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O-CELL RESPONSE USING ELASTIC PILE AND SEISMIC PIEZOCONE TESTS

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SUMMARY: The responses of the individual base and side components of O-cell load tests on drilled shaft foundations can be evaluated within an elastic continuum framework using results from seismic piezocone tests (SCPTu). The SCPTu is an optimal means for collection of geotechnical data because the same sounding can provides information on soil behavior at opposite ends of the stress-strain-strength curves, namely the peak strength for capacity and the small-strain stiffness (G_{max}) for the initial deformations. Using a Randolph-type elastic pile model, a case study involving axial shaft response in stiff clay till is presented.

Keywords: clay, drilled shafts, in-situ testing, load test, shear wave, stiffness

INTRODUCTION

The Osterberg load cell (O-cell) is an innovative and convenient means for mobilizing both the axial side and base resistance components of drilled shaft foundations[1]. The O-cell does not require a cumbersome reaction frame or anchor pilings as with conventional pile load test setups. Instead, it utilizes a novel (and sacrificial) hydraulic jack that is embedded within the bored pile at a specified vertical elevation[2]. The O-cell is placed in the drilled shaft foundation during the installation of the rebar cage and then concreted in-place. The hydraulic jack is inflated using a high-pressure pump, thereby lifting the one shaft section upward while simulaneously pushing the other shaft section downward. The results are evaluated to obtain the mobilized side and base components, as well as an equivalent top-down curve for the axial load-displacement-capacity response of the bored pile[3]. In the original design setup, the O-cell was positioned at the base of the bored pile. In later scenarios, the O-cell can be installed at any convenient elevation within the drilled shaft in order to provide comparable forces in upwards and

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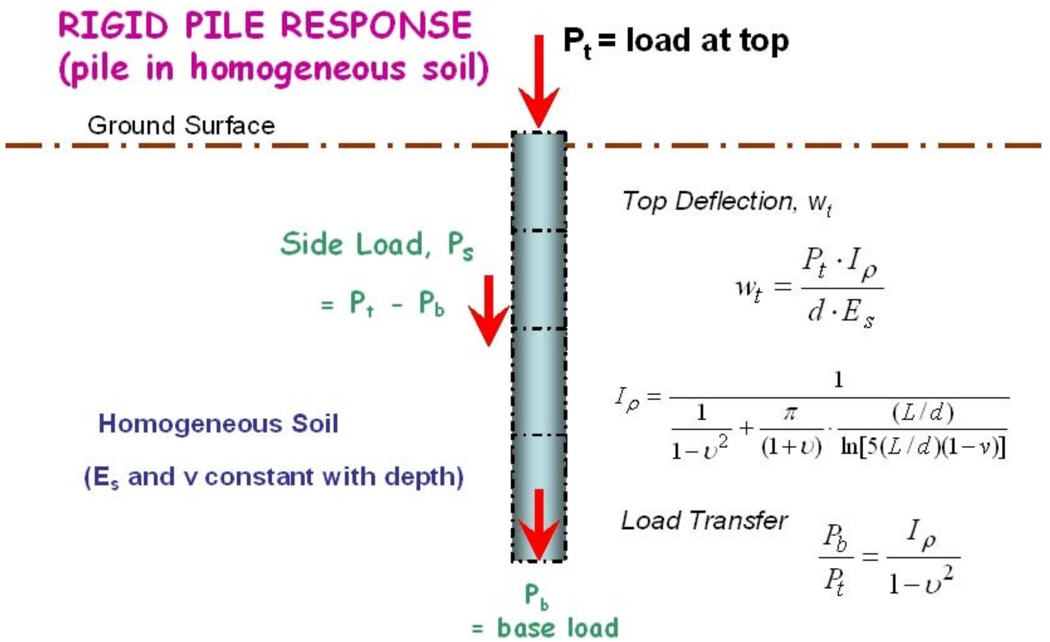


Fig. 1. *Rigid pile displacements in elastic soil continuum using Randolph-Wroth solution.*

downwards directions and fully mobilize resistances. Also, multiple levels of O-cells can be installed to stage-load the bored pile, thereby achieving huge capacities during axial load testing[4].

ELASTIC CONTINUUM PILE

The axial load-displacement response of piles can be evaluated within an elastic continuum solution[5]. For a rigid pile of length L and diameter d which is embedded in an an elastic soil medium having an equivalent modulus E_s and Poisson's ratio ν, the vertical displacement (w_t) under an applied load P_t is given by:

$$w_t = \frac{P_t \cdot I_p}{d \cdot E_s} \tag{1}$$

where I_p = displacement influence factor given by the elasticity solution. For rigid piles, the value of I_p depends simply upon the slenderness ratio (L/d) and ν, as indicated by Figure 1. Poulos & Davis[5] develop the values of I_p using boundary element solutions, which are seen to be in good agreement with the closed-form analytical solution of Randolph & Wroth[6,7] shown in Figure 2. For the latter, the influence factor is simply:

$$I_\rho = \frac{1}{\frac{1}{1-\upsilon^2} + \frac{\pi}{(1+\upsilon)} \cdot \frac{(L/d)}{\ln[5(L/d)(1-\nu)]}} \tag{2}$$

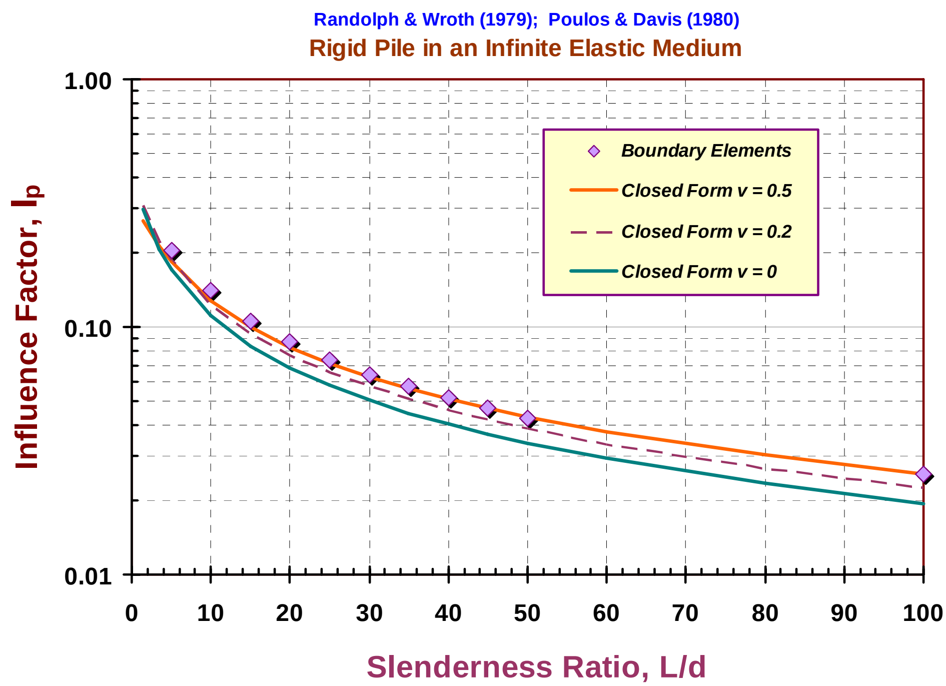


Fig. 2. *Displacement influence factors from elasticity solutions for rigid axially-loaded pile.*

Additional facets of the deep foundation construction can also be considered, including presence of a lower stiffer soil or rock layer, soil modulus variation with depth, and relative soil-pile compressibility[8].

In the conventional top-down axial loading of deep foundations, most often a majority of the applied load is transferred in side shear and that a smaller proportion of the load reaches the toe or base. In fact, elastic continuum theory provides a rational framework for evaluating the amount of load transfer. In the simplest arrangement, the O-cell hydraulic jack is located at the base, therefore the side shear segment and lower circular base plate of the Randolph-Wroth model can be treated separately. Alternatively, the O-cell can be placed within the a lower mid-section elevation of the drilled shaft, in order to optimize the degree of mobilization for both components[4]. In that case, the upper and lower portions can be considered as separate pile segments, as shown later.

AXIAL CAPACITY OF DRILLED SHAFTS

The axial capacity of deep foundations can be evaluated from methods based in static equilibrium, limit plasticity, or cavity expansion theory, or the directly from the results of in-situ tests. A review of selected and various methods is given elsewhere[9,10].

Table 1 lists three relatively recent direct methods for pile capacity evaluation from piezocone plus one beta-method for drilled shafts. The UNICONE, KTRI, and CUFAD methods apply to both driven piles and drilled or bored piles in all soil types, whereas the NGI-BRE method was actually developed for jacked and driven piles in clays. Details on the calculation procedures for these CPT-based methods are reviewed in the 2007 *Synthesis on Cone Penetration Testing*[11].

Table 1. *Selected pile capacity methods using cone penetration test results*

Method	Reference	Input Parameters	Remarks
UNICONE	Eslami & Fellenius[12]	Uses effective cone tip resistance q_t-u_2 and sleeve friction f_s	Based on 102 load tests on driven and drilled piles in soils
NGI-BRE	Almeida et al.[13]	Net cone tip resistance (q_t-σ_{vo}) for f_p and q_b	For driven and jacked piles in clay
KTRI	Takesue, et al.[14]	Uses f_s and Δu_2 to obtain f_p for driven and drilled piles	Applies to all soils, yet evaluates only unit side resistance
CUFAD	Kulhawy et al.[15]	OCR and ϕ' evaluations from CPT for f_p and q_b	Applies to drilled shaft foundations

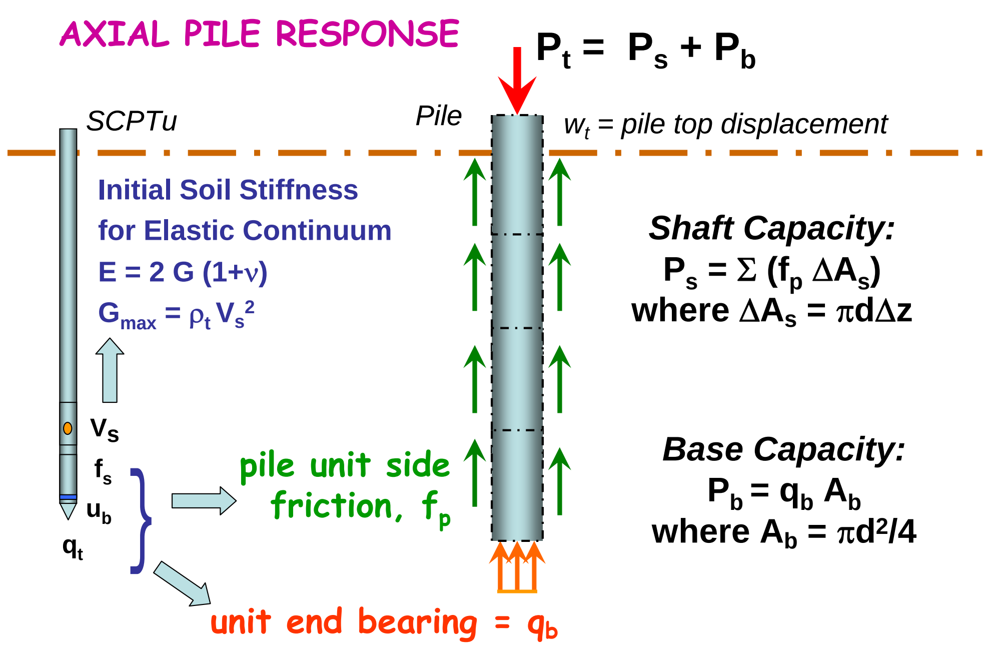


Fig. 3. *Application of seismic piezocone results for evaluating axial drilled shaft response.*

Of particular value in geotechnical site characterization for foundation systems is the seismic piezocone test (SCPTù) as it provides four separate readings on soil behavior with depth from a single sounding[11]. The SCPTu obtains profiles of the cone tip resistance (q_t), sleeve friction (f_s), penetration porewater pressures at the shoulder (u_2), and shear wave velocity (V_s). The SCPTu test results allow for an opportunity for capacity analyses by both direct and indirect in-situ methods (Figure 3). Moreover, the V_s profile provides the small-strain stiffness ($G_{max} = \rho_t V_s^2$) which is the beginning of all stress-strain-strength curves in geomaterials, where $\rho_t = \gamma_t/g_a$ = total soil mass density, γ_t = total unit weight, and $g_a = 9.8$ m/s^2 = gravitation constant.

NONLINEAR SOIL STIFFNESS

The stiffness of geomaterials is highly nonlinear over many orders of scale over its range in logarithmic shear strains. The initial small-strain shear modulus ($G_0 = G_{max}$) is fundamental and can be considered a state parameter corresponding to the in-situ geostatic conditions. As G_{max} is within the true elastic region of soil behavior corresponding to nondestructive shear strains ($\gamma_s < 10^{-6}$), the value must be reduced to appropriate strain levels or stress levels applicable to the working load levels for utilization in the elastic continuum equations. One simple algorithm for this purpose is a type of modified hyperbola[16] whereby the reduction factor is given by:

$$G/G_{max} = 1 - (1/FS)^g \tag{3}$$

where $FS = P_{ult}/P$ = calculated factor of safety and g = exponent fitting parameter. Thus, as the working load P increases toward the capacity (P_{ult}), the modulus reduces accordingly. For uncemented and non-highly structured soils, values of the exponent parameter are generally observed to be $g \approx 0.3 \pm 0.1$ for many soils[11]. The shear modulus (G) can be readily converted to an equivalent Young's modulus (E) by the well-known elasticity relationship:

$$E = 2\,G\,(1 + \nu) \tag{4}$$

with a value of $\nu = 0.2$ taken appropriate for the small-strain region.

CALGARY TEST SITE

The construction of a new Foothills Medical Center (FMC) in Calgary, Alberta warranted the use of drilled shaft foundations for support of the building loads. The site is underlain by thin shallow fill and surficial sandy silt layers overlying a thick deposit of very stiff to very hard silty clay till. Index properties of the till include: natural water content (w_n) between 13 to 17%, liquid limit (LL) = 27%, plasticity index (PI) = 10%, and clay fraction (CF < 0.002 mm) varying between 5 to 22%. The site investigation program included soil borings with standard penetration testing (SPT), piezocone penetration tests (CPTu), and one seismic piezocone test (SCPTu). A representative soil profile and SPT resistances from one boring (BH-8) at the site is shown in Figure 4. The many SPT N-values between 30 and 60 blows/0.3 m indicate the very hard nature of the clay till bearing stratum.

The geotechnical design team initiated a load test program on a drilled shaft constructed within the clay till layer to confirm design capacities and performance. The shaft was built with a 14-m embedded length and diameter of 1.4 m having the top of the foundation located 8 m below the original ground surface in order to accommodate a basement level. The test shaft was outfitted with an O-cell at a mid-elevation position located approximately 4 m above its base. Additional details on the shaft construction and instrumentation are given in the LoadTest Report[17]. Referring back to Figure 4, the constructed reinforced concrete shaft for FMC is totally contained within the hard clay till stratum.

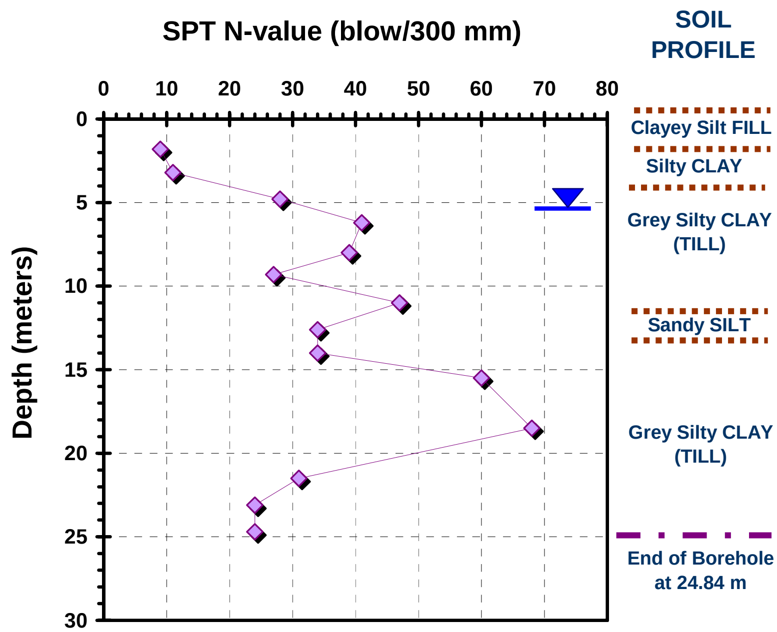


Fig. 4. *Soil profile and representative SPT resistances with depth at Calgary FMC.*

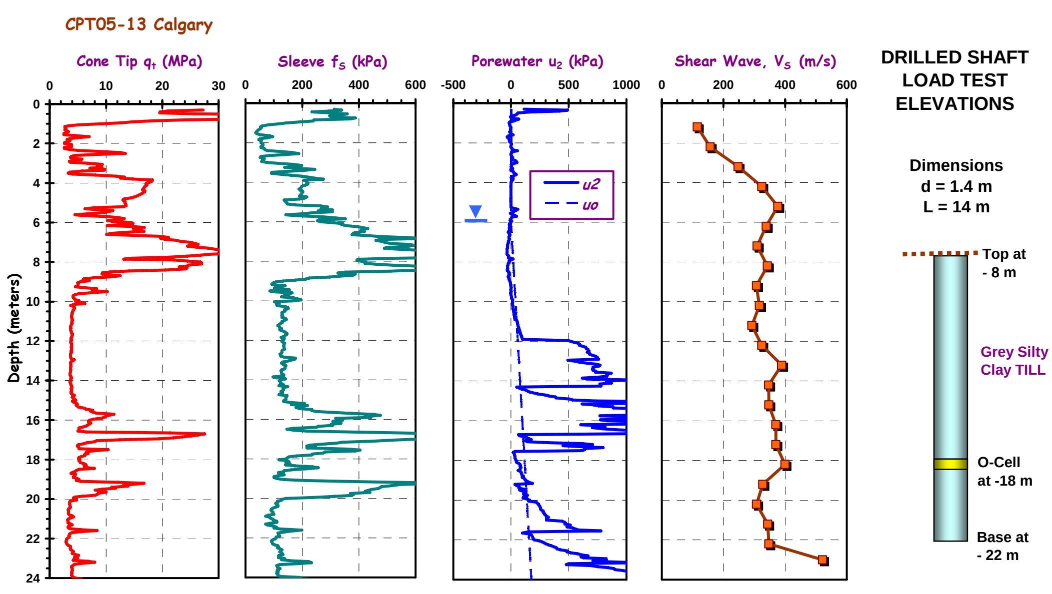


Fig. 5. *Seismic piezocone sounding and setup for O-cell load test for drilled shaft at Calgary FMC.*

The results of a seismic piezocone test at the Calgary FMC site are presented in Figure 5. As the conventional soil boring produces SPT N-values at approximate 1.5-m depth intervals, in contrast the SCPTu offers three continuous profiles of q_t, f_s, and u_2 with depth, plus downhole V_s data at 1-m intervals. The shear wave data provided an initial elastic modulus E_{max} = 537 MPa, assuming a homogeneous case.

O-CELL LOAD TEST RESULTS

During the O-cell load testing, a maximum sustained bi-directional loading of 5.37 MN was applied to the drilled shaft. The O-cell was outfitted with three linear variable wire displacement transducers (LVWDTs) to monitor displacements[17]. Vibrating wire type strain gages were attached to the sister bars to measure axial loads in the two pile segments. Results from the load test are presented in Figure 6. At the fully applied load, the maximum recorded displacements above and below the O-cell were 39.1 and 14.8 mm, respectively. The mean unit side resistance for the 10 m long pile segment loaded upward by the O-cell is backcalculated at f_p = 122 kPa. Using this value for the lower 4 m pile segment loaded downward gives an operational unit end bearing of q_b = 2.1 MPa.

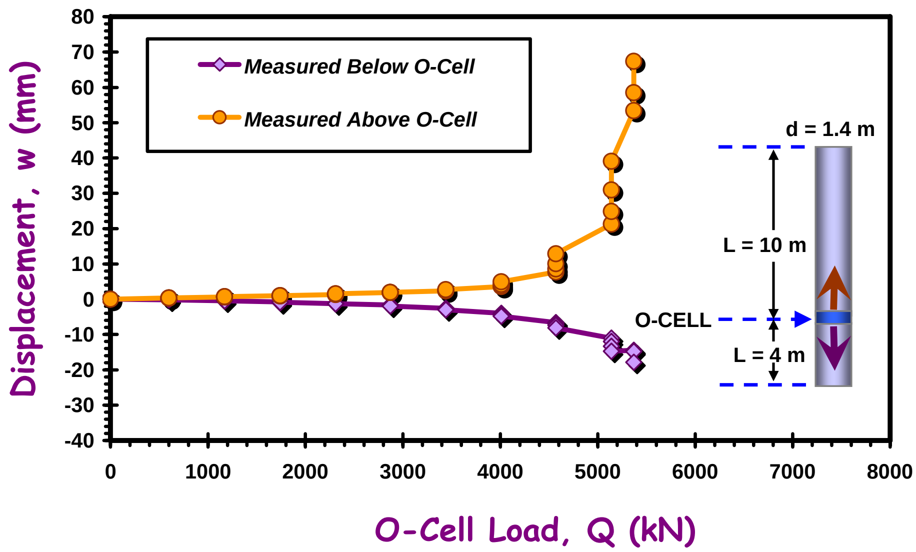


Fig. 6. *Measured load-displacement response during O-cell test on Calgary drilled shaft.*

The unit side shearing resistance for the Calgary FMC drilled shaft was evaluated by Elbanna et al.[18] using a variety of direct in-situ methods for both cone penetration tests (CPT) and flat plate dilatometer testing (DMT). In Figure 7, the selected CPT methods from Table 1 are presented and compared with the backfigured f_p from the O-cell loading over the depth range from 8 to 22 m. For the beta analysis, the effective friction angle was evaluated using the normalized cone tip resistance (Q) and porewater pressure measurements (B_q), based on the simplified NTH method[11]. A mean value of $\phi' = 37.7°$ for the till was determined in this manner. The OCR was evaluated from the global correlation with G_{max} and effective overburden stress (σ_{vo}') for varied soil types[19]. As such, the shear wave velocity was estimated from a method based on both the q_t and f_s profiles, as discussed by Hegazy & Mayne[20], which provided a very good agreement with the measured V_s profiles from the SCPTu. The corresponding G_{max} values gave an estimated average OCR ≈ 2.67 ± 0.55 by this approach. This allowed for a line-by-line evaluation, as shown in Figure 7. The resulting profiles of ϕ' and OCR for the clay till were then utilized to evaluate the lateral stress coefficient:

$$K_0 = (1 - \sin\phi')\, OCR^{\sin\phi'} \tag{5}$$

which gave calculated mean values of $K_0 = 0.70 \pm 0.05$ over the depths of interest. Using the beta method detailed by Kulhawy et al.[15], the unit side friction is obtained from:

$$f_p = C_M\, C_K\, K_0 \tan\phi'\, \sigma_{vo}' \tag{6}$$

where C_M = pile material coefficient (= 1.0 for drilled shafts) and C_K = lateral stress modifier for installation (= 0.9 for good quality drilled shaft construction). The calculation gave a mean f_p = 114 kPa for the Calgary FMC site.

As noted by Elbanna et al.[18], the KTRI method produces values too high (average f_p = 320 kPa) for this clay till, notably because of the limited database for which the KTRI correlations were developed. The NGI-BRE method gave somewhat high values for the till on the order of f_p = 188 kPa, yet as noted earlier, this method was developed for driven and jacked piles in clay, thus not truly applicable here. In contrast, the UNICONE method gave values less than the measured load tests with a mean estimated f_p = 80 kPa. By averaging three profiles (Beta, NGI-BRE, and UNICONE), a relatively good estimate of the unit side friction (f_p = 127 kPa) was obtained.

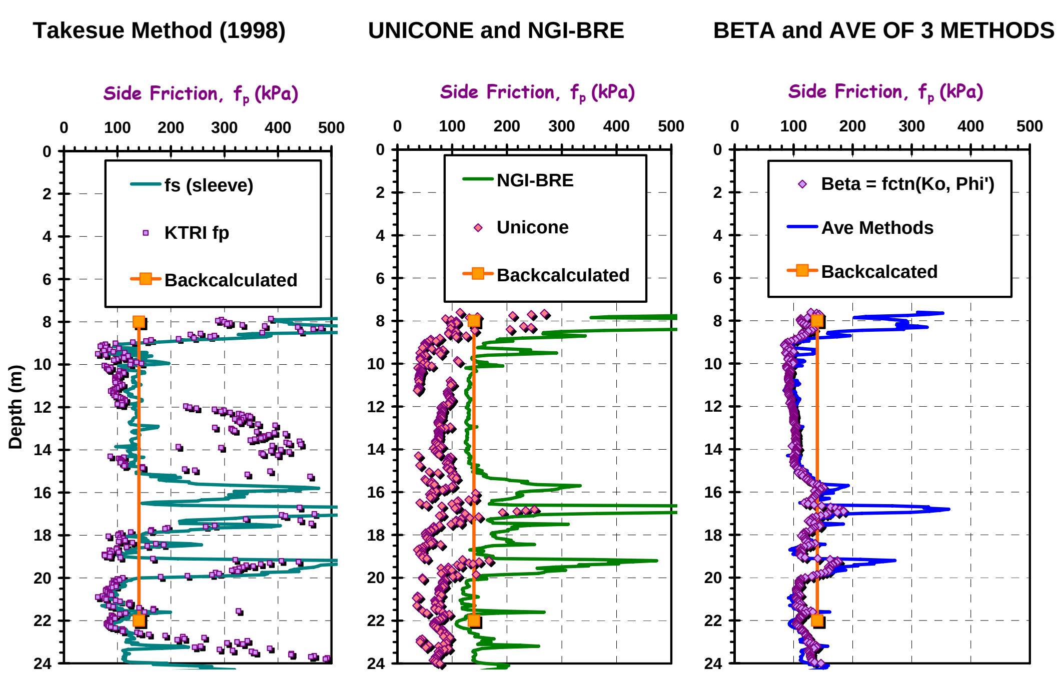


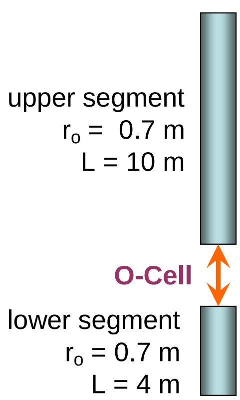
Fig. 7. *Calculated unit side friction resistances with measured values for Calgary drilled shaft.*

For the unit end bearing, there are three methods available for the calculation of q_b below the foundation base. These values are summarized in Table 2 and compared with the backfigured value from the O-cell results. Measured CPT data that are averaged about one diameter deep beneath the base elevation include: q_t = 3619 kPa, σ_{vo} = 448 kPa, and u_2 = 498 kPa. For the limit plasticity solution, the value of undrained shear strength was obtained for an equivalent direct simple shear (DSS) mode from $s_u = (q_t - \sigma_{vo})/N_{kt}$ with a representative value N_{kt} = 15. As seen from the results, the NGI-BRE and limit plasticity methods gave answers comparable to the load tests, whereas UNICONE gives a value too high using the usual adopted toe factor r_t = 1. Of course, better agreement for this clay till is found by adopting a site specific value r_t = 0.7.

Table 2. *End-bearing resistances for Calgary drilled shaft foundation*

Method	Reference	Resistance, q_b (kPa)	Notes/Remarks
Measured by O-Cell Testing	Kort (2005)	2100 kPa	Backfigured
UNICONE	Elsami & Fellenius (1997)	3121 kPa (for r_t = 1)	$q_b = r_t(q_t - u_2)$
NGI-BRE	Almeida et al. (1996)	2113 kPa (for k_2 = 1.5)	$q_b = (q_t - \sigma_{vo})/k_2$
Limit Plasticity	Kulhawy et al. (1983)	1972 kPa	$q_b = N_c\, s_u$ where N_c = 9.33

O-Cell Elastic Solution



Rigid pile under tension loading

$$\frac{P_t}{G_{sL} r_o w_t} = \frac{2\pi\rho_E}{\zeta} \cdot \frac{L}{r_o}$$

Rigid pile under compression loading

$$\frac{P_t}{G_{sL} r_o w_t} = \frac{4\eta}{(1-\upsilon)\xi} + \frac{2\pi\rho_E}{\zeta} \cdot \frac{L}{r_o}$$

L = pile length
r_o = pile shaft radius (r_b = base radius)
$\eta = r_b/r_o$ = 1 for shaft shaft
E_p = pile modulus
G_{sL} = soil shear modulus at z = L
$\rho_E = G_{sM}/G_{sL}$ = Gibson parameter

G_{sM} = soil shear modulus at mid-shaft
$\lambda = E_p/G_{sL}$ = soil-pile stiffness ratio
$\xi = G_{sL}/G_{sb}$
G_{sb} = soil stiffness below pile base
$\zeta = \ln(r_m/r_o)$ = soil zone size around pile
$r_m = L\{0.25 + \xi\,[2.5\,\rho_E(1-\nu) - 0.25]\}$

Fig. 8. *Elastic continuum solution for upper and lower pile segments of O-cell load test arrangement.*

PILE RESPONSE WITHIN ELASTIC MODEL

The results of the load tests can be represented within the framework of the elastic continuum solution. Using the rigid pile solution, the O-cell can be partitioned into two pile segments, as detailed in Figure 8. The results of the capacity analyses can be used to reduce the initial measured G_{max} = 224 MPa to an appropriate G for each fraction of applied load level (P/P_{ult}), or FS per equation (3). The resulting curves are seen to well match the measured and separate responses for the upward and downward pile segments from the load tests in Figure 9.

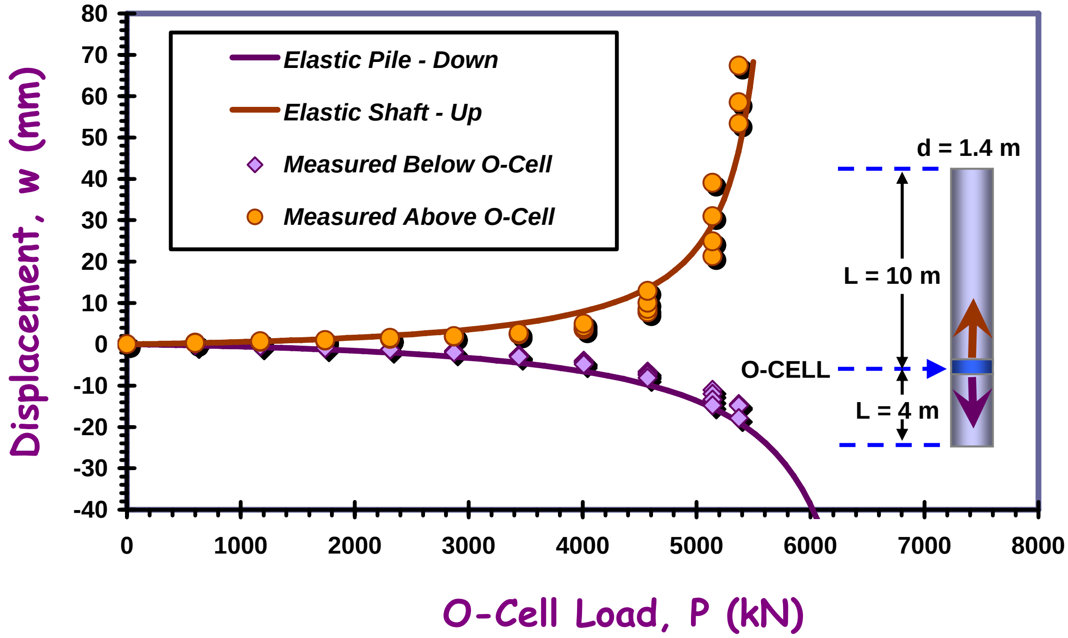


Fig. 9. *Elastic continuum solution applied to components of O-cell load test in Calgary.*

CONCLUSIONS

Elastic continuum solutions provide a rational framework for assessing and evaluating field load test results on axially-loaded pile foundations. Using the fundamental initial stiffness of geomaterials (G_{max}) within this context, an approximate nonlinear load-displacement-capacity representation can be afforded via a modified hyperbola to achieve intermediate stiffnesses of soil as the applied loads increase towards capacity. Results from an O-cell load test on a drilled shaft situated in very stiff to hard clay till in Calgary are utilized to illustrate application of the method. Geotechnical soil parameters for the analyses are conveniently obtained from seismic piezocone tests made at the site, including tip, sleeve, and porewater resistances for capacity and shear wave velocity for the initial small-strain shear modulus.

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