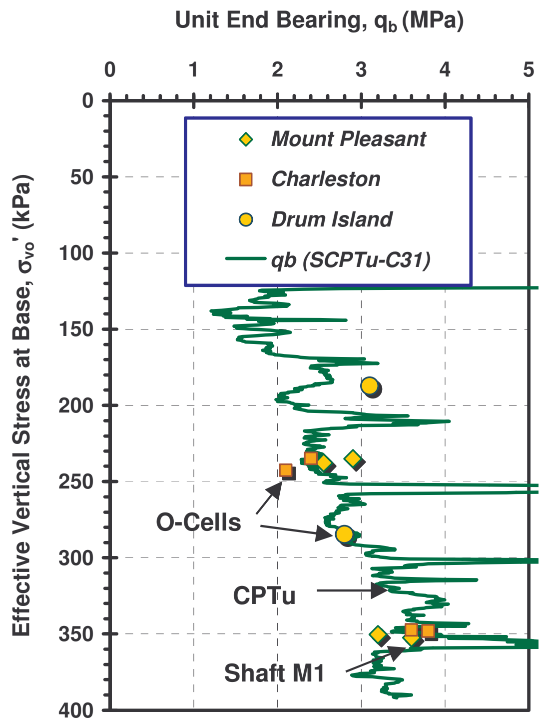

Figure 7. End bearing resistances in Cooper marl

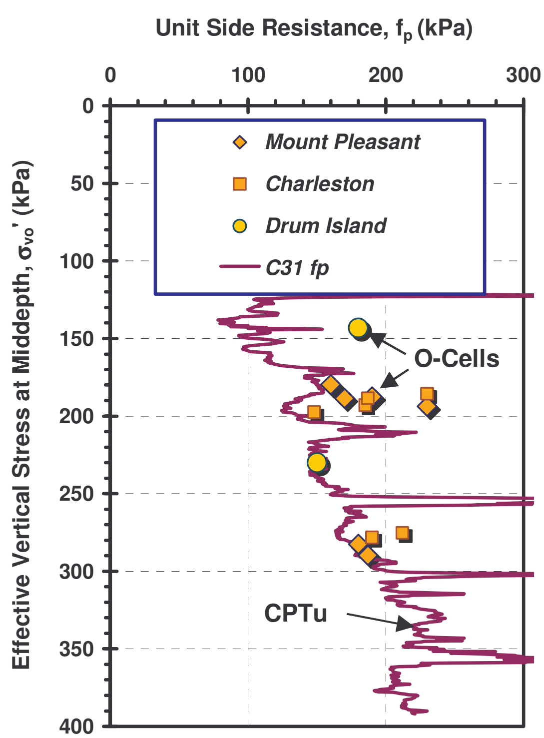

Figure 8. Side resistances in Cooper marl formation

4 LOAD-DISPLACEMENT RESPONSE OF O-CELLS

The elastic continuum framework presented in Figure 1 can now be implemented in conjunction with the capacities of side and base components. In this case, the fundamental stiffness of the ground is obtained from the shear wave velocity measurements:

$$G_{max} = \rho_T V_s^2 \qquad [8]$$

where ρ_T = total mass density of the soil. This is within the true elastic region of soil corresponding to nondestructive loading, thus P = 0. In order to account for nonlinearity of the stress-strain-strength behaviour of soils, a modified hyperbolic form is adopted (Fahey, 1998):

$$G = G_{max} [1 - (P/P_{ult})^g] \qquad [9]$$

where P = applied force, P_{ult} = capacity of the pile segment, and the exponent "g" is a fitting parameter (Mayne, 2007a). Data from torsional shear and triaxial shear tests on clays and sands under drained and undrained loading have been compiled to evaluate the fitting algorithms useful for nonlinear modulus trends. Figure 9 show data for a variety of soils (Mayne 2007b). The y-axis (G/G_{max}) can be considered as a modulus reduction factor to apply to the measured small-strain stiffness attained by [8] using site-specific V_s field data. The x-axis (q/q_{max} = 1/FS) can be considered the reciprocal of the factor of safety (FS) corresponding to the load level relative to full capacity.

The modified hyperbola given by [9] can be seen to take on values of "g" exponent ranging from 0.2 (low) to 0.5 (high). A representative g = 0.3 has been suggested for insensitive and nonstructured clays and uncemented quartz sands.

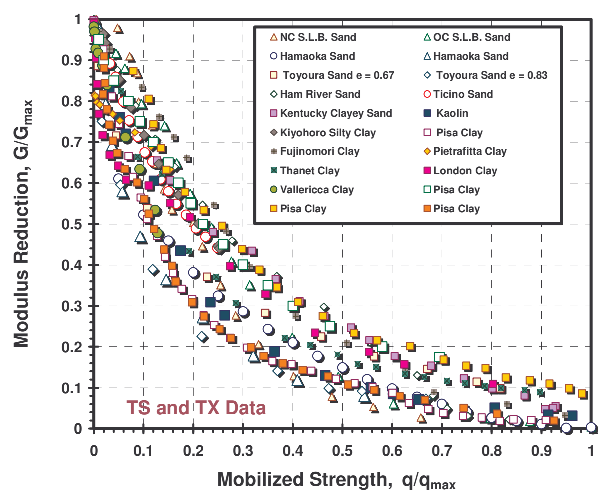

Figure 9. Lab trends of modulus reduction factor with mobilized shear strength of clays and sands.

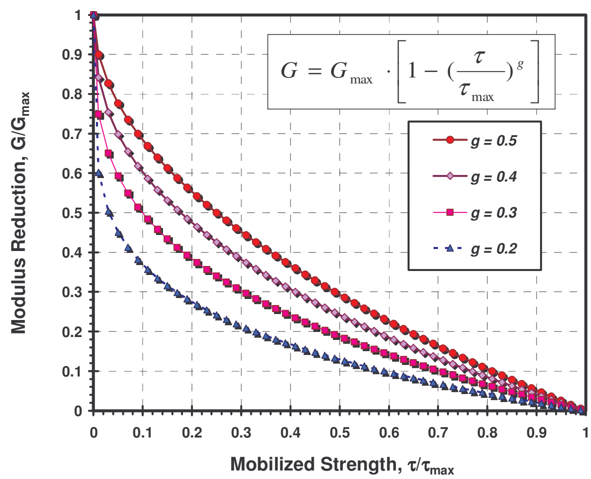

Figure 10. Modified hyperbola form for modulus reduction

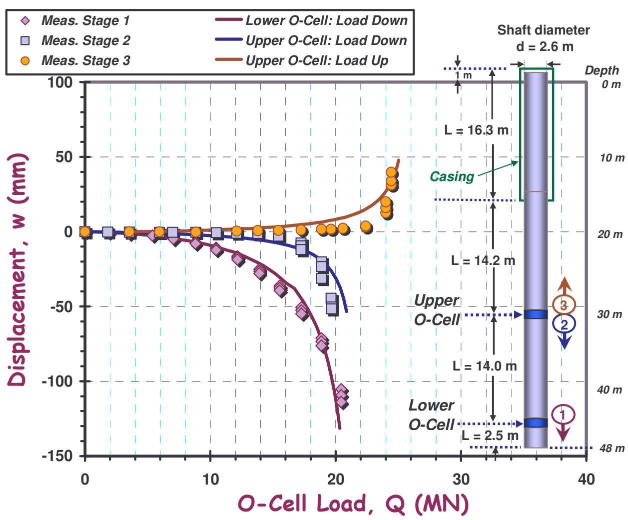

Figure 11. Elastic continuum pile applied to three-stage O-cell load tests of shaft MP1 in Mount Pleasant, SC

For the Cooper marl, the high calcium carbonate content would suggest a structured geomaterial, thus an appropriate exponent "g" = 0.5, or perhaps higher.

Using the calculated side friction ($f_p \approx 185$ kPa) and base resistance ($q_b = 3500$ kPa) given by the CPTu data together with the shear moduli obtained from the V_s profile, the pile segments for the O-cell arrangements can be represented by the simple elastic continuum equations given in Figure 1. The final curves are depicted in Figure 11 for all three stages of loading on test shaft MP-1. A reasonable agreement is observed for all loading stages of both O-cell jacks. A similar set of results can be done using the more complex expressions for a compressible pile, but the curves are quite similar.

5 CONCLUSIONS

The upward and downward pile segments of an O-cell load test can be adequately & conveniently represented by a simple elastic continuum solution. The results of seismic piezocone testing provides an advantageous collection of data to ascertain the axial side and base resistances of the deep foundations, as well as the shear wave velocity for the fundamental small-strain stiffness needed in the deformation analysis. A case study involving a two-level O-cell arrangement for a large drilled shaft in the calcareous Cooper marl was presented to illustrate the application.

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