Recent developments and applications in geotechnical field investigations for deep foundations

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ABSTRACT. For routine site investigations, the use of hybrid tools such as the seismic piezocone and seismic flat dilatometer offer superior efficiency and economy to provide sufficient profiling of subsurface conditions and the evaluation of geoparameters, as compared with routine soil boring, drilling, and sampling. A single sounding provides up to five separate measurements on soil behavior that can be used to calculate capacity and displacements for axial and lateral pile foundation capacity, as well as in applications involving quality control for ground modification projects. Axial pile capacity can be assessed using traditional approaches that utilize limit plasticity and static equilibrium, or alternatively via direct in-situ testing methods. Load tests from a bridge project in Minnesota are presented to illustrate the applicability of seismic piezocone testing.

1. INTRODUCTION

Each and every civil & environmental project requires a geotechnical site investigation to determine the makeup of the underlying ground at that particular location. This can be accomplished today using a combination and assortment of geophysical methods, soil drilling & sampling for laboratory testing, and in-situ field testing.

There are currently many different types of deep foundation systems being used commercially in support of civil engineering works for building loads, bridge piers, ports, and tower structures both onshore and offshore. Pile types can include driven steel H and pipe (open-end versus closed-end), precast versus prestressed concrete, timber, composite, tapered, and monotubes, or systems of bored or augered deep foundations, such as cased or uncased drilled piers, slurry shafts, augercast pilings, and caissons, as well as specialty types (e.g., Fundex, Omega, Screw, Dewaal). Consequently, the notion that a single test number such as the SPT-N value can suffice for all the needs in the analysis and design of modern piling foundations must be replaced with a more rational belief that multiple measurements are paramount.

Herein, we shall explore the utilization of more modern tests, such as the seismic cone and seismic dilatometer, for the collection of several measurements concerning soil behavior. In addition to assessing soil stratigraphy and geoparameters in an efficient and economic manner, these tests permit an evaluation of the nonlinear axial load-displacement response of deep foundations.

1.1 Conventional Site Investigation

For the past century, the conventional approach to geotechnical subsurface investigation has been the use of rotary drilling, augering, and sampling methods to create boreholes. Most common, the use of open drive samplers permits the collection of small disturbed cylindrical soil specimens via the standard penetration test (SPT). This provides a crude index (N-value) that needs an important
correction for the energy inefficiency of various drop hammer systems in use since its debut in 1902, including: pinweight, donut, safety, and automatic hammers.

At the time that the geotechnical profession became aware of the energy efficiency issues (Seed et al. 1985; Skempton 1986), the average energy efficiency of SPTs in practice was approximately 60%, so this became the reference value for the correction, designated; \( N_{60} \), which is obtained:

\[
N_{60} = CE \cdot N_{measured} = \left(\frac{ER}{60}\right) \cdot N_{measured}
\]

where

- \( CE \) = correction factor for energy efficiency
- \( ER \) = energy ratio for the particular hammer system used (ASTM D 4633)
- \( N_{measured} \) = number of hammer blows to drive a split-spoon sampler a vertical distance of 300 mm (or 1 foot), reported as blows/0.3 m.

An illustration showing the importance of this correction is depicted in Figure 1 using SPT data from the National Geotechnical Experimentation Site (NGES) at Northwestern University (Finno et al. 2000). Here, the upper soils are comprised of clean fine sands \((0.15 \text{ mm} < D_{50} < 0.30 \text{ mm})\) that were subjected to two sets of SPTs in soil test borings using a safety hammer and an automatic hammer. Figure 1a shows the raw uncorrected \(N\)-values while Figure 1b presents the energy-corrected values. The significance and importance of the energy corrections on the \(N\)-values is quite evident.

![SPT profiles at the national test site at Northwestern University: (a) uncorrected \(N\)-values; (b) corrected \(N_{60}\) (data from Finno et al. 2000) ](image)

There is a general misconception by geoengineers that if the SPT is performed using an autohammer, the results do not need to be corrected for energy content. Recent studies have made compilations of energy measurements by different organizations at various sites in the USA, all using automatic hammers (Davidson et al. 1999; Honeycutt et al. 2014; Sjoblom et al. 2002). These ER data represent over 20,000 field measurements per ASTM D 4633 and indicate a documented range of ER from 45% to 95% for autohammer efficiencies. Therefore, one cannot assume a value of ER for a valid correction of energy on a particular system. Interestingly, the state-of-the-practice
using autohammers has now risen to a mean value ± one standard deviation of ER (ave) ≈ 82% ± 7% based on measurements taken in the past five years (Honeycutt et al. 2014).

1.2 In-Situ Test Methods

A variety of in-situ devices and probes have been devised for characterizing the subsurface conditions and providing quantification on the geotechnical engineering properties for pile foundation analysis and design. Figure 2 provides an overview that depicts the various instruments, penetrometers, blades, vanes, innovative devices, and inventions that are available. Certain of these tests are rather specialized towards assessment of a particular geoparameter (e.g., total stress cell for $K_0$ evaluation; ball penetrometer for assessing undrained shear strength in very soft soils).

Some new developments specific to geotechnical foundation design include:

(a) thermal conductivity dissipation tests (TCT) that use temperature-measuring penetrometers to assess the thermal properties of the ground for energy pile design, as detailed by Akrouch et al. (2016);
(b) cone load test (CLT) that derives modulus and stiffness data (i.e., t-z curves) for piling foundation design (Ali et al. 2010);
(c) an in-situ scour erosion evaluation probe (ISEEP) that provides field measurements on the potential for scour in sands;
(d) multi-piezo-friction penetrometer (MPFP) for measuring soil-pile roughness and quantification of soil-structure interaction effects on a site-specific basis (Hebeler and Frost 2006).
(e) parallel seismic method (PSM) for determining the unknown lengths of existing pile foundations, often needed in bridge replacement or retrofit projects (Niederleithinger 2012).

In contrast to these specifically-focused devices, some in-situ tests are quite versatile and can provide multiple measurements on soil engineering parameters during a single sounding, such as the seismic piezocone test (SCPTu) and seismic dilatometer test (SDMT), as depicted in Figure 3. These are both hybrid devices which combine downhole geophysics with a static penetrometer or probe that collect four to five independent readings with depth (Mayne and Campanella 2005).

![DIRECT-PUSH TECHNOLOGY](image)

**Fig. 3.** Hybrid in-situ geotechnical tests: seismic piezocone and seismic dilatometer

### 1.3 Seismic Piezocone Test

The SCPTu obtains the following measurements: \( q_t \) = cone tip resistance, \( f_s \) = sleeve friction, \( u_2 \) = dynamic porewater pressures, and \( V_s \) = shear wave velocity. All of the data are collected in the field by computer, either analog or digital format, and can be processed on-situ immediately and/or transmitted by wireless to the engineering office. The first three penetrometer readings are useful in assessing geostratigraphy, layering, and soil type, as well as provide estimated pile shaft friction.
and end-bearing resistances. Of additional value, the shear wave velocity provides a fundamental stiffness of the ground via:

\[ G_0 = G_{\text{max}} = \rho_t \cdot V_s^2 \]  

(1)

where \( G_0 \) = initial tangent shear modulus and \( \rho_t = \gamma_t/g_a = \) total soil mass density, \( \gamma_t = \) soil unit weight, and \( g_a = 9.81 \text{ m/s}^2 = \) gravitational constant. This is particularly important in calculations involving the evaluation of pile displacements and axial load transfer distributions along the pile length.

A representative SCPTu sounding performed in coastal plain sediments of Norfolk, Virginia is shown in Figure 4. The upper 9 m of soils are comprised of recent Holocene deposits with sandy and silty layers in the first 5 meters and a soft clay layer in the depth intervals from 5 to 9 m. From depths below 9 m, the sounding penetrates into the Yorktown formation which a Miocene age marine deposit composed of calcareous sandy clays to sandy clays. Generally, \( V_s \) readings obtained by standard downhole testing (DHT) are taken at 1-m depth intervals, as shown by the square dots in Figure 4d. Also presented in this figure are the results from special continuous interval \( V_s \) recordings using the GT autoseis source which can generate a wavelet every 1 s (Ku et al. 2013). This offers the opportunity for the field performance of the SCPTu to be as fast, productive, and efficient as regular cone penetration testing (CPT).

**Fig. 4.** Representative seismic piezocone sounding in Norfolk, Virginia, USA

1.4 Seismic Flat Dilatometer

In a similar manner, the seismic flat dilatometer (SDMT) can provide a comparable number of readings from a single sounding: \( p_0 = \) contact pressure, \( p_1 = \) expansion pressure, \( V_p = \) compression wave velocity, and \( V_s = \) shear wave velocity. If additional information is desired about the soil permeability and time rate of consolidation, then the SCPTu can also obtain a fifth reading, i.e. \( t_{50} \)
Example results from SDMT are presented by Marchetti et al. (2008) and Amoroso et al. (2014).

2 INTERPRETATION OF CONE PENETRATION TESTS

2.1 Geoparameter Evaluation from CPTu

The interpretation of cone and piezocone penetration tests can be made on the basis of theoretical, analytical, numerical, empirical, and statistical relationships (Lunne et al. 1997; Mayne 2007; Schnaid 2009). Efforts at calibration of the interpretative CPT procedures rely on one or more of the following: (a) matching field results with benchmark values obtained from laboratory tests on undisturbed samples, (b) backcalculating parameters from full-scale foundation performance, (c) 1-g model testing in chambers; and (d) centrifuge testing using mini- and micro-penetrometers. Advantages and shortcomings are associated with each of these approaches.

For evaluating the axial capacity of deep foundations, the CPTu must evaluate the following geoparameters: (a) soil type; (b) unit weight; (c) effective friction angle; (d) stress history; (e) undrained shear strength; and (f) lateral stress coefficient. Moreover, in assessing pile displacements and axial load distributions, the small strain shear modulus \( G_0 = G_{\text{max}} \) plays an important role. Each of these are briefly discussed in subsequent sections, as restricted to uncemented sands and clays of low to medium sensitivities.

2.2 Soil Behavioral Type

Since soil samples are not normally collected during CPT, indirect methods for soil classification are often used. These generally include: (a) approximate "rules of thumb"; (b) soil behavioral type charts; and (c) probabilistic methods. The approximate rules suggest that sands are identified when \( q_t > 5 \text{ MPa} \) and \( u_2 \approx u_0 \), whereas intact clays occur when \( q_t < 5 \text{ MPa} \) and \( u_2 > u_0 \) (Mayne et al. 2002). For fissured overconsolidated clays, \( u_2 \) readings are often negative. Probability-based methods are discussed by Tumay et al. (2013).

The most popular methods are based on soil behavioral charts with popular favoring of the 9-zone classification system (Lunne et al. 1997; 2009) that uses normalized piezocone readings: (a) normalized tip resistance: \( Q = (q_t - \sigma_{\text{vo}})/\sigma_{\text{vo}}' \), (b) normalized sleeve friction: \( F(\%) = 100 \cdot f_s/(q_t - \sigma_{\text{vo}}) \); and normalized porewater pressure: \( B_q = (u_2 - u_0)/(q_t - \sigma_{\text{vo}}) \). Any of these indirect CPT soil classification approaches should be cross-checked and verified for a particular geologic setting before routine use in practice.

The development of a CPT material index \( I_c \) has been found advantageous in the initial screening of soil types and helps to organize the sounding into 9 different zones of similar soil response. In this case, the CPT index is found from (Robertson 2009):

\[
I_c = \sqrt{(3.47 - \log Q_{\text{tn}})^2 + (1.22 + \log F)^2}
\]

where \( Q_{\text{tn}} = [(q_t - \sigma_{\text{vo}})/\sigma_{\text{atm}}]/(\sigma_{\text{vo}}'/\sigma_{\text{atm}})^n \) = stress-normalized net cone resistance

\( n \) = exponent that is soil-type dependent: \( n = 1 \) (clays); \( \approx 0.75 \) (silts); \( \approx 0.5 \) (sands)

\( \sigma_{\text{atm}} \) = atmospheric pressure (1 atm \( \approx 1 \) bar = 100 kPa)

Initially, an exponent \( n = 1 \) is used to calculate the starting value of \( I_c \) (i.e., \( Q_{\text{tn}} = Q \)) and then the exponent is upgraded to:

\[
n = 0.381 \cdot I_c + 0.05 \cdot (\sigma_{\text{vo}}/\sigma_{\text{atm}}) - 0.05 \leq 1.0
\]
Then the index $I_c$ is recalculated. Iteration converges quickly and generally only 3 cycles are needed to secure the operational $I_c$ at each depth. The soil zones and associated $I_c$ values are detailed in Figure 5. The sensitive soils of zone 1 can be screened using the following expression:

$$Q_{mn} < 12 \exp(-1.4 \cdot F_r)$$  

(4)

The stiff soils of zone 8 ($1.5 < F_r < 4.5\%$) and zone 9 ($F_r > 4.5\%$) can be identified from:

$$Q_{mn} \geq \frac{1}{0.005(F_r - 1) - 0.0003(F_r - 1)^2 - 0.002}$$  

(5)

Then, the remaining soil types are identified by the CPT material index: Zone 2 (organic clayey soils: $I_c \geq 3.60$); Zone 3 (clays to silty clays: $2.95 \leq I_c < 3.60$); Zone 4 (silt mixtures: $2.60 \leq I_c < 2.95$); Zone 5 (sand mixtures: $2.05 \leq I_c < 2.60$); Zone 6 (clean sands: $1.31 \leq I_c < 2.05$); and Zone 7 (gravelly to dense sands: $I_c \leq 1.31$). The red dashed line at $I_c = 2.60$ represents an approximate boundary separating drained ($I_c < 2.60$) from undrained behavior ($I_c > 2.60$).

### 2.3. Soil Unit Weight

Soil unit weight can be estimated from the CPT sleeve friction resistance (Mayne 2015):
\[
\gamma_n / \gamma_w = 1.22 + 0.15 \cdot \ln (100* f_s / \sigma_{atm} + 0.01)
\]  \hspace{1cm} (6)

where  \( \gamma_w = \text{unit weight of water.} \)

### 2.4 Effective Stress Friction Angle

The effective stress friction angle (\( \phi' \)) is one of the most important soil properties as it governs the strength of the geomaterial, as well as affects soil-pile interface and pile side friction. While an effective cohesion intercept (\( c' \)) can also be considered, this is usually reserved for cemented or bonded geomaterials or unsaturated soils and may lose its magnitude with time, ageing, or with prolonged environmental changes.

For clean quartz to silica sands where porewater pressures are essentially hydrostatic (\( B_q = 0 \)), the following expression has been calibrated with triaxial compression test results from undisturbed sand samples and normalized cone resistances, as presented in Figure 6 (Mayne 2007; Robertson & Cabal 2015):

\[
\phi' (\text{deg}) = 17.6^\circ + 11.0^\circ \log (Q_m)
\]  \hspace{1cm} (7)

![Diagram of effective friction angle](image)

**Fig. 6.** Evaluation of effective friction angle in undisturbed sands from CPT
In the case of soft to firm intact clays and silty clays, the effective friction angles is determined from the normalized cone resistance and porewater pressure parameters (Senneset et al. 1989; Mayne 2016), as shown in Figure 7. The exact solution when the angle of plastification $\beta = 0$ is given as:

$$Q = \frac{\tan^2(45^\circ + \phi'/2) \cdot \exp(\pi \cdot \tan \phi') - 1}{1 + 6 \cdot \tan \phi' \cdot (1 + \tan \phi') \cdot B_q}$$

which can be approximately inverted into the form (Mayne 2007):

$$\phi' = 29.5^\circ \cdot B_q^{0.121} \cdot [0.256 + 0.336 \cdot B_q + \log Q]$$

This algorithm is specifically applicable for the following ranges of porewater pressure parameter ($0.1 < B_q < 1$) and effective stress friction angles ($20^\circ < \phi' < 45^\circ$).

2.5 Stress History

The stress history can be characterized by an apparent yield stress or preconsolidation stress ($\sigma_{p}'$), as well as by its normalized and dimensionless form, $OCR = \sigma_p' / \sigma_{vo}' = \text{overconsolidation ratio}$. A generalized approach for soils using net cone resistance has been formulated (Mayne 2015):

$$\sigma_{p}' = 0.33 \cdot (q_t - \sigma_{vo})^m \cdot (\sigma_{atm}/100)^{1-m}$$

where $m' = \text{exponent depends on soil type: } m' = 1 \text{ (intact clays); } \approx 0.85 \text{ (sils); } \approx 0.72 \text{ (sands), as presented in Figure 8.}$
Fig. 8. Evaluation of yield stress or preconsolidation stress in soils from CPT

The exponent $m'$ has also been calibrated with CPT material index:

$$m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{15}}$$  \hspace{1cm} (11)

### 2.6 Undrained Shear Strength

For undrained loading of clays at constant volume, a temporary and transient condition occurs which is represented by the undrained shear strength ($s_u$). This can be related to the effective stress strength envelope ($c' = 0$; $\phi'$) and stress history (i.e., OCR) in the form:

$$s_u = (\sin\phi'/2) \cdot (OCR)^\lambda \cdot \sigma_{vo}'$$ \hspace{1cm} (12)

which corresponds to a simple shear mode.

### 2.7 Lateral Stress Coefficient

The horizontal geostatic state of stress is represented by the lateral stress coefficient, $K_0 = \sigma_{ho}'/\sigma_{vo}'$, commonly referred to as the at-rest condition. The magnitude of $K_0$ for soils that have been loaded and unloaded can be approximately estimated from:

$$K_0 = (1 - \sin\phi') \cdot OCR^{\sin\phi'}$$ \hspace{1cm} (13)
The value of $K_0$ finds applicability in assessing the pile side friction via the beta method, whereby as a first approximation: $\beta = K_0 \cdot \tan\phi$.

2.8 Additional GeoParameters

If desired, additional engineering properties can be determined during SCPTu (e.g., Robertson and Cabal 2015). For instance, the effective cohesion intercept ($c'$) can also be determined from plotting ($q_t - \sigma_{vo}$) versus $\sigma_{vo}'$ (Mayne 2016). If piezo-dissipation measurements are taken, the coefficient of consolidation ($c_{vh}$) and permeability ($k$) can be assessed (Mayne and Campanella 2005). This can be useful for pile driving projects in clays and silts that require an estimate of time for equilibrium of excess porewater pressures caused during installation, as well as for ground modification projects where these data are used for calculating time-rate-of-consolidation and wick drain spacings.

3 RELEVANCE OF SCPTu IN DEEP FOUNDATION DESIGN

The analysis of the axial load-displacement-capacity response of deep foundations is usually separated into two analytical components: (a) capacity; and (b) displacements. The SCPTu provides sufficient data input to handle the requirements using conventional calculations using limit equilibrium and plasticity solutions, as well as direct methods that are based on statistical analyses of large foundation load test databases, most recently funded by the offshore energy industry, including oil, gas, and wind.

3.1 Traditional Methods for Evaluating Side Friction and Toe Resistance

In common practice, pile shaft friction ($r_s$) is often calculated using alpha methods for clays and beta methods for sands (Brown et al. 2010). The beta method has also shown applicable for both clays and sands (O'Neill 2001; Fellenius 2016). Using the information from Section 2.7, a generalized expression for shaft friction can be given by:

$$r_s = C_M \cdot C_K \cdot K_0 \cdot \sigma_{vo}' \cdot \tan\phi$$  \hspace{1cm} (14)

where $C_M =$ pile material factor = 1 (rough cast-in-place concrete); 0.9 (prestressed concrete) 0.8 (timber); and 0.7 (steel);

$C_K =$ installation factor = 0.9 (bored or augered); 1.0 (low displacement, e.g. H-pile or open end pipe); and 1.1 (driven solid, e.g. prestressed concrete, closed-end pipe).

The evaluation of toe resistance ($r_t$) is often taken from limit plasticity solutions with associated shape and depth factors (Brown et al. 2010). For undrained toe response, the full value may be attained, thus:

$$undrained: \quad r_t = N_{c'} \cdot s_u$$  \hspace{1cm} (15)

where $N_{c'} = 9.33$ for a circular pile.

For drained toe response, the situation is more complex as the pile toe resistance increases with toe displacement (Fellenius 2016). Thus, from a practical standpoint, only a portion of the theoretical resistance will be realized:

$$drained: \quad r_t = \sigma_{vo}' \cdot N_q' \cdot f_x'$$  \hspace{1cm} (16)
where \( N_q \approx 0.77 \cdot \exp (\phi/7.5^\circ) \) = approximate expression for bearing factor

\( f' \) = strain incompatibility factor = 0.1 (bored piles); 0.2 (jacked); 0.3 (driven)

### 3.2 Direct CPT methods for Axial Pile Capacity

In lieu of assessing soil parameters \((\gamma, \phi, s_u, K_0, OCR)\) from CPT results that are input into theoretical formulae, considerable research efforts have centered on Direct CPT methods whereby the measured CPT results are scaled straightaway to provide unit shaft \( (r_s) \) and unit toe \( (r_t) \) resistances. At least 35 different Direct CPT methods are available (Niazi and Mayne 2013).

A particular interesting and versatile approach is the UniCone Method (Eslami and Fellenius 1997) as it employs all three piezocone readings \( (q_t, f_s, u_2) \) in its assessment. The unit shaft and toe resistances are obtained from the effective cone resistance: \( q_E = (q_t - u_2) \) according to:

\[
\begin{align*}
    r_s &= C_{se} \cdot (q_t - u_2) \quad (17) \\
    r_t &= C_{te} \cdot (q_t - u_2) \quad (18)
\end{align*}
\]

where \( C_{se} \) is a coefficient for shaft friction based on soil type and \( C_{te} \) = toe resistance coefficient. Figure 9 provides a quick summary overview of the approach and full details on UniCone are given elsewhere (Fellenius 2016).

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**Fig. 9. Overview of UniCone Method (after Eslami and Fellenius 1997; Fellenius 2016)**


**www.fellenius.net**

A. Determine effective cone resistance:

\[ q_E = q_t - u_2 \]

B. Plot \( q_E \) vs \( f_s \) to determine soil zonal type

C. Unit Side Resistance:

\[ r_s = C_{se} \cdot q_E \]

D. Unit Tip Resistance:

\[ B < 0.4m: r_{te} = q_E \]

\[ B > 0.4m: r_{te} = q_E/(3B) \]

where \( B = \) pile width (m)
A modified UniCone approach has been devised that uses the CPT material index ($I_c$) to assign values to the $C_{se}$ and $C_{te}$ coefficients (Niazi and Mayne 2016). For SBTn zone 1 (sensitive clays), the $C_{se}$ coefficient is obtained from:

$$SBTn \text{ Zone 1: } C_{se} = 0.074 - 0.004 \cdot [Q_{tn} - 12 \cdot \exp(-1.4 \cdot F)]$$  \hspace{1cm} (19)$$

and for the remaining soil types, the shaft coefficient may be estimated from the following expression:

$$SBTn \text{ Zones 2 to 9: } C_{se} = \theta_1 \cdot \theta_2 \cdot \theta_3 \cdot 10^{[0.732 \cdot I_c - 3.605]}$$  \hspace{1cm} (20)$$

where $\theta_1$ = pile type factor: 0.84 (bored piles), 1.02 (jacked), 1.13 (driven); $\theta_2$ = load direction factor: 1.11 (compression) and 0.85 (tension), and $\theta_3$ = loading rate factor (1.0 for soils with $I_c < 2.6$ and for $I_c > 2.6$: 0.97 (stepped load) and 1.09 (constant rate of penetration).

The toe coefficient may be estimated from:

$$\text{All SBTn Zones: } C_{te} = 10^{[0.325 \cdot I_c - 1.218]}$$  \hspace{1cm} (21)$$

The total axial compression capacity ($Q_{ult}$) is the sum of shaft capacity ($R_s$) plus toe capacity ($R_t$):

$$Q_{ult} = R_s + R_t = \int (A_s \cdot r_s) \, dz + A_t \cdot r_t$$  \hspace{1cm} (22)$$

where $A_s$ = circumferential area of the pile at depth $z$ and $A_t$ = toe area of the pile.

### 3.3 Pile Displacements and Axial Load Transfer

Elastic continuum theory provides closed-form expressions for calculating pile top displacements under axial loading, as well as detail the magnitude of axial load transfer with depth. These solutions require an evaluation of the ground stiffness that can be expressed either in terms of an equivalent Young's modulus ($E$) or shear modulus ($G$), since $E = 2 \cdot G(1 + \nu)$, where $\nu$ = Poisson's ratio (i.e., drained $\nu' = 0.2$ and undrained $\nu_u = 0.5$).

The simple case of a rigid pile situated in a single soil layer is depicted in Figure 10 showing the expressions for axial pile displacement and proportions of load transfer to the toe and shaft. The soil modulus can be constant with depth (i.e., homogeneous with $\rho_E = 1$), pure Gibson ($\rho_E = 0.5$) or generalized Gibson condition ($\rho_E = E_{M}/E_{L}$). More complex solutions for compressible pilings situated with toes bearing on stiff strata are also available (e.g., Brown et al. 2010).

### 3.4 Approximate Nonlinear Modulus

The equivalent elastic soil modulus is nonlinear with applied load level, especially from the nondestructive small-strain to middle-strain to strength ranges (Jardine et al. 2004). One simple algorithm that finds application to pile foundation loading is a modified hyperbola that provides a reduction factor to the initial small-strain elastic modulus (Mayne 2006):

$$E/E_{max} = 1 - (Q_t/Q_{tUlt})^{g'}$$  \hspace{1cm} (23)$$

where $g'$ = fitted exponent ($\approx 0.3 \pm 0.1$ for uncemented and non-structured soils).
The seismic piezocone test (SCPTu) thus finds special application to deep foundation analysis because it provides sufficient data ($q_t$, $f_s$, and $u_2$) for axial pile capacity calculations, as well as supplying the necessary soil stiffness ($E_{\text{max}}$) for the evaluation of displacements and load transfer. To illustrate the usefulness of the SCPTu results in axial pile evaluation, a case study from load tests for the Wakota Bridge are presented.

\section*{4 CASE STUDY - WAKOTA BRIDGE, MINNESOTA}

The ten-lane Wakota Bridge is located southeast of Saint Paul, Minnesota and was completed in 2010. The bridge enables interstate loop 494 to cross the Mississippi River. During the design phase, load tests on both driven open-end and closed end steel pipe piles were conducted with test piles having an outer diameter $d = 0.457$ m, an embedded length $L = 32$ m, and wall thickness $t = 12.5$ mm (Dasenbrock 2006). Both piles were loaded in axial compression, then afterwards loaded in tension, using a static reaction frame arrangement.

Soil conditions at the site can be assessed via the SCPTu sounding presented in Figure 11, indicating primarily firm sands with a few interbedded clay layers found at depths of 1, 3.5, 7 - 12, 17, and 22 - 27 m. The post-processing of the SCPTu provided direct estimates of the soil type via
Fig. 11. Seismic piezocone results at I-494 Wakota Bridge site, Saint Paul, Minnesota

Material index ($I_c$), unit weight ($\gamma_t$), effective friction angle ($\phi'$), preconsolidation stress ($\sigma_p'$), and $K_0$ profiles for input into beta method for pile shaft resistance (ave. $r_s = 65$ kPa), as well as by direct CPT methods using Unicone (ave. $r_s = 70$ kPa) and Modified Unicone (ave. $r_s = 75$ kPa). Evaluation of the toe resistance determined $r_{te} = 3593$ kPa by the Modified Unicone expression.

The evaluation of the two tests in axial compression are presented in Figure 12, while Figure 13 shows the two tests in tension. Overall, the agreement between the load-displacement responses

Fig. 12. Axial compression load tests at Wakota site: (a) open-end pile; (b) closed-end pile
are comparable between the measured field load tests and those generated using the elastic continuum solutions that rely on SCPTu data for input. For reference, the Euro criterion for "capacity" is taken as that load corresponding to \((s/d) = 10\%\) is shown on the measured load test curve.

5 DEVELOPMENTS IN GEOPHYSICS

5.1 Electromagnetics vs. Mechanical Wave Geophysics
Corresponding series of advancements in geophysics have also been made in the past two decades that are of direct use and benefit to site investigations concerning deep foundations. The broad categories of geophysics include: (a) electromagnetic wave techniques, and (b) mechanical wave methods. Electromagnetic wave methods include ground penetrating radar (GPR), surface resistivity surveys (SRS), and electromagnetic conductivity (EM). Mechanical wave approaches include crosshole testing (CHT), downhole testing (DHT), and Rayleigh wave methods, also referred to as surface wave techniques. The geophysical tests can be either invasive in boreholes or direct push probes, or noninvasive where the sensors (geophones, accelerometers, or electrodes) are placed in patterned arrays or configurations at the surface of the ground. An extensive review of geophysical methods is detailed by Wightman et al. (2003).

5.2 Rayleigh Wave Mapping
Of special mention in deep foundations applications is the utilization of Rayleigh wave methods to map the degree of heterogeneity of shear wave velocities in the ground and present these as subsurface profiles that are cross-sections across the project site. For instance, Figure 14 presents the results of a series of multi-channel analysis of surface wave (MASW) tests using streamers (movable plastic sleds) that contain the geophones and the individual surveys were taken at 1-m horizontal intervals across the site. Each MASW took less than 1 second using a sledge hammer. The streamer was pulled to the next location and the entire series took about 3 hours.
The MASW results were combined to form the cross section shown in Figure 14 with a clear indication of poor ground (\(V_s < 150\) m/s), normal soil conditions (\(V_s \approx 200\) m/s), and very strong ground conditions (\(V_s > 300\) m/s). Such information would be valuable prior to selection of locations of pile load tests, production pile installation, and/or necessary pile length determinations, as well as beneficial to applications in extent of ground modification and other construction activities.

6 CONCLUSIONS

Site characterization is an important component for the proper selection of deep foundation alternatives. Several advances in geotechnical site investigation have been made that quantify needs in foundation performance, including specialized probes for evaluation of scour potential, thermal soil properties, pile friction roughness, and soil stiffness, as well as a suite of noninvasive geophysics techniques. Of particular benefit is the utilization of hybrid tests such as seismic piezocone and seismic dilatometer that collect information at two ends of the stress-strain-strength curves in soils, namely the small-strain stiffness (\(G_{\text{max}}\) and \(E_{\text{max}}\)) from the shear wave velocity and the shear strength (\(\tau_{\text{max}}\) or \(s_u\), \(c'\), and \(\phi'\)) that are both needed in the assessment of the axial load-displacement-capacity of deep foundations.

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8 References


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