Cone Penetration Testing Manual for Highway Geotechnical Engineers

MnDOT Manual on CPT

prepared by

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Note: These documents are those task reports prepared by the Georgia Tech author for a joint GT-UMD manual entitled: Cone Penetration Test Design Guide for State Geotechnical Engineers prepared for MnDOT. The full report is located at:

Direct CPT Methods for Evaluating Bearing Capacity and Displacements of Shallow Foundations

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PREFACE

The analysis of shallow foundations on soils is classically handled as a two-step and two-part set of calculations involving: (a) bearing capacity, commonly reliant on limit plasticity solutions, and (b) settlement, or more appropriately termed displacements, that are assessed via elastic continuum theory. The utilization of cone penetration tests (CPT) provides the necessary geotechnical data for the site-specific information on the subsurface conditions at the project site, including the geostratigraphy and evaluation of input parameters. This is still a viable approach where the CPT readings are interpreted to give the soil unit weight ($\gamma_t$), effective friction angle ($\phi'$), undrained shear strength ($s_u$), preconsolidation stress ($\sigma_p'$), and elastic moduli ($D'$ and $E'$) for analysis.

An alternative approach is the use of CPT results to provide direct assessments of bearing capacity and/or settlements. A comparison of these two distinctly different and alternate paths is depicted in Figure 1.
A number of available direct CPT approaches for shallow footings are reviewed within this report. Moreover, as the entire load-displacement-capacity response of shallow foundations to loading occurs as a continuous nonlinear phenomenon, a single direct CPT solution is presented for the behavior of footings on sands (drained) and clays (undrained). The methods discussed herein are specifically to address the general case of vertical loading of shallow foundations. Additional more complex situations that consider: load eccentricity, moments, inclined forces, sloping ground, and other facets may be handled using well-established procedures that are discussed elsewhere (e.g., Vesić 1975; Kulhawy et al. 1983).

The focus and applicability were directed for foundations on granular soils and/or soils exhibiting drained behavior, since FHWA studies have concluded that less than 1% of shallow foundations for highway bridges are placed on clay soils (Paikowsky et al. 2010). One reason for this is because that for soft-firm intact clays, a more complicated process is needed that assesses both a short-term analysis (undrained) as well as long-term analysis (drained), thus requiring undrained bearing capacity, undrained distortion displacements, drained bearing capacity, and drained primary consolidation settlements.
Direct CPT Methods for Bearing Capacity of Footings on Sands

Classical bearing capacity solutions are based most often in limit plasticity theory, albeit can be formulated from limit equilibrium, cavity expansion, and numerical methods. Because the limit plasticity solutions are prevalent and adopt total stress analysis, they are usually developed as either fully drained (\(\Delta u = 0\)) or fully undrained (\(\Delta V/V = 0\)), where \(\Delta u\) = excess porewater pressure and \(\Delta V/V\) = volumetric strain. An extensive review of various solutions is detained in Paikowski et al. 2010) and not covered here.

There are a number of direct CPT methods for the evaluation of the bearing capacity of footings and shallow foundations. Table 1 lists a variety of these direct CPT approaches for footings on sands. In the direct approach, a one-step process is used to scale the measured cone penetrometer readings (i.e., \(q_c\) = measured cone tip resistance, \(q_t\) = total cone tip resistance, and/or \(f_s\) = sleeve friction, and/or \(u_2\) = measured porewater pressure) to obtain the ultimate bearing stress (\(q_{ult}\)) of the foundation.

Table 1. Direct CPT Methods for Bearing Capacity of Footings on Clean Sands

<table>
<thead>
<tr>
<th>Method</th>
<th>Surface Footing</th>
<th>Remarks and Embedment</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhof (1956)</td>
<td>(q_{ult} = q_c (B/12) \cdot c_w) where (B =) footing width (m); (c_w =) water table correction</td>
<td>(q_{ult} = q_c (B/12)(1+D_e/B)\cdot c_w) where (D_e =) depth of embedment</td>
<td>(c_w = 1.0) dry or moist sand (c_w = 0.5) submerged sand</td>
</tr>
<tr>
<td>Meyerhof (1974)</td>
<td>(q_{ult} = q_c (B + D)/40) with stresses in tsf</td>
<td>Presumably dry sands</td>
<td>(B =) fdn width (feet) and (D =) depth (feet)</td>
</tr>
<tr>
<td>Schmertmann (1978)</td>
<td>N/A (applies to embedded footings)</td>
<td>Square: (q_{ult} = 0.55\sigma_{atm}(q_c/\sigma_{atm})^{0.78}) Strip: (q_{ult} = 0.36\sigma_{atm}(q_c/\sigma_{atm})^{0.78})</td>
<td>Embedment applies: (D_e &gt; 0.5(1+B)) for (B &lt; 1m) (D_e &gt; 1.2) m for (B &gt; 1m) (B =) footing width</td>
</tr>
<tr>
<td>Canadian Geotech Society (CFEM, 1992)</td>
<td>(q_{ult} = R_{k0}q_c) where (R_{k0} = 0.3)</td>
<td>Applied to FS = 3 where FS = factor of safety</td>
<td></td>
</tr>
<tr>
<td>Tand, Warden, &amp; Funegård (1995)</td>
<td>N/A</td>
<td>(q_{ult} = R_k q_c + \sigma_{vo}) where (R_k =) fctn((D_e, B))</td>
<td>See Figure 2</td>
</tr>
<tr>
<td>Frank and Magnan (1995)</td>
<td>(q_{ult} = R_{k0}q_c) where (R_{k0} =) fctn(sand consistency): Loose: (R_{k0} = 0.14) Medium: (R_{k0} = 0.11) Dense: (R_{k0} = 0.08)</td>
<td>(q_{ult} = R_{k1} q_c + \sigma_{vo}) where (R_{k1} =) function((D_e, B, L,) &amp; sand consistency). Factor (R_{k1} = R_{k0}[1+R_{k2}(0.6+0.4(B/L))(D_e/B)]) and (R_{k2} = 0.35) (loose), 0.50 (medium), and 0.85 (dense)</td>
<td>Loose sand: (q_c &lt; 5) MPa Medium sand: (8) MPa $&lt; q_c &lt; 15$MPa Dense sand: (q_c &gt; 20) MPa</td>
</tr>
<tr>
<td>Author(s)</td>
<td>Equation</td>
<td>Notes</td>
<td></td>
</tr>
<tr>
<td>-----------------------------</td>
<td>----------</td>
<td>----------------------------------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>Eslami &amp; Robertson (1996)</td>
<td>( q_{ult} = K\Phi \cdot q_c )</td>
<td>See surface footing equation, ( K\Phi = \text{function (B/D_e, shape, and density)} )</td>
<td></td>
</tr>
<tr>
<td>Lee &amp; Salgado</td>
<td>( q_{ult} = \beta_{bc} \cdot q_{AVG} )</td>
<td>Not addressed, See Figure 4 for factor ( \beta_{bc} = \text{fctn(B, D_e, K_0, and s/B)} )</td>
<td></td>
</tr>
<tr>
<td>Eslami &amp; Gholami (2005, 2006)</td>
<td>( q_{ult} = R_{k1} \cdot q_c )</td>
<td>See embedded footing solution, where ( R_{k1} = \text{function(ratio D_e/B and normalized } q_c/\sigma_{vo}') )</td>
<td></td>
</tr>
<tr>
<td>Robertson &amp; Cabal (2007)</td>
<td>( q_{ult} = K\Phi \cdot q_c ) with ( K\Phi = 0.16 )</td>
<td>See surface footing equation, ( K\Phi = 0.16 )</td>
<td></td>
</tr>
<tr>
<td>Briaud (2007)</td>
<td>( q_{ult} = K\Phi \cdot q_c ) with ( K\Phi = 0.23 )</td>
<td>Based on full-scale tests at Texas A&amp;M, ( K\Phi = 0.23 )</td>
<td></td>
</tr>
<tr>
<td>Mayne &amp; Illingworth (2010)</td>
<td>( q_{ult} = 0.18 \cdot q_c_{\text{Mean}} )</td>
<td>Note: ( q_c_{\text{Mean}} ) is averaged CPT cone resistance over depth of influence ( z = 1.5 \cdot B \text{ deep} ), Based on 30 footing load tests on 12 sands, ( K\Phi = 0.16 )</td>
<td></td>
</tr>
<tr>
<td>Lehane (2013)</td>
<td>( q_{ult} = 0.16 \cdot q_c_{\text{AVE}} )</td>
<td>Note: ( q_c_{\text{AVE}} ) is averaged CPT cone resistance within ( z ) ( \text{(m)} = [B \text{ (m)} ]^{0.7} ), Based on 47 load tests, ( K\Phi = 0.16 )</td>
<td></td>
</tr>
</tbody>
</table>

Notes: 
- \( B \) = minimum footing width (or diameter), \( L \) = footing length, \( D_e \) = embedment depth, \( \sigma_{vo} \) = total overburden stress at bearing elevation.

Figure 2. Direct bearing capacity relationship for embedded square and strip footings situated on clean sands (after Schmertmann, 1978).
Figure 3. Direct CPT bearing factor $R_k$ as function of footing width $B$ and embedment depth from finite element analyses (Tand, et al. 1995).

Figure 4. Direct CPT bearing factor $R_k$ as function of footing shape, size-to-embedment ratio, and sand consistency (Eslaamizaad & Robertson, 1996).
Figure 5. Direct CPT factors for bearing capacity of sands from FEM analysis by Lee & Salgado (2005) in terms of footing size, relative density, and base settlement.

Figure 6. Direct CPT bearing factor $R_k$ as function of footing embedment to size ratio and normalized cone tip resistance (after Eslami & Gholami 2005)
Direct CPT Methods for Bearing Capacity of Footings on Clays

For direct CPT evaluations of foundation bearing capacity in clays, Table 2 summarizes some of the well-documented methods that are available. Generally, these assume that the loading is fast enough and the permeability of the clay is also sufficiently low, such that an undrained (i.e., constant volume) condition is maintained. This is fine for short term loading of foundations, particularly for soft to firm intact clays. However, the undrained case is not permanent. Given sufficient time, excess porewater pressures in the clay will eventually dissipate and hydrostatic conditions will return to equilibrium. During this dissipation phase, primary consolidation will occur that results in drained settlements. Also, a drained bearing capacity condition will prevail. As the CPT is advanced at a constant rate of 20 mm/s, the recorded readings normally constitute undrained behavior from a direct measurement viewpoint. Thus, direct CPT methods are normally focused at an undrained foundation response. Nevertheless, the drained footing case can be addressed using CPT results via use of conventional analysis approach and effective stress limit plasticity solutions that require piezocone penetrometers with porewater pressure measurements.

Many of the earlier solutions were based on data from mechanical penetrometers and/or older electric cones that measure the uncorrected tip resistance ($q_c$). It is now well-recognized that this measurement must be updated to the total cone resistance ($q_t$) because of porewater pressure effects that act on the mechanics of the load cell and geometry of the particular penetrometer design. The results are particularly significant in clays, silts, and mixed soils that are soft to firm to stiff where excess porewater pressures are recorded.

Table 2. Direct CPT Methods for Bearing Capacity of Footings on Clays

<table>
<thead>
<tr>
<th>Method</th>
<th>Footing</th>
<th>Comments</th>
<th>Notes/Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Meyerhof (1974)</td>
<td>$q_{ult} = \alpha_{bc} q_c$</td>
<td>Applicable to saturated insensitive clays under short-term loading</td>
<td>Cone tip resistance measured by mechanical CPT</td>
</tr>
<tr>
<td></td>
<td>where $0.25 \leq \alpha_{bc} \leq 0.50$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Trofimenkov (1974)</td>
<td>Approximation:</td>
<td>Strip footings on clays and sandy clays: $0.6m \leq B \leq 1.5m$; $1m \leq z_{emb} \leq 2.5m$</td>
<td>Mechanical CPT with $q_c$ and $q_{ult}$ in kg/cm²</td>
</tr>
<tr>
<td></td>
<td>$q_{ult} \approx (q_c/33)^{0.9}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>(Assuming $FS = 3$)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Schmertmann (1978) | $q_{ult} = \text{function (} q_c \text{ and foundation shape})$ | Relationship shown in Figure 6 for square and strip footings | Cone tip resistance measured by mechanical CPT

Tand, et al. (1986) | $q_{ult} = R_k(\ast q_c - \sigma_{vo}) + \sigma_{vo}$ | $\ast q_c = (q_{c1} \cdot q_{c2})^{0.5}$ where $q_{c1}$ is geometric mean from bearing elevation to 0.5B deeper and $q_{c2}$ is geometric mean from 0.5B to 1.5B beneath foundation base | Mix of data from mechanical CPT $q_c$ and electric CPT $q_c$

LCPC Method (Frank & Magnan, 1995) | Footing a surface: $q_{ult} = k_c q_c + \sigma_{vo}$ | Embedment case: $k_c = 0.32 \left[ 1 + 0.35(0.6 + 0.4 \frac{B}{L}) \cdot \frac{D}{B} \right]$ | $B =$ footing width $L =$ footing length $D =$ embedment

Note: Methods above generally consider undrained bearing capacity

![Graph](image)

Figure 7. Direct relationship between ultimate bearing stress in clays and measured cone tip resistance (Schmertmann, 1978).
Figure 8. Direct relationship between ultimate foundation bearing stress in clays and net cone tip resistance (after Tand et al. 1986).

Direct CPT Assessments of Settlement and Displacements of Shallow Foundations

Methods for evaluating the magnitude of foundation displacements ($s$) can be found using elastic theory, subgrade reaction methods, spring models, and numerical simulations. The classical approach to footing settlement calculations on sands is to utilize elastic theory solutions (e.g., Poulos & Davis 1974) which take on the form:

$$ s = \frac{q \cdot B \cdot I \cdot (1 - \nu)}{E_s} $$

where $s =$ settlement, $q =$ applied footing stress, $B =$ footing width, $\nu =$ Poisson's ratio, $E_s =$ soil modulus, and $I =$ displacement influence factor from elasticity theory. The value of $I$ depends upon foundation geometry, footing rigidity, embedment depth, variation of soil modulus with depth, compressible layer thickness, and other factors, as discussed by Mayne & Poulos (1999). The results of in-situ field tests such as pressuremeter tests (PMT), standard penetration tests (SPT), cone penetration tests (CPT), flat dilatometer tests (DMT), and other methods can be used to ascertain the input value of soil modulus, $E_s$. 
Alternatively, direct CPT methods have been investigated to provide a one-step assessment of foundation displacements. A summary of selected methods for direct CPT evaluation of shallow foundation settlements on granular soils is given in Table 3.

Table 3. Selected direct CPT methods for evaluating footing settlements in sands.

<table>
<thead>
<tr>
<th></th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>DeBeers and Martens (1957)</td>
</tr>
<tr>
<td>2.</td>
<td>Meyerhof (1965)</td>
</tr>
<tr>
<td>3.</td>
<td>DeBeer (1965)</td>
</tr>
<tr>
<td>4.</td>
<td>Thomas (1968)</td>
</tr>
<tr>
<td>5.</td>
<td>Schmertmann (1970)</td>
</tr>
</tbody>
</table>

Large Scale Footing Load Tests

The measured load-displacement response of shallow foundations is conducted in the field using either stepped loading procedures or continuous rate of displacement methods. Results from the well-documented test program at the Texas A&M site (Briaud & Gibbens 1999; Briaud 2007) for five spread footings on sand are shown in Figure 9. Each footing clearly shows a nonlinear behavior to loading over the range of testing. This site is one of the national geotechnical experimentation sites (NGES) that were funded by the National Science Foundation (NSF), Federal Highway Administration (FHWA), and American Society of Civil Engineering (ASCE).
Generalized Direct CPT Method for Footing Response on Soils

The use of applied foundation stress versus normalized displacement curves (q vs. s/B) is applied to footings on sands, silts, intact clays, and fissured clays. A square root plotting of the normalized displacements (s/B) permits a single parameter characterization for each specific soil type, that in turn, tracks with the net cone tip resistance. The generalization is based on a statistical review of data from large foundations (B > 0.5m) involving 70 full-scale load tests where, at all sites, CPT cone tip resistances are available. In fact, for fine-grained soils, the data are from electronic piezocone tests where the proper correction from q_c to q_t has been obtained to allow the best possible results. The methodology permits an evaluation of the load-displacement-capacity response of shallow square footings based on CPTs performed in the four types of ground.
conditions.

For the TAMU footings, the characteristic stress-normalized displacement response is shown in Figure 10 where it is clearly evident that all five foundations can be represented by a single curve.

Figure 10. Characteristic nonlinear stress-normalized displacement curve for the five TAMU footings.

**Characteristic Stress-Displacement Curves**

The concept of a characteristic stress (q) vs. normalized displacement (s/B) curve for foundation response on a given soil formation was recommended by Fellenius and Altaee (1994), and later supported by Decourt (1999), Briaud and Gibbens (1999), Lutenegger and Adams (2003), and Briaud (2007). In this approach, the measured load (Q) vs. displacement (s) from individual
footings of various sizes that rest on the same soil conditions all collapse to a single unified relationship given by:

\[ q_{\text{applied}} = a_f (s/B)^{b_f} \]  

(1)

where \( a_f \) and \( b_f \) are empirical fitting coefficients (Decourt 1999; Uzielli & Mayne 2011, 2012).

In a review of measured load tests data from large spread footings situated on different sands, it has been suggested that the aforementioned can be reduced to a single parameter expression by use of square root plotting (Mayne & Illingworth 2010; Mayne et al. 2012):

\[ q_{\text{applied}} = r_s \cdot \sqrt{(s/B)} \]  

(2)

where \( r_s \) = fitted soil parameter from best fit line regression. In the case of sands, the coefficient \( r_s \) represents the supporting soil conditions including particle size, relative density, and fines content, as well as geologic origin, ageing, and overconsolidation effects.

The results from the five TAMU footings are presented in this format in Figure 11 that shows the formation factor \( r_s = 4.86 \) MPa for this sand deposit. This single coefficient can be used to express the nonlinear load versus settlement of any size footing on this sand site. Moreover, one criterion for "bearing capacity" that is used by the European community identifies that stress (or load) which corresponds to \( (s/B) = 10\% \). That is, when the displacement equals ten percent of the footing width, then the "capacity" has been reached. Therefore, adopting this criterion for footings on sands, the ultimate bearing stress \( (q_{\text{ult}}) \) for footings on sand can be taken when \( (s/B) = 0.10 \), or when \( \sqrt{(s/B)} = 0.316 \). In that case, this gives: \( q_{\text{ult}} = 0.316 \ r_s \). For the TAMU site, this gives \( q_{\text{ult}} = 0.316 \times 4.86 = 1.53 \) MPa, as illustrated by Figure 12.
Figure 11. Characteristic stress versus square root normalized displacements for TAMU footings.

Figure 12. Eurocode or LCPC criterion applied to determine bearing capacity of TAMU sand.
FOOTING DATABASE

A database of spread footings and large plates was assembled which considers only full-size shallow foundations (0.5 ≤ B ≤ 6 m) that rest on sands, silts, and clays. These foundations were also subjected to cone penetration testing (CPT). The inclusion solely of large footings is significant because many prior studies also incorporated results from small-scale model footings which exhibit scale effects. In fact, experimental centrifuge work and numerical simulation studies have surprisingly shown that bearing capacity factors are size-dependent, especially for footings on sands with the factor $N_c$ decreasing with footing width $B$ (Kimura, et al. 1985; Cerato and Lutenegger 2007; Mase & Hashiguchi 2009). A direct approach based on full-size footings would alleviate such a difficulty and provide a more reliable means for foundation evaluation.

The compiled database is given in Table 4 with a total of 67 large footings and plates. These include a listing of 32 foundations on 13 sands, 11 footings on 4 silt deposits, 12 footings or large plate load tests on 6 intact clays, and 11 foundations on 5 fissured clays. Most of the footings were square or nearly square (80%), while the remaining were circular. The largest foundation is that at Shellhaven where a mat 5 m by 14 m in plan was loaded by stacking kentledge concrete blocks to cause a bearing failure in the underlying soft clay. For sands, the largest footing consisted of the 6 m square concrete pad at Durbin. In terms of equivalent square footings, mean footing sizes were 1.55 m (sands), 1.14 m (silts), and 1.26 m (clays).

Additional details on the individual load tests and site conditions are given elsewhere (Mayne 2009; Mayne & Illingworth 2010; Uzielli & Mayne 2011, 2012; Mayne et al. 2012; Mayne & Woeller 2014).
TABLE 4. Summary of large footings, soil conditions, and reference sources of database

<table>
<thead>
<tr>
<th>Sand Site</th>
<th>Location</th>
<th>Soil Conditions</th>
<th>Footings: Numbers, Shapes, and Sizes</th>
<th>Reference/Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>College Station</td>
<td>Texas</td>
<td>Pleistocene sand</td>
<td>5 Square: 1, 1.5, 2.5, 3 m</td>
<td>Briaud &amp; Gibbens (1999, JGE)</td>
</tr>
<tr>
<td>Kolbyttumen</td>
<td>Sweden</td>
<td>Glaciofluvial sand</td>
<td>4 Rect: B = 0.6; 1.2, 1.7, 2.4 m</td>
<td>Bergdahl, et al. (1985 ICSMFE, 1986)</td>
</tr>
<tr>
<td>Fittja</td>
<td>Sweden</td>
<td>Glaciofluvial sand</td>
<td>3 Rect: B=0.6m, 1.7, 2.4 m</td>
<td>Bergdahl, et al. (1984, 1985 ICSMFE)</td>
</tr>
<tr>
<td>Alvin West</td>
<td>Texas</td>
<td>Alluvial sand</td>
<td>2 Circular: D = 2.35 m</td>
<td>Tand, et al. (1994, GSP 40)</td>
</tr>
<tr>
<td>Alvin East</td>
<td>Texas</td>
<td>Alluvial sand</td>
<td>2 Circular: D = 2.2 m</td>
<td>Tand, et al. (1994, GSP 40)</td>
</tr>
<tr>
<td>Perth</td>
<td>Australia</td>
<td>Siliceous dune sand</td>
<td>4 Square: B = 0.5 and 1.0 m</td>
<td>Lehane (2008, 4th DCG)</td>
</tr>
<tr>
<td>Grabo T1C</td>
<td>Sweden</td>
<td>Compacted sand fill</td>
<td>1 Square: B = 0.46 m</td>
<td>Long (1993, SGI 43)</td>
</tr>
<tr>
<td>Grabo T2C</td>
<td>Sweden</td>
<td>Compacted sand fill</td>
<td>1 Square: B = 0.63 m</td>
<td>Long (1993, SGI 43)</td>
</tr>
<tr>
<td>Grabo T3C</td>
<td>Sweden</td>
<td>Compacted sand fill</td>
<td>1 Square: B = 0.80 m</td>
<td>Long (1993, SGI 43)</td>
</tr>
<tr>
<td>Labenne</td>
<td>France</td>
<td>Dune sand</td>
<td>5 Square: B = 0.7 and 1.0 m</td>
<td>Amar et al. (1998, ISC-1)</td>
</tr>
<tr>
<td>Green Cove</td>
<td>Florida</td>
<td>Brown silty sand</td>
<td>1 Circular: D = 1.82 m</td>
<td>Anderson et al. (2006, JGGE)</td>
</tr>
<tr>
<td>Durbin</td>
<td>South Africa</td>
<td>White fine sand</td>
<td>1 Square: B = 6.09 m</td>
<td>Kantley (1965, ICSMFE)</td>
</tr>
<tr>
<td>Porto</td>
<td>Portugal</td>
<td>Resilvay sand</td>
<td>1 Circular D=1.2m and 1 plate</td>
<td>Viana da Fonseca (2003)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Silt Site</th>
<th>Location</th>
<th>Soil Conditions</th>
<th>Footings: Numbers, Shapes, and Sizes</th>
<th>Reference/ Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jossigny</td>
<td>France</td>
<td>Soft clayey silt</td>
<td>2 Square Footings: B = 1 m</td>
<td>Amar et al. (1998, ISC-1)</td>
</tr>
<tr>
<td>Tomhill</td>
<td>Sweden</td>
<td>Glacial Baltic till</td>
<td>3 Square: B = 0.5, 1, and 2 m</td>
<td>Larsson (2001, SGI R-59)</td>
</tr>
<tr>
<td>Vagverket</td>
<td>Sweden</td>
<td>Stiff medium silt</td>
<td>3 Square: B = 0.5, 1, and 2 m</td>
<td>Larsson (1997, SGI R-54)</td>
</tr>
<tr>
<td>Vattahammar</td>
<td>Sweden</td>
<td>Bm-Gry layered silt</td>
<td>3 Square: B = 0.5, 1, and 2 m</td>
<td>Larsson (1997, SGI R-54)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Clay Site</th>
<th>Location</th>
<th>Soil Conditions</th>
<th>Footings: Numbers, Shapes, and Sizes</th>
<th>Reference/Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Baytown</td>
<td>Texas</td>
<td>fissured Beaumont</td>
<td>1 Sq.B=2.76; Plates=0.76m</td>
<td>Stuedlein &amp; Holtz (2008)</td>
</tr>
<tr>
<td>Belfast</td>
<td>Ireland</td>
<td>soft clay <em>sleech</em></td>
<td>1 Square Pads: B = 2.0 m</td>
<td>Lehane (Geot Engr 2003, ICE)</td>
</tr>
<tr>
<td>Bothkennar</td>
<td>Scotland</td>
<td>Soft silt clay</td>
<td>2 Square Pads: B = 2.2, 2.4 m</td>
<td>Jardine, et al. (Geot. 1995)</td>
</tr>
<tr>
<td>Bangkok</td>
<td>Thailand</td>
<td>Soft to stiff clay</td>
<td>4 Square: 0.67, 0.75, 0.90, 1.0m</td>
<td>Brand, et al. (1972, ASCE Purdue)</td>
</tr>
<tr>
<td>Hage</td>
<td>Norway</td>
<td>Stiff OC clay</td>
<td>2 Square Footings: B = 1.0m</td>
<td>Andersen &amp; Stenhammar (JGE 1982)</td>
</tr>
<tr>
<td>Rio Grande</td>
<td>Brazil</td>
<td>Sandy residual clay</td>
<td>3 Square: 0.4, 0.7, and 1.0m</td>
<td>Consoli, et al. (1998, JGGE)</td>
</tr>
<tr>
<td>Shellhaven</td>
<td>England</td>
<td>Soft estuarine clay</td>
<td>Rectangular: 5m by 14 m</td>
<td>Schnaid et al. (1993, Wroth Symp)</td>
</tr>
<tr>
<td>Texas City A</td>
<td>Texas Coast</td>
<td>fissured Beaumont</td>
<td>3 circular plates: d = 0.58 m</td>
<td>Tand, et al. (In-Situ 1986, GSP 6)</td>
</tr>
<tr>
<td>Texas City B1</td>
<td>Texas Coast</td>
<td>fissured Beaumont</td>
<td>2 circular plates: d = 0.58 m</td>
<td>Tand, et al. (In-Situ 1986, GSP 6)</td>
</tr>
<tr>
<td>Texas City B2</td>
<td>Texas Coast</td>
<td>fissured Beaumont</td>
<td>1 circular plate: d = 0.58 m</td>
<td>Tand, et al. (In-Situ 1986, GSP 6)</td>
</tr>
<tr>
<td>Alvin, Texas</td>
<td>Texas Coast</td>
<td>fissured Beaumont</td>
<td>3 circular plates: d = 0.58 m</td>
<td>Tand, et al. (In-Situ 1986, GSP 6)</td>
</tr>
</tbody>
</table>

For the series of footings on sands, Figure 13 shows the summary of measure formation factors ($r_s$) for the 30 foundation load tests. Also indicated on the graph are the corresponding mean cone tip resistances of the sands, averaged over 1.5 B deep beneath the bearing elevations. (The mean over 1B and 2B were also determined but essentially these all give similar values for a representative $q_c$ of the each sand.

Note also in sands that the corrected cone resistance ($q_c$) and net resistance ($q_{net}$) are all very close to the measured $q_c$, since: (a) porewater pressure corrections for the a-net value are negligible in clean sands; and (b) overburden stresses are typically only 1% to 3% of the magnitude of $q_c$. This is especially true of shallow depths. Therefore, for clean sands: $q_{net} \approx q_t \approx q_c$. 
As suggested by Briaud (2007), various in-situ measurements can be used to normalize the results, in his case, using the data from pressuremeter tests. Herein, the measured cone resistances from the CPT soundings in the sands are used to normalize the footing stress axis, as presented in Figure 14. It is evident that the results of the load-displacement response of all 32 footings on 13 different sands can be captured by the characteristic stress versus normalized displacement with the inclusion of the site-specific cone tip resistance. In this case, the mean trend for all footings and sand sites indicates a simple expression:

\[
\frac{q_{FOOTING}}{q_c} = 0.585 \cdot \sqrt{\frac{s}{B}}
\]
Figure 14. Normalized foundation stresses vs. square root of normalized displacement for spread footings on sand (modified after Mayne, Uzielli, & Illingworth, 2012)
The results of measured force-displacement curves (Q vs s) from footing load tests are nonlinear and thus the definition of "capacity" must be addressed. In this regard, Kulhawy (2004) has identified three main curve types that occur during load testing, as depicted in Figure 15. The most common response (Type A) shows no clear peak or plateau and is observed in sands and silts, as well as slow loading of insensitive clays and fissured clays. In this case, a "capacity" can be defined by the well-known European criterion corresponding to the load (or stress) when s/B=10% (Amar et al. 1998). For foundations situated on many clays, a plateau value is reached that can be construed as the “capacity”, as depicted as Type B response. In rare cases where sensitive clays or structured soils are encountered, a Type C relationship can be obtained whereby the load-displacement curve reaches a peak followed by strain softening. In the one case observed in this study (Haga clay), the corresponding “peak capacity” was reached at a pseudo-strain of s/B = 4% (Fig 16), followed by some strain softening. In the cases of soft clays at Belfast and Bothkennar, a plunging type (plateau failure) was achieved at around s/B=7%.
In the case of fine-grained soils that exhibit porewater pressures during cone penetration testing, the use of the net total cone resistance $q_{tnet} = (q_t - \sigma_v)$ must be considered (Lunne et al. 1997). As noted earlier, the correction of CPT data in sands is minimal because of the low value of penetration porewater pressures ($u_2 \approx u_0$) and the fact that total overburden stress is small relative to the cone resistance, especially for shallow soundings. Thus, the use of net total resistance may be substituted for $q_c$ into the trend of Figure 1 without any significance.

Figure 16. Summary of applied footing stresses normalized to CPT net cone resistance versus pseudo-strains ($s/B$) for all footings.
UNDRAINED AND DRAINED CAPACITY

The footings on sands can be considered drained response with no excess porewater pressures developed during loading. For the foundations situated on silts, analyses by Larsson (1997, 2001) concluded that these too were conducted under drained conditions. For shallow foundations on clays, the possibilities exist over the entire range of drainage cases, ranging from undrained to partially-drained to fully-drained, depending upon the applied rate of loading relative to the permeability of the geomaterial. If sufficient data were available for each of the clay sites, then the recent evaluation of drainage condition could be assessed using the normalized velocity criteria detailed by Chung et al. (2006). However, this was not possible with the current data set. Instead, the test loadings of footings on clays have essentially been assumed to primarily occur under undrained conditions, following the study by Tand et al. (1986). For this case, a limiting bearing stress can be calculated from bearing capacity analyses and related directly to the CPT readings.

From classical bearing capacity calculations using limit plasticity solutions, the ultimate foundation stress is:

\[ q_{ult} = N_c \cdot s_u \]  

(3)

where \( N_c \) = bearing factor for constant volume (= 5.14 for strip and 6.14 for square or circular plan) and \( s_u \) = undrained shear strength of the clay. For the case of soft to firm clays of low to medium sensitivity, the operational strength can be related to the preconsolidation stress (Mesri 1975; Jamiolkowski et al. 1985):

\[ s_u = 0.22 \sigma'_p \]  

(4)

Finally, the effective preconsolidation stress has been linked directly to the net cone resistance (Chen & Mayne 1996; Demers & Leroueil 2002):

\[ \sigma'_p = 0.33 (q_l - \sigma_{vo}) \]  

(5)
Combining equations (3), (4), and (5) results in direct expressions for undrained bearing capacity on intact clays from CPT results:

\[ q_{ult} = 0.373 \left( q_t - \sigma_{vo} \right) \quad (6a) \]

\[ q_{ult} = 0.445 \left( q_t - \sigma_{vo} \right) \quad (6b) \]

Equation (6b) is in excellent agreement with the database study compiled by Tand et al. (1986) for the case of intact clays and footings with no embedment. Their study also provided a lower relationship for foundations situated on jointed clays, specifically recommending that the bearing stress not exceed 0.30 \( q_{t,net} \). Review of the available data in Figure 3 shows this to be rather conservative and a more realistic value may be taken at 0.40 \( q_{t,net} \) for fissured clays.

**NORMALIZED UNDRAINED FOOTING RESPONSES**

The characteristic stress vs. normalized displacement curves for footings on clays exhibiting undrained conditions can be verified using a relatively recent case study on Beaumont clay reported by Stuedlein and Holtz (2010). The Baytown, Texas site was investigated using an exploration program of soil borings, piezocone soundings, and laboratory testing. The field testing included a large square footing (\( B = 2.74 \) m) and two small circular plates (\( D = 0.76 \) m). The measured load-displacement responses for all three foundations are presented in Figure 17. Of course, the large footing carried considerably higher loads than the two small plates on the same soil deposit. Yet, using the aforementioned characteristic stress (q) vs. square root of normalized displacement (s/B) essentially gave a unique relationship for all 3 foundations, as presented in Figure 18. In this case, utilizing the form of equation (2), the characteristic coefficient \( r_s = 1.77 \) MPa for this deltaic clay formation.
Figure 17. Measured load vs. displacement curves for 3 shallow foundations on clay at Baytown, Texas (data from Stuedlein and Holtz 2010)

Figure 18. Characteristic stress vs. square root of normalized displacement for all three foundations at Baytown site
GENERAL DIRECT CPT METHOD

In a manner analogous to the normalization of applied foundation stresses shown in Figure 14 for footings on sands, a generalized direct CPT method for shallow foundations on different soils can be made in the form:

\[ q = h_s \cdot q_{\text{net}} \cdot (s/B)^{0.5} < q_{\text{capacity}} \]  

where \( h_s \) = empirical fitting term that depends on soil type, specifically: \( h_s = 0.58 \) (sands), \( h_s = 1.12 \) (silts), \( h_s = 1.47 \) (fissured fine-grained soils), and \( h_s = 2.70 \) (clays), and \( q_{\text{capacity}} \) = foundation bearing capacity of the ground. A summary of the normalized stress versus the normalized displacement curves for these four categories of is presented in Figure 19 where the data fitting to obtain the \( h_s \) parameter on clays are limited to \((s/B) < 0.04\) corresponding to undrained loading. The data on silts and sands are considered as fully drained, whereas the fissured clay subset may be partially drained to undrained. The statistical measures for obtaining the fitted parameter \( h_s \) are quite good as shown in Figure 20 with the coefficient of determinations \( (r^2) \) values being \( r^2 = 0.947 \) (sands), 0.88 (silts), 0.935 (fissured), and 0.925 (intact clays). Additional statistical evaluations on the database are provided in Uzielli & Mayne (2011).

Figure 19. Normalized foundation stress vs square root of normalized displacements for all 67 large footings in database
The same data are shown on Figure 21 in a log-log plot to emphasize the early parts of the load-displacement responses of these footings. In consideration of all the data, the best fit line from regression analyses show excellent statistical correlations. As detailed earlier, the undrained bearing capacity of square or circular footings on clays can be taken as approximately 0.45 $q_{\text{net}}$. For silts and sands that experience drained loading with no excess porewater pressures being developed and no clear peak value for capacity is evident, the $(s/B)=10\%$ criterion can be used. In this case, equation (7) can be used directly to obtain stress $q = q_{\text{capacity}}$ when $(s/B)^{0.5} = 0.316$.

An alternate approach is presented in Figure 22 which summarizes the direct CPT method for estimating the load-displacement-capacity of shallow square footings on four soil types. The maximum values of soil bearing capacity ($q_{\text{max}}$) can be capped at stresses corresponding to a percentage of the average measured net cone tip resistance, determined over the depth interval from the foundation bearing elevation to 1.5 $B$ deep. In such an approach the foundation bearing
capacity can be taken as \((q_{\text{capacity}}/q_{\text{net}}) = 0.2 \) (sands), 0.35 (silts), 0.40 (fissured), and (0.45) intact clays. Using the assigned values of the \(h_s\) parameter, the corresponding cut-off for the general stress-normalized displacement relationship given by equation (7) can be established, specifically giving corresponding values of the normalized displacements, which can also be considered as pseudo-strains, equal to \((s/B)_{\text{max}} = 4\% \) (clays), 7\% (fissured clays), 10\% (silts), and 12\% (sands).

**CPT SOIL MATERIAL INDEX, \(I_c\)**

The soil behavioral type can be assessed indirectly by CPT using a material index \((I_c)\) which is defined by (Robertson 2009a,b):

\[
I_c = \sqrt{[3.47 - \log Q_{in}]^2 + [1.22 + \log F_r]^2} \tag{8}
\]
Figure 22. Summary graph and equations of direct CPT method for evaluating the vertical load-displacement-capacity response of square footings on soils

where $Q_{tn} = $ stress-normalized cone tip resistance and $F_r = $ normalized sleeve friction are calculated from the cone penetrometer readings as follows:

$$Q_m = \frac{(q_i - \sigma_{vo}) / \sigma_{atm}}{(\sigma_{vo}' / \sigma_{atm})^n}$$

$$F_r (\%) = 100 \cdot \frac{f_s}{(q_i - \sigma_{vo})}$$

where $\sigma_{vo} = $ total vertical overburden stress, $\sigma_{vo}' = $ effective vertical stress, and $\sigma_{atm} = $ a reference stress equal to atmospheric pressure ($\sigma_{atm} = 1 \ atm \approx 1 \ bar \approx 100 \ kPa$). In the initial evaluation, the exponent $n$ is set to $n = 1$ to find the soil behavioral type (SBT) which is based on a 9-zonal chart; e.g. zone 3 = clay and zone 6 = sand. The value of exponent $n$ varies with soil type; ranging from $n = 1$ in intact clays and decreasing with increasing grain size to around $n \approx 0.75$ in silts and...
around \( n \approx 0.5 \pm 0.2 \) in clean sands. The appropriate value of exponent \( n \) is found by iteration until conversion using the following relationship (Robertson 2009b):

\[
 n = 0.381 \cdot I_c + 0.05 \left( \frac{\sigma_{vo'}}{\sigma_{atm}} \right) - 0.15 \leq 1.0
\]  

(11)

As the distinction that footings on intact clays subjected to fast loading are behaving primarily under undrained conditions (i.e., constant volume), it has been suggested (Robertson 2009a) that this occurs when \( I_c > 2.7 \), while in contrast, \( I_c < 2.5 \) corresponds more or less to drained loading (i.e., no excess porewater pressures).

The CPT material index can be used to identify soil type and the corresponding the characteristic \( h_s \) value for estimating footing load-displacement response. In a number of the case studies investigated, the results of the sleeve friction readings were also available to allow the calculation of \( I_c \) with depth at these sites. Figure 23 shows the tentative relationship between the fitted values

![Figure 23](image_url)

Figure 23. Foundation soil formation parameter \( h_s \) versus CPT material index, \( I_c \).
of the $h_s$ parameter plotted versus the corresponding CPT material index, with an approximate trend given by:

$$h_s = 2.8 - \frac{2.3}{1 + (I_c / 2.4)^{15}}$$

(12)

Soil behavioral type from the $I_c$ ranges given by Robertson (2009) are also shown on the figure with an overall general agreement with the known soil classifications.

In summary, the CPT can provide an evaluation of the load-displacement-capacity response directly via:

Footing stress:  $$q = q_{net} \cdot \sqrt{(s/B)} \cdot \left[ 2.8 - \frac{2.3}{1 + (I_c / 2.4)^{15}} \right]$$

(13)

which may be of interest in mixed soil types, such as silty sands, sandy silts, and the like. As discussed earlier, foundation “bearing capacity” may be taken by a limiting value of pseudo-strain, $(s/B)_{max}$, or by a limiting footing stress ($q_{max}$) as specified as a percentage of the measured net cone tip resistance ($q_{net}$).

**CONCLUSIONS**

A direct CPT method for square and circular shallow footings is developed using a database based on 67 full-scale field load tests where the minimum footing size (equivalent square width: $B > 0.5 \text{ m}$) is established to avoid scaling issues identified by previous researchers. The use of load vs. displacement is generalized by characteristic stress vs. normalized displacement ($s/B$) and further simplified by a square root plotting technique that captures the ground response in a single parameter that is related to the net cone tip resistance. Data are grouped according to four main soil categories that include: sands, silts, fissured clays, and intact clays. It is generally believed that the footings on sands and silts exhibit fully drained behavior, while in intact clays, an undrained response occurs under conditions of constant volume.
The summary equation for evaluating the vertical stress-displacement-capacity of square footings is given by:

\[ q = h_s \cdot q_{\text{net}} \cdot (s/B)^{0.5} < q_{\text{capacity}} \]

where \( q \) = applied footing stress, \( q_{\text{net}} = q_t - \sigma_{\text{vo}} \) = mean value of net cone tip resistance within 1.5 \( B \) below the foundation bearing elevation, \( s \) = displacement, \( B \) = footing width, and \( h_s \) = characteristic formation parameter (\( h_s = 0.58 \) in clean sands, 1.12 in silts, 1.47 in fissured clays, and 2.70 for intact clays). The parameter \( h_s \) also relates directly to CPT material index, \( I_c \). The foundation capacity (\( q_{\text{capacity}} = q_{\text{max}} \)) depends upon the mode of failure, as well as drainage conditions, and can be taken as a fraction of the net cone tip resistance, where \( q_{\text{max}}/q_{\text{net}} = 0.20 \) (sands), 0.35 (silts), 0.40 (fissured clays), and 0.45 (intact clays). Alternatively, the “capacity” can be defined using a limiting pseudo-strain, given by \( (s/B)_{\text{max}} = 4\% \) (clays), 7\% (fissured), 10\% (silts), and 12\% (sands).

Further studies are ongoing to investigate the extension of the approach from square and circular shaped foundations to include long rectangular foundations and continuous strip footings (\( A > B \)) which would be of practical interest and value to foundation engineers.

ACKNOWLEDGMENTS

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REFERENCES


INTRODUCTION

Geotechnical site characterization is an important first step towards the evaluation of subsurface conditions and determination of soil layering, geomaterial classification, and the evaluation of soil engineering parameters for the analysis and design of foundations, retaining walls, tunnels, excavations, embankments, and slope stability. Increasing use of the electronic cone penetrometer in highway applications is prominent in the USA because the results are obtained much faster and less expensive than traditional methods that rely on rotary drilling, augering, sampling, and laboratory testing. Moreover, the cone penetration test (CPT) provides at least three independent and continuous readings on soil behavior that are digitally recorded and fully available at the end of the sounding. As such, the reliable and consistent interpretation of CPT data is important, since civil engineering works will best realize the efficiency and economy of this technology in constructed facilities for county, state, and interstate projects.

CONE PENETRATION TEST

The cone penetrometer is an electronically-instrumented steel probe that is vertically pushed into the ground at constant rate of 20 mm/s (0.8 in/sec) using a hydraulic system. The penetrometer is outfitted with load cells, pressure transducers, and inclinometers that measure at least three readings continuously with depth: (a) cone tip resistance, $q_t$, (b) sleeve friction, $f_s$, and (c) porewater pressure, $u_2$. With the latter reading, the device is called a piezocone, thus the designation CPTu is used to indicate porewater pressure. Additional sensors are available that can be used to measure: inclination, resistivity, pH, temperature, shear wave velocity, and other readings, if desired.

Figure 1 shows a schematic rendering of the cone penetration test that is performed in the field per ASTM D 5778 procedures. The hydraulic system is nominally rated at 180 kN (20 tons) capacity and often mounted on a truck, but can also be positioned on tracked vehicles or anchored frames. Figure 2 shows a MnDOT cone truck that uses the full dead weight of the rig for the hydraulic reaction forces which are applied at the center of the vehicle. The cab is enclosed so that soundings may proceed during inclement weather and the data acquisition system and operator are protected.
Figure 1. Setup and procedures for cone penetration testing (CPT)

Figure 2. CPT truck-mounted rig used by MnDOT
A representative sounding taken at the I-35 test site just northeast of Saint Paul is presented in Figure 3. Here the separate profiles of $q_t$, $f_s$, and $u_2$ with depth are shown in side-by-side plots. In the last graph, the hydrostatic porewater pressure ($u_0$) due to the groundwater table is indicated by the blue dashed line.

These readings are used to interpret the soil layers, types, and properties of the ground, as discussed in the following sections.

Figure 3. Representative CPTu sounding at I-35 test site northeast of St. Paul, MN showing profiles of: (a) cone resistance, (b) sleeve friction, and (c) penetration porewater pressures.

SOIL PARAMETER INTERPRETATION

A variety of soil engineering parameters can be interpreted from CPT results on the basis of theoretical frameworks, analytical models, or numerical simulations, otherwise by empirical methods based on correlations and statistics of databases. Figure 4 shows a conceptual
utilization of the readings from the CPT for interpretation of geostratigraphy, compressibility, flow characteristics, and stress-strain-strength behavior of soils.

Table 1 lists a selection of geoparameters that have been addressed for these purposes. The most common or useful relationships will be discussed in subsequent sections and reference is made to other sources (Lunne et al. 1997; Mayne 2007; Robertson & Cabal 2015) for additional details.

**Geoparameter Evaluation from CPT**

Figure 4. Conceptual post-processing of CPTu results for geoparameter evaluation
Table 1. Common geoparameters interpreted from CPT data

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Parameter</th>
<th>Remarks / Notes</th>
</tr>
</thead>
</table>
| SBT    | Soil behavior type (SBT) | 1. Rules of thumb for $q_t$, $u_2$, and/or $f_s$  
2. Effective cone resistance vs. sleeve friction:  
\( q_e = q_t - u_2 \) versus $f_s$ (Eslami & Fellenius 1997)  
3. Charts of Q, F, and B_q (Robertson 1990, 1991)  
4. CPT material index $I_c$ (Robertson 2009) |
| $I_c$  | CPT material index | $I_c = fctn (Q_m \text{ and } F)$ |
| $\gamma_t$ | Unit weight | 1. Estimate from $f_s$ reading (Mayne, CPT'10)  
2. Estimate from $q_t$ and FR (Robertson & Cabal, CPT'10) |
| $\rho_t$ | Mass Density | $\rho_t = \gamma_t/g_a$ where $g_a = 9.8 \text{ m/s}^2$ |
| $\sigma_{vo}$ | Overburden stress | $\sigma_{vo} = \int \gamma_t \, dz = \sum (\gamma_t \cdot \Delta z_i)$ |
| $u_0$ | Hydrostatic pressure | 1. Calculated from depth to groundwater table $(z_w)$, if known  
2. Evaluate from piezo-dissipations to $\Delta u = 0$ |
| $\sigma_{vo}'$ | Effective stress | $\sigma_{vo}' = \sigma_{vo} - u_0$ |
| $\sigma_p'$ | Preconsolidation stress | $\sigma_p' = 0.33 (q_{net})^{m'}$ where $m' = fctn (I_c)$  
where stresses are in kPa |
| YSR | Yield stress ratio | $\text{YSR} = \sigma_p'/\sigma_{vo}'$ (formerly OCR) |
| $c'$ | Effective cohesion intercept | 1. NTH method in plot of $q_{net}$ vs. $\sigma_{vo}'$  
2. Empirical: $c' \approx 0.03 \sigma_p'$ (SGI 2003)  
3. In clays, $c' \approx 0.1 c_o$ (SGI 2003) |
| $\phi'$ | Effective friction angle | 1. NTH method for clays & silts = fctn (Q, B_q)  
2. Quartz and silica sands: $\phi' = 17.6^\circ + 11.0^\circ \log(q_{ut})$  
(Mayne 2006; Robertson & Cabal 2015) |
| $s_u$ | Undrained shear strength | $s_u = q_{net}/N_{kt}$  
1. where factor $N_{kt} = 12$ for triaxial compression  
mode in soft clays (Low et al. 2012)  
2. Factor $N_{kt} = fctn (B_q)$  
(Georgia Tech & Fugro Study, 2014)  
3. $s_u = (\sin\phi'/2) (\text{YSR})^\Lambda \sigma_{vo}'$ where $\Lambda = (1 - C_s/C_c)$  
(Critical State Soil Mechanics for DSS mode) |
| $D'$ | Constrained modulus | $D' \approx \alpha \cdot q_{net}$ (Sanglerat 1976, Gardner & Mitchell 1975; Mayne 2007) |
G₀  Small-strain modulus  

\[ G₀ = \rho_t V_s^2 \]

\[ G = \frac{E}{2(1+\nu')} \quad \text{where} \quad \nu' = 0.2 \] (drained)  
and \[ \nu_u = 0.5 \] (undrained)

2. Reduction factor (RF) obtained from either: (a) mobilized strength \( \tau/\tau_{\text{max}} \) or (b) shear strain \( \gamma_s \); and applied to small strain shear modulus, \( G₀ \):

\[ G = RF \cdot G₀ \]

E  Young's modulus

1. \( E' = 2G (1+\nu') \) where \( \nu' = 0.2 \) (Poisson's ratio)

2. \( D' = E' (1-\nu')/(1+\nu')(1-2\nu') \)

3. \( K' = E'/[3((1-2\nu')] \)

\( c_{vh} \)  Coefficient of consolidation

Evaluated from \( \Delta u \) dissipation test, typically taken to 50% consolidation at time, \( t_{50} \)

\( K' \)  Bulk modulus

Obtained from laboratory isotropic consolidation test

\( K₀ \)  Lateral stress coefficient

Using \( \phi' \) and YSR for virgin loading and unloading in soil: \( K₀ = (1 - \sin \phi') \cdot \text{YSR} \sin \phi' \)

k  Hydraulic conductivity

1. Direct from \( \Delta u \) dissipations, often using \( t_{50} \) measurement (Parez & Fauriel 1988)

2. Convert from \( c_v \) interpretation

3. Estimate from \( I_c \) relationship (Robertson 2010)

LI  Liquefaction index

Using cyclic stress ratio (CSR), cyclic resistance ratio (CRR), and normalized cone tip resistance, \( Q_{tn} \)

Notes:  
\( q_t = \) cone tip resistance  
\( f_s = \) sleeve friction  
\( u_2 = \) shoulder porewater pressure reading  
\( q_{\text{net}} = (q_t - \sigma_{vo}) = \) normalized cone resistance  
\( Q = (q_t - \sigma_{vo})/\sigma_{vo}' = \) modified normalized cone resistance using units of bars and exponent \( n \) varies with soil type: \( n = 1.0 \) (clay), \( n = 0.75 \) (silt), \( n = 0.5 \) (sands)  
\( Q_{\text{net}} = [(q_t - \sigma_{vo})/\sigma_{\text{am}}]/(\sigma_{vo}/\sigma_{\text{am}}) = \) modified normalized cone resistance using any units, where \( \sigma_{\text{am}} \) is a reference stress equal to 1 atmosphere. Note that approximately:

\[ \sigma_{\text{am}} \approx 1 \text{ bar} = 100 \text{ kPa} \approx 1 \text{ kg/cm}^2 \approx 1 \text{ tsf} = 14.7 \text{ psi} \]

\[ F = 100 \cdot f_s/q_{\text{net}} \] is normalized sleeve friction (%)  
\( B_q = \Delta u/q_{\text{net}} = (u_2 - u_0)/(q_t - \sigma_{vo}) \) is normalized porewater pressure parameter  
NTH = Norwegian Institute of Technology, Trondheim.

\( \tau/\tau_{\text{max}} \) = mobilized strength where: \( \tau = \) shear stress; \( \tau_{\text{max}} = \) shear strength.  
\( t_{50} = \) time to reach 50% consolidation
GEOSTRATIGRAPHY and SOIL BEHAVIORAL TYPE (SBT)

In routine CPT, soil samples are not normally taken and thus the measured stress, friction, and/or porewater pressure readings are used to infer the types of soils. This can be accomplished using three basic approaches:

a. "Rules of Thumb" or approximate guidelines for a quick visual assessment.

b. Soil Behavioral Type (SBT) Charts that are based on either the three readings \(q_t, f_s, u_2\), net readings, including net cone resistance \(q_{net} = q_t - \sigma_{vo}\), effective cone resistance \(q_E = q_t - u_2\), friction ratio \(R_f \% = 100\cdot f_s/q_t\), and excess porewater pressures \(\Delta u = u_2 - u_0\); or using normalized CPT readings, such as \(Q, F, B_q,\) or \(U^*\);

c. Probabilistic methods where the CPT readings have been calibrated from lab tests on recovered soil samples (e.g., Tumay et al. 2013).

Approximate Rules of Thumb

The approximate rules of thumb provide simple guidelines for soil type and suggest that sands are identified when \(q_t > 5\) MPa and \(u_2 \approx u_0\), whereas intact clays are prevalent when \(q_t < 5\) MPa and \(u_2 > u_0\) (Mayne et al. 2002). The magnitude of porewater pressures reflects the consistency of the intact clay, such that: soft \(u_2 \approx 2\cdot u_0\), firm \(u_2 \approx 4\cdot u_0\), stiff \(u_2 \approx 8\cdot u_0\), and hard \(u_2 \approx 20\cdot u_0\). However, for fissured overconsolidated clays, the measured porewater pressures are less than hydrostatic, in fact, often negative: \(u_2 < 0\). On land, negative porewater pressure readings at the shoulder filter element of the piezocone should be greater than -1 bar; i.e., \(u_2 > -100\) kPa (-14.7 psi). Also, for clean sands, the friction ratio \(FR = 100\cdot f_s/q_t < 1\%\) while for insensitive clays \(FR > 4\%\).

Figure 5 presents a 36-m deep piezocone sounding from the MnDOT Wakota Bridge site which crosses the Mississippi River at I-494 (Dasenbrock 2006). The \(q_t\) and \(u_2\) readings have been annotated using the aforementioned "rules of thumb", indicating the predominance of sands at this site. As noted, there are five interbedded clay layers evident at depths of 1 m, 3-4 m, 7 - 12 m, 17 m, and 22-27 m.
Soil Behavioral Type Charts

The most common approach for identification of soil types is based on soil behavioral charts. Reviews of several chart methods are provided elsewhere (Kulhawy & Mayne 1990; Fellenius & Eslami 2000; Shahri et al. 2015). Current popularity favors the 9-zone classification system to evaluate soil behavioral type (SBT) that uses normalized piezocone readings (Robertson 1990, 1991; Lunne et al. 1997):

(a) normalized tip resistance: $Q = (q_t - \sigma_{vo})/\sigma_{vo}'$  \hspace{1cm} (1a)
(b) normalized sleeve friction: $F (%) = 100 \cdot f_s/(q_t - \sigma_{vo})$  \hspace{1cm} (1b)
(c) normalized porewater pressure: $B_q = (u_2 - u_0)/(q_t - \sigma_{vo})$  \hspace{1cm} (1c)
Figure 6. Nine-zone soil behavioral type charts: (a) normalized cone resistance vs. normalized sleeve friction; (b) normalized cone resistance vs. normalized porewater pressure parameters (adapted after Robertson 1991; Lunne et al. 1997).

While the Q-F-B_q classification is actually a three-dimensional plot, it is often presented as a set of two-dimensional plots, specifically: Q vs. F and Q vs. B_q, as shown in Figure 6. Data are grouped according to layers and can be superimposed to identify their association with the nine soil behavioral types.

All indirect CPT soil classification approaches should be cross-checked and verified for a particular geologic setting and local geotechnical conditions 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 their respective SBT zones of similar soil response. In this case, the CPT index is found from (Robertson 2009):

\[ I_c = \sqrt{(3.47 - \log Q_m)^2 + (1.22 + \log F_r)^2} \]  

(2)
where  \( Q_{tn} = \text{modified stress-normalized net cone resistance, given by:} \)

\[
Q_{tn} = \frac{(q_t - \sigma_{vo})/\sigma_{atm}}{(\sigma_{vo}/\sigma_{atm})^n}
\]

where \( \sigma_{atm} = \text{reference stress equal to atmospheric pressure (1 atm \approx 1 \text{ bar} \approx 100 \text{ kPa} \approx 1 \text{ tsf} \approx 14.7 \text{ psi})} \). The exponent \( n \) is soil-type dependent: \( n = 1 \) (clays); \( n \approx 0.75 \) (silts); and \( n \approx 0.5 \) (sands). If units of bars are used, then \( Q_{tn} \) is simply:

\[
Q_{tn} = \frac{(q_t - \sigma_{vo})}{(\sigma_{vo}/\sigma_{atm})^n} \quad \text{(units of bars)}
\]

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

\[
n = 0.381 \cdot I_c + 0.05 \cdot (\sigma_{vo}/\sigma_{atm}) - 0.05 \leq 1.0
\]

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 7.

The sensitive soils of zone 1 can be screened using the following expression:

\[
\text{Zone 1: } \quad Q_{tn} < 12 \exp (-1.4 \cdot F_t)
\]

If any soil layers are found within zone 1 (sensitive soils), then caution should be exercised as these structured clays are prone to instability, collapses, and difficulties in construction and performance. Another indicator of zone 1 sensitive soils is when the normalized porewater parameter \( B_q > 0.8 \). When these criteria are evident, the geoengineer should consult with senior geotechnical personnel or the chief engineer for guidance.
Figure 7. Delineation of soil behavioral type zones using CPT material index, $I_c$

Stiff overconsolidated clayey sands and sandy clays soils of zone 8 ($1.5% < F_r < 4.5%$) and zone 9 ($F_r > 4.5%$) can be identified from the following criterion:

$$Q_m > \frac{1}{0.005(F_r - 1) - 0.0003(F_r - 1)^2 - 0.002}$$ (6)

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 \textit{drained} ($I_c < 2.60$) from undrained behavior ($I_c > 2.60$).

Once the specific zone has been assigned to a layer, a visual representation can be made to show either the zone number or colorization so that the predominant layers and soil types can be realized.

**Probabilistic Soil Types from CPT**

The inference of soil type has been addressed using probabilistic methods, as discussed by Abu-Farshakh et al. (2008) and Tumay et al. (2013). Here, the CPT readings can be post-processed to provide the percentage of sand, silt, and clay components with depth. The output is consistent and compatible with the current MnDOT soil classification chart (Figure 8) which is similar in content to that used by the USDA.

![MnDOT Soil Classification System](image)

Figure 8. Chart for soil classification system by MnDOT
Soil Unit Weight

As shown by Figure 9, total soil unit weight can be estimated from the CPT sleeve friction resistance (Mayne 2014). The equation can be expressed in dimensionless terms by:

\[
\gamma_t = \gamma_w \cdot [1.22 + 0.15 \cdot \ln (100f_s/\sigma_{atm}+0.01)]
\]  

(7)

where \( \gamma_w \) = unit weight of water and \( \sigma_{atm} \) = atmospheric pressure.
Effective Stress Friction Angle

Sands

The effective stress friction angle ($\phi'$) is one of the most important soil properties as it governs the strength of geomaterials, 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 10 (Mayne 2007; 2014)

$$\phi' \text{ (deg)} = 17.6^\circ + 11.0^\circ \log (q_{t1})$$

(8)

Fig. 10. Evaluation of effective stress friction angle in quartz-silica sands from CPT
Note: Relationship applies to drained soil behavior when $I_c < 2.6$ and/or $B_q < 0.1$ where $q_{t1}$ is an earlier form of stress normalized cone tip resistance for sands given by:

$$q_{t1} = \frac{q_t / \sigma_{atm}}{(\sigma_{vo} / \sigma_{atm})^{0.5}}$$  \hspace{1cm} (9a)

In terms of units of bars, this is expressed simply as:

$$q_{t1} = \frac{q_t}{\sqrt{\sigma_{vo}}} \quad \text{(in units of bars)}$$  \hspace{1cm} (9b)

Recently, Robertson & Cabal (2015) recommended the use of the modified form of normalized cone resistance ($Q_{tn}$) in equation (8):

$$\phi' \ (\text{deg}) = 17.6^\circ + 11.0^\circ \log (Q_{tn})$$  \hspace{1cm} (10)

In sands, $q_{net} \approx q_t$ since the overburden term is small relative to the cone tip resistance. Figure 11 shows that the two normalizations give very comparable results.

![Figure 11. Comparable values from $q_{t1}$ and $Q_{tn}$ normalization schemes for CPTs in sands](image-url)
Clays and Silts

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 12. The exact solution when the angle of plastification $\beta = 0$ is given as:

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

(11)

![Figure 12. Evaluation of effective stress friction angle in clays and silts from CPTu results](image)

Note: Relationship applies to undrained soil behavior when $I_c \geq 2.6$ and/or $B_q \geq 0.1$

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

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

(12)
This algorithm is specifically applicable for the following ranges of porewater pressure parameter $(0.1 \leq B_q < 1)$ and effective stress friction angles $(20^\circ \leq \phi' < 45^\circ)$.

**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, $\text{YSR} = \sigma_p' / \sigma_{vo}' = \text{yield stress ratio}$. The YSR is in effect the same as overconsolidation ratio (OCR), however, now generalized to accommodate mechanisms of preconsolidation that occur beyond just erosion, glaciation, and removal of overburden stresses, but also due to ageing, desiccation, repeated cycles of wetting-drying, bonding, repeated freeze-thaw cycles, groundwater changes, and other factors.

A generalized approach for evaluating the yield stress or preconsolidation in soils using net cone resistance has been formulated, as presented in Figure 13 (Mayne 2015).

![Fig. 13. Evaluation of yield stress or preconsolidation stress in soils from CPT](image)
The algorithm can be expressed in dimensionless form by:

$$\sigma_p' = 0.33 \cdot (q_t - \sigma_{vo})^{m'} \cdot (\sigma_{atm}/100)^{1-m'}$$  \hspace{1cm} (13)

where \( m' \) = exponent depends on soil type: \( m' = 1 \) (intact clays); \( 0.85 \) (silts); \( 0.80 \) (sandy silts to silty sands), and \( 0.72 \) (sands). For fissured clays, the exponent \( m' \) may be 1.1 or higher, depending upon the age of the formation and degree of jointing and discontinuities. Note that fissured clays can be indentified when \( I_c < 2.6 \) and \( B_q < 0.1 \).

The exponent \( m' \) has also been calibrated with CPT material index, as presented in Figure 14. An algorithm to express this relationship for non-fissured soils is given by:

$$m' = 1 - \frac{0.28}{1 + (I_c / 2.65)^{25}}$$  \hspace{1cm} (14)

This allows the post-processing of CPT to automatically choose the appropriate exponent \( m' \) for a layer by layer analysis.

Figure 14. Yield stress exponent \( m' \) in terms of CPT material index, \( I_c \).
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:

$$K_0 = (1 - \sin \phi') \cdot OCR^{\sin \phi'}$$

(15)

Data compiled from in-situ $K_0$ measurements using self-boring pressuremeter tests (SBPMT), total stress cells (TSC) or push-in spade cells, and/or laboratory methods (instrumented consolidometers, triaxials, and/or suction measurements) on a variety of clays, silts, and sands have been compiled and reported by Ku & Mayne (2015), as shown in Figure 15, verifying equation (15) as a means for evaluating $K_0$ in soils.

![Figure 15. Relationship between lateral stress coefficient $K_0$ and YSR or OCR in soils.](from Ku and Mayne 2015)
Undrained Shear Strength

The loading of soils can occur under conditions of being fully drained ($\Delta u = 0$), partially drained, or full undrained ($\Delta V/V_0$), where $\Delta u =$ excess porewater pressures (above hydrostatic) and $\Delta V/V_0 =$ volumetric strain. Further details on specific stress paths are best explained in terms of critical state soil mechanics, or CSSM (Mayne et al. 2009; Holtz, Kovacs, & Sheahan 2011). The prevailing drainage conditions depend upon the rate of loading and permeability characteristics of the soil. Normally, in sands that are pervious and exhibit high permeability, a drained response occurs. Exception to this may occur in loose sands during fast earthquake loading, resulting in soil liquefaction. In clays that exhibit low permeability, a fast rate of loading will result in undrained loading at constant volume. This, in fact is a temporary and transient condition, often termed short term loading. In the long term, eventually porewater pressures will dissipate (albeit slowly), and a drained response will prevail, thus termed long term loading.

The overall soil strength is controlled by the effective stress strength envelope. Most commonly, this is represented by a simple linear relationship termed the Mohr-Coulomb criterion where the maximum shear stress ($\tau_{\text{max}}$, called the "shear strength) is given as:

$$\tau_{\text{max}} = c' + \sigma' \cdot \tan \phi'$$  \hspace{1cm} (16)

where $c' =$ effective cohesion intercept, $\sigma' =$ effective normal stress, and $\phi' =$ effective stress friction angle. As a starting point, values of $c' = 0$ and $\phi' = 30^\circ$ can be adopted for all soil types, at least until the specific soil type, geologic formation, and results from high-quality laboratory or field test data are available.

Peak Undrained Shear Strength from Stress History

Using simplified critical state soil mechanics, the peak undrained shear strength ($\tau_{\text{max}} = s_u$) can be evaluated from the effective stress strength envelope ($c' = 0$; effective friction angle $\phi'$) and stress history (i.e., OCR) in the form:

$$s_u = (\sin \phi'/2) \cdot (\text{OCR})^{\lambda} \cdot \sigma_{vo}'$$  \hspace{1cm} (17)

DSS: $s_u = (\sin \phi'/2) \cdot (\text{OCR})^{\lambda} \cdot \sigma_{vo}'$
Figure 16. Normalized undrained shear strength from simple shear tests on various clays versus YSR or OCR

which corresponds to a simple shear mode. Figure 16 presents the aforementioned relationship together with data from 17 different clays tested in simple shear.

For the triaxial compression (TC) mode, the equation would give slightly higher strengths that are calculated from:

\[ s_u = \left(\frac{6 \cdot \sin \phi'}{3 - \sin \phi'}\right) \cdot \sigma_{vo}' \cdot (\frac{Mc}{2}) \cdot \left(\frac{OCR}{2}\right)^{\Lambda} \]  

(18)

where \( Mc = (6 \cdot \sin \phi')/(3-\sin \phi') \).
Peak Undrained Shear Strength from CPTu

A more direct approach to assessing undrained shear strength is via bearing capacity theory whereby:

$$s_u = \frac{q_t - \sigma_{vo}}{N_{kt}}$$  \hspace{1cm} (19)

where $N_{kt}$ = bearing factor that depends on mode of shearing, sensitivity of the clay, degree of overconsolidation, and other factors. For soft offshore clays, the backfigured $N_{kt}$ from data collected at 14 well-documented sites (Low et al. 2010) determined mean values based on mode of shearing: $N_{kt} = 11.9$ (triaxial compression), $N_{kt} = 13.6$ (simple shear), and $N_{kt} = 13.3$ (vane).

A recent study of 51 clays that were tested by both field piezocone and laboratory CAUC triaxial tests showed that essentially $N_{kt} = 12$ for intact clays and clayey silts of low to medium sensitivity (Mayne, Peuchen & Baltoukas 2015). Figure 17 shows the slightly different trends for offshore versus onshore clays. For sensitive clays, a lower value of $N_{kt} = 10$ would be appropriate and for fissured clays, a higher value of $N_{kt}$ would be in the range of 20 to 30.

![Figure 17. Database from 51 clays with CPT $q_{net}$ versus lab measured triaxial shear strength (Mayne, Peuchen, & Baltoukas 2015)](image-url)
For soft to firm clays, an alternate means to evaluate undrained shear strength is via the excess porewater pressures ($\Delta u = u_2 - u_0$) from the following:

$$
S_u = \frac{\Delta u_2}{N_{\Delta u}}
$$

(20)

where $N_{\Delta u}$ = porewater bearing factor, also dependent on the aforementioned factors. The study by Low et al. (2010) found $N_{\Delta u} = 5.9$ (triaxial compression), $N_{\Delta u} = 6.9$ (simple shear), and $N_{\Delta u} = 7.1$ (vane shear).

A third alternate is found from the effective cone resistance ($q_E = q_t - u_2$) whereby:

$$
S_u = \frac{q_t - u_2}{N_{kE}}
$$

(21)

where $N_{kE}$ is a bearing term for this approach. Mayne et al. (2015) indicated a mean value of $N_{kE} = 8$ for soft-firm intact clays.

**Remolded Undrained Shear Strength from CPT**

The remolded undrained shear strength ($s_{ur}$) is obtained from either field vane tests, lab mini-vane shear tests, or lab fall cone devices. This affords the evaluation of the clay sensitivity ($S_t$) which is defined as the ratio of peak to remolded strengths at a given water content:

$$
S_t = \frac{s_u(\text{peak})}{s_{ur}}
$$

(22)

For the CPT, the measured sleeve friction ($f_s$) has been noted to give values that are comparable to the remolded undrained shear strength (Powell & Lunne 2015). Therefore,

$$
s_{ur} \approx f_s
$$

(23)
Ground Stiffness and Soil Moduli

The stiffness of the ground can be represented by several geoparameters, including the compressibility indices ($C_r$, $C_c$, $C_s$), spring constants ($k_v$), and moduli. Regarding the latter, there are several moduli that are used in geotechnical engineering, including: shear modulus ($G$ and $G_u$), Young's modulus ($E$ and $E_u$), constrained modulus ($D'$), bulk modulus ($K'$), resilient modulus ($M_R$), and subgrade reaction modulus ($k_s$).

Soil Modulus

The definition of modulus is taken as $E = \Delta \sigma / \Delta \varepsilon$, with Figure 18 showing a typical deviator stress ($q = \sigma_1 - \sigma_3$) versus axial strain ($\varepsilon_a$) curve. Theoretical interrelationships between the elastic moduli: $G$, $E$, $D$, and $K$ have a dependence on the Poisson's ratio, $v$. The value of Poisson's ratio can be taken as $v_u = 0.5$ for undrained loading (i.e., constant volume), while for drained loading which is accompanied by volumetric strains, a value of $v' = 0.2$ may be used. If we adopt the reference modulus as $E'$, then the interrelationships with the other elastic moduli are given by:

a. Reference Stiffness: $E' = \text{drained Young's modulus}$

b. Shear Modulus: $G' = E' / [2(1+\nu')]$

c. Constrained Modulus: $D' = E' (1-\nu') / [(1+\nu')(1-2\nu')]$

d. Bulk Modulus: $K' = E' / [3 \cdot (1-2\nu')]$

\[ q = (\sigma_1 - \sigma_3) \]

Figure 18. Representative stress-strain-strength curve for soils in triaxial compression
For the extreme case when ν' = 0, in fact D' = E'.  When ν' = 0.2, the two moduli are only 10% different: D' = 1.1·E', thus the constrained modulus and drained Young's modulus are often considered somewhat interchangeably.

The resilient modulus (MR) applies to pavement analysis and design, most commonly measured by cyclic triaxial testing under repeated load applications. In fact, MR is a special case of Young's modulus that relates to the small strain stiffness measured in the nondestructive range but has developed permanent plastic strains after many cycles of loading (Brown 1996; Dehler & Labuz 2007).

The subgrade reaction modulus is actually a combined soil-structural parameter, as its value depends on the ground stiffness and the size of the loaded element. The subgrade modulus is defined as: 

$$k_s = q/\delta$$

where q = applied stress and $\delta$ = measured deflection. In terms of elasticity solutions, the deflection of a flexible circle of diameter d is given by: 

$$\delta = q \cdot d \cdot (1-\nu^2)/E'$$

Therefore, 

$$k_s = E'/[d \cdot (1-\nu^2)]$$  \hspace{1cm} (28)$$

which has units of kN/m³ or pcf.

Table 2 summarizes several selected studies towards the approximate evaluation of these moduli obtained directly from measured CPT data. Note however that the results from in-situ penetrometer tests generally represent a peak strength, as indicated in Figure 18. Thus, the measured cone tip resistance (qt) reflects the top of the stress-strain curve, either the undrained strength in clays (su) or the effective friction angle (ϕ') of sands, or even a strength intermediate between these two values. Thus, qt is probably not the best means to obtain the slope of the stress-strain curve. Instead, the SCPTu that provides the initial tangent shear modulus (Gmax) from the shear wave velocity (Vs) is a better choice.
Table 2. Selected Modulus Relationships with Cone Penetration Test Measurements

<table>
<thead>
<tr>
<th>Source</th>
<th>Soils Studied</th>
<th>Reference Modulus</th>
<th>Findings</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kulhawy &amp; Mayne 1990</td>
<td>8 stiff OC clays</td>
<td>Lab constrained modulus</td>
<td>$D' \approx 8.25 \cdot (q_t - \sigma_{vo})$</td>
</tr>
<tr>
<td>Mohammed et al. (2000)</td>
<td></td>
<td>Resilient modulus</td>
<td></td>
</tr>
<tr>
<td>Abu-Farsakh 2004</td>
<td>7 Louisiana sites</td>
<td>Lab constrained modulus</td>
<td>$D' \approx 3.58 \cdot (q_t - \sigma_{vo})$</td>
</tr>
<tr>
<td>Mayne (2007b)</td>
<td>All soil types</td>
<td>Constrained moduli from lab consolidation</td>
<td>First-order estimate: $D' \approx 5 \cdot (q_t - \sigma_{vo})$</td>
</tr>
</tbody>
</table>
| Robertson (2009)              | All soil types          | Constrained modulus from consolidation | $D' = \alpha_m \cdot (q_t - \sigma_{vo})$ when $I_c > 2.2$:  
  1. $\alpha_m = Qtn$ when $Qtn \leq 14$  
  2. $\alpha_m = 14$ when $Qtn > 14$.  
  when $I_c < 2.2$, then:  
  $\alpha_m = 0.03 \cdot 10^{0.55 I_c + 1.68}$ |
| Liu et al. (2016)             | 16 clay sites in China  | Resilient modulus         | $M_R = (1.46q_t^{0.53} + 13.55 f_s^{1.4} + 2.36)^{2.44}$ (all in MPa) |
| Casey et al. (2016)           | 8 clays                 | Triaxial tests for undrained Young's modulus | $E_u/(\sigma_{vc}')^{0.7} = 316 - 2.3 \cdot LL$ where $E_u$ and $\sigma_{vc}'$ (MPa) and LL = liquid limit (%) |

Nonlinear Modulus

The small-strain shear modulus ($G_{max}$ or $G_0$) represents the initial stiffness of all soils and rocks. It is the beginning of all stress-strain-strength curves for geomaterials and obtained from elasticity theory:

$$G_{max} = G_0 = \rho_t \cdot V_s^2$$  \hspace{1cm} (29)

where $V_s =$ shear wave velocity, $\rho_t = \gamma_t/g_a =$ total soil mass density, $\gamma_t =$ total soil unit weight, and $g_a =$ gravitational acceleration constant ($9.8$ m/s$^2$).

From a general viewpoint on stiffness, the shear modulus $G$ of soil is defined as the slope of shear stress versus shear strain: $G = \Delta \tau / \Delta \gamma_s$ for a tangent definition, and by: $G = \tau / \gamma_s$ as a secant...
definition. The shear modulus is related to its associated Young's modulus: \( E = 2G(1+\nu) \). Both moduli are in fact highly nonlinear, ranging from a maximum value at the small-strain stiffness \( G_{\text{max}} = G_0 = \rho_t \cdot V_s^2 \), where \( \rho_t = \text{total soil mass density} \) and \( V_s = \text{shear wave velocity} \) to intermediate \( G \) values at medium strains (\( \approx 1\% \)) to low values at peak strength. As such, a variety of algorithms and formulae have been developed to represent either a partial range or the full stress-strain-strength behavior of soils over a range of interests (e.g., Mayne 2005). In these formulations, a variable number of input parameters may be required in order to produce a stress-strain-strength curve.

A modified hyperbolic form suggested by Fahey & Carter (1993) has favorable attributes in that the modulus reduction factor can be established with only a single variable. This allows the initial stiffness to be small-strain stiffness \( (G_0) \) that is reduced in terms of level of mobilized strength, e.g. \( \tau/\tau_{\text{max}} = q/q_{\text{max}} = 1/FS \), which is simply the reciprocal of the calculated factor of safety (FS). The magnitude of secant shear modulus \( (G) \) corresponding to the particular level of loading is given by:

\[
G = \text{MRF} \cdot G_0 = \frac{G}{G_0} \cdot G_0 \quad (30)
\]

where the \( \text{MRF} = \text{modulus reduction factor} \) determined as:

\[
\text{MRF} = \frac{G}{G_0} = 1 - \left( \frac{\tau}{\tau_{\text{max}}} \right)^g \quad (31)
\]

and \( g = \text{empirical exponent term} \). Figure 19 shows the relationships for normalized shear stress \( (\tau) \) versus shear strain \( (\gamma_s) \) and corresponding normalized secant shear modulus \( (G = \tau/\gamma_s) \) with shear strain over a range of exponent values: \( 0.1 \leq g \leq 1.0 \). For a discussion of tangent \( G \), please see Fahey & Carter (1993).

Using laboratory data from resonant column-torsional shear tests and triaxial specimens with local strain measurements for \( G_{\text{max}} \) reference values, modulus reduction curves for a selection of sands and clays are presented in Figure 20. The data indicate that the exponent \( g \) falls within the range: \( 0.2 < g < 0.5 \) for many soils, tested drained and undrained. A mean value of \( g = 0.3 \) is recommended for preliminary site investigations and designs, until additional information can be obtained.
The value of $\tau_{\text{max}}$ is the shear strength of the soil, generally taken as either: (1) drained ($c' = 0$): $\tau_{\text{max}} = \sigma'_{vo} \cdot \tan \phi'$, or (2) undrained, where $\tau_{\text{max}} = su =$ undrained shear strength, as discussed earlier. Thus, each shear stress ($\tau$) can be associated with its shear modulus ($G$) and the relevant shear strain is found from: $\gamma_s = \tau/G$. This allows for generation of nonlinear stress-strain-strength curves at all depths from SCPTu data, in clays, sands, and mixed soil types.

Figure 19. Normalized modulus ($G/G_{\text{max}}$) and normalized shear stress ($\tau/\tau_{\text{max}}$) versus shear strain ($\gamma_s$) for modified hyperbolic relationship of Fahey & Carter (1993).
**Coefficient of Consolidation**

The rate at which foundation and embankment settlements occur, as well as the dissipation of excess porewater pressures, is controlled by the coefficient of consolidation \( (c_v) \). The magnitude of \( c_v \) is also required in the design of vertical wick drains that can be installed in soft ground to expedite the time for consolidation. Using the results of CPT\(u\) dissipation tests, which measure the rate at which the \( u_2 \) readings vary with time, the in-situ profile of \( c_v \) can be evaluated. Often, the piezocone-interpreted values of \( c_v \) are validated by comparison with results from laboratory one-dimensional consolidation tests (e.g. Abu-Farsakh, M.Y. 2004). Alternatively, a better method is to cross-check the values with the measured full-scale performance of instrumented embankments that are constructed over the ground and the recorded time-rate of consolidation and settlements can provide the best \( c_v \) for that site (Abu-Farsakh, M.Y., et al. 2011).

**Monotonic Dissipation Tests**

An illustrative dissipation curve from piezocone testing at IDT State Highway 95 at an embankment and bridge crossing is presented in Figure 21. After the CPT\(u\) sounding was halted
at a depth of 51.2 feet, the recorded decay of porewater pressures was observed to be monotonic with time until the dissipations were ended at 1000 seconds. During that time, the $u_2$ readings decreased from 115 psi to 47 psi. At this location, the depth to the groundwater table is about 16 feet, thus the calculated equilibrium water pressure is 15 psi. Commonly, a characteristic time for piezo-dissipations is taken at 50% degree of consolidation, although other degrees may be adopted. By evaluating the value of $u_2$ at 50%, as shown in Figure 21, the characteristic $t_{50} = 404$ s can be obtained.

It is also common to plot normalized excess porewater pressures relative to the measured initial $\Delta u_2$ that is obtained during penetration at the constant rate of 20 mm/s, i.e. $\Delta u/\Delta u_i$ versus time. Figure 22 shows a set of piezo-dissipation records for a bridge and embankment site in southern Louisiana reported by the LTRC. While the porewater pressure axis is arithmetic, the time scale is often plotted on either logarithmic or square root scales. In either case, the $t_{50}$ is then found from the measured dissipation curve when $\Delta u/\Delta u_i = 0.50$. 

Figure 21. Piezo-dissipation at Sandpoint Bridge Crossing, Idaho for depth $z = 51.2$ feet illustrating the evaluation of characteristic time $t_{50}$
Dilatory Dissipation Curves

In a number of situations, a monotonic type dissipation does not occur on the onset but instead a dilatory type curve is observed. Here, after stopping the push of the penetrometer, the recorded porewater pressures initially begin to rise and eventually reach a peak value, then afterwards follow a phase where the $\Delta u$ values decrease to hydrostatic (Figure 23). A full solution is available for both cases (Burns & Mayne 2002). Dilatory responses are most often associated with overconsolidated clays and silts, although occasionally are observed in fine-grained soils with low OCRs.

The selection of the characteristic $t_{50}$ value may found by using a square root of time plot to find the initial $u_2$ value by projection of the post-peak dissipatory data back to the ordinate axis, as shown by the example in Figure 24. In this example, the initial $u_2 = 480$ kPa, and then the same procedures in Figure 21 may apply.
Figure 23. Monotonic and dilatory porewater dissipation responses in soils
(after Burns & Mayne 1998)

Figure 24. Method for obtaining $t_{50}$ from dilatory dissipation curves
(Schneider & Hotstream 2010)
Interpretation of Dissipation Test Data

There are a number of different solutions available for the interpretation of the coefficient of consolidation ($c_v$) from the piezocone dissipation curves. For monotonic type response, the strain path method (SPM) developed at Oxford University (Teh & Houlsby 1991) is well-recognized, while the Georgia Tech solution by Burns & Mayne (2002) is based on spherical cavity expansion and critical state soil mechanics (SCE-CSSM) and handles both monotonic and dilatory curves. For both solutions, a simplified procedure can be recommended that relies on the aforementioned $t_{50}$ value obtained from the measured field data, as given in Table 3. The units for $c_v$ are in cm$^2$/s, as shown, or equivalent.

Table 3. Recommended procedures for calculating $c_v$ from $t_{50}$ obtained in dissipation tests

<table>
<thead>
<tr>
<th>Method</th>
<th>Equation for coefficient of consolidation, $c_v$</th>
<th>Remarks/Notes</th>
<th>Eqn</th>
</tr>
</thead>
</table>
| SPM                     | $c_v = \frac{0.245 \cdot (a_c)^2 \cdot \sqrt{I_R}}{t_{50}}$ | $t_{50}$ = measured time to reach 50% dissipation  
$I_R = G/s_u$ = undrained rigidity index  
$G =$ shear modulus  
$s_u =$ undrained shear strength  
$a_c =$ penetrometer radius ($a_c = 1.78$ cm for 10-cm$^2$ cone; $a_c = 2.20$ for a 15-cm$^2$ size) | (32) |
| SCE-CSSM                | $c_v = \frac{0.030 \cdot (a_c)^2 \cdot (I_R)^{0.75}}{t_{50}}$ | $t_{50}$ = measured time to reach 50% dissipation  
$I_R = G/s_u$ = undrained rigidity index  
$G =$ shear modulus  
$s_u =$ undrained shear strength  
$a_c =$ penetrometer radius ($a_c = 1.78$ cm for 10-cm$^2$ cone; $a_c = 2.20$ for a 15-cm$^2$ size) | (33) |

Evaluating rigidity index

Both the SPM and SCE-CSSM solutions require an estimate of the in-situ rigidity index ($I_R = G/s_u$) of the soil. If the results of SCPTU are available, Krage et al. (2014) have derived an expression for $I_R$ that depends upon the small-strain shear modulus, net cone tip resistance, and effective overburden stress:
\[ I_{R50} = 1.811 \cdot \frac{G_{\text{max}}}{q_{\text{net}}^{0.75} \cdot \sigma'_{\text{vo}}^{0.25}} \]  

(31)

where consistent units are input for \( G_{\text{max}} \), \( q_{\text{net}} \), and \( \sigma'_{\text{vo}} \). The value of \( G_{\text{max}} \) is obtained from (29) using the measured shear wave velocity (\( V_s \)) and unit weight evaluated from (7).

If the shear wave velocity is not measured, it can be estimated from the CPT data. A number of methods have been reviewed for CALTRANS by Wair et al. (2012), including one that is applicable to sands, silts, and clays of low sensitivity that are inorganic and uncemented:

\[ V_s (\text{m/s}) = [10.1 \cdot \log(q_t) - 11.4]^{1.67} \cdot (100 \cdot f_s/q_t)^{0.3} \]  

(32)

where \( q_t \) is in units of kPa (Hegazy & Mayne 1995). An alternative relationship is given by Robertson & Cabal (2015).

**Hydraulic Conductivity**

The hydraulic conductivity is a parameter that expresses the flow characteristics of the soil. In geotechnics, it is also called the coefficient of permeability (\( k \)) and has units of cm/s, or feet/day. Through consolidation theory, the hydraulic conductivity relates directly to the coefficient of consolidation:

\[ k = \frac{c_v \cdot \gamma_w}{D'} \]  

(33)

where \( \gamma_w \) = unit weight of water and \( D' = \) constrained modulus. Therefore, one approach to evaluating \( k \) can be from the site-specific \( c_v \) obtained from the dissipation tests using an estimate of \( D' \) from one of the relationships given in Table 2.

An approach developed for soft normally-consolidated soils that uses \( t_{50} \) directly to assess the magnitude of \( k \) is presented in Figure 25 (Parez and Fauriel 1988). The mean trendline through the wedges of soil type can be approximated by:

\[ k_h (\text{cm/s}) \approx \left( \frac{1}{251 \cdot t_{50}(\text{sec})} \right)^{1.25} \]  

(34)
Figure 25. Hydraulic conductivity versus dissipation time for 50% consolidation
(after Parez and Fauriel 1988)

REFERENCES


INTRODUCTION

The axial response of deep foundations includes the load-displacement-capacity and axial load transfer when driven pilings and drilled shafts are loaded in compression and uplift. The use of cone penetration testing (CPT), especially seismic piezocone tests (SCPTu), are advantageous since they provide information on the subsurface soils and their geomaterial properties. The SCPTu provides at least four measurements on soil behavior with depth, including: (a) cone tip resistance, \( q_t \), (b) sleeve friction, \( f_s \), (c) penetration porewater pressure, \( u_2 \), and (d) shear wave velocity, \( V_s \). This offers the opportunity to determine the geostratigraphy, unit weight, effective overburden stress, shear strength, stress state, and stiffness of the ground from a single exploratory sounding, thus values in economic, expediency, and reliability. The axial pile capacity can be calculated on the basis of static equilibrium of forces acting along the sides and base of the pile foundation. The displacement of the pile can be ascertained using elasticity theory from closed-form solutions, boundary elements, and/or finite element analyses. Load transfer occurs along the length of the pile and only a portion of axial forces are transmitted to the base or toe or tip of the pile. These too can be assessed using elasticity solutions.

An alternate approach to pile capacity involves the utilization of direct CPT methods, available since the 1970's, but in the past 10+ years a number of new and statistically reliable algorithms have been developed which can implemented for highway design and construction.

AXIAL PILE CAPACITY

The axial compression capacity of a single pile foundation is composed of a shaft or side component and end-bearing component at the base, as depicted in Figure 1. For a circular pile, the side capacity \( Q_s \) is determined from the unit side friction \( f_p \) acting along the surface area of the shaft which is: \( A_s = \pi d L \), where \( d \) = pile diameter and \( L \) = length embedded below grade. If the magnitude of side friction is uniform and constant with depth, the side capacity is simply:

\[
(1) \quad Q_s = f_p \cdot A_s
\]

Moreover, however, many piles extend through multiple layers and a summation of unit side frictions acting on various pile segments must be tabulated over the length of the pile, as suggested by Figure 1.

The unit-end bearing resistance \( q_b \) acts over the base of the pile tip, where the area of a circular pile is given by \( A_b = \pi d^2/4 \). For piles in compression loading, the base capacity is determined from:

\[
(2) \quad Q_b = q_b \cdot A_b
\]

and for piles in tension (or uplift), it is normally taken that \( Q_b = 0 \).

---

**Figure 1. Components of Axial Pile Capacity**

\[ Q_t = Q_s + Q_b \]

\[ Q_s = \int f_p \, dA_s \]

\[ Q_b = q_b \cdot A_b \]
PILE UNIT SIDE FRICTION

A number of different approaches can be adopted for evaluating the unit pile side friction ($f_p$) prior to full-scale load testing and construction (Poulos & Davis 1980; O'Neill 2001). The most common types including the alpha and beta methods, as summarized in Tables 1 and 2, respectively. In the alpha method, an empirical coefficient ($\alpha$) is applied to the undrained shear strength ($s_u$) of clay soils to obtain:

$$f_p = \alpha \cdot s_u$$  \hspace{1cm} (3)

An illustrative example of alpha curves is shown in Figure 2. A difficulty with the alpha method is the evolution of many variants and changes to the expressions for its estimation. It is also restricted in its use for specific types of piles in clays and fine-grained soils.

In the beta method, the coefficient $\beta$ is applied to the effective overburden stress ($\sigma_{vo}'$) at the point of concern along the pile length:

$$f_p = \beta \cdot \sigma_{vo}'$$  \hspace{1cm} (4)

![Figure 2. Selected alpha relationships as function of the undrained shear strength](image)
Table 1. Alpha Methods for Axial Pile Side Friction where $f_p = \alpha \cdot s_u$

<table>
<thead>
<tr>
<th>Reference Source</th>
<th>Alpha Equation</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tomlinson (1957)</td>
<td>$\alpha = 0.11 \cdot s_u^2 - 0.68 \cdot s_u + 1.27$</td>
<td>All pile types (steel, concrete, timber) Note: $s_u$ in ksf</td>
</tr>
<tr>
<td>Tomlinson (1957)</td>
<td>$\alpha = 0.11 \cdot s_u^2 - 0.62 \cdot s_u + 1.26$</td>
<td>Concrete piles Note: $s_u$ in ksf</td>
</tr>
<tr>
<td>McClelland (1974)</td>
<td>Kerisel: $\alpha = 0.7 - 0.31 \cdot \ln(s_u)$ Peck: $\alpha = 1.0 - 0.2 \cdot s_u$ Woodward: $\alpha = 0.91 \cdot (s_u)^{0.91}$</td>
<td>Driven piles in clay Note: $s_u$ in ksf</td>
</tr>
<tr>
<td>Semple (1980)</td>
<td>$\alpha = 1 - \frac{(OCR - 1)}{1 + 1.5 \cdot (OCR - 1)}$</td>
<td>Summary from 9 series of pile load tests</td>
</tr>
<tr>
<td>American Petroleum Institute (API 1981)</td>
<td>For $s_u/\sigma_{vo}' \leq 0.35$: $\alpha = 1.0$ For $s_u/\sigma_{vo}' &gt; 0.80$: $\alpha = 0.5$ Otherwise: $\alpha = 1 + 1.111 \cdot (0.35 - s_u/\sigma_{vo}')$</td>
<td>Driven steel pipe piles</td>
</tr>
<tr>
<td>Tomlinson (1986)</td>
<td>1. $\alpha = 55 \cdot (s_u)^{0.91}$ 2. $\alpha = 78.5 \cdot (s_u)^{1.02}$ 3. $\alpha = 60.5 \cdot (s_u)^{1.00}$ where $75 &lt; s_u$ (kPa) &lt; 210</td>
<td>1. Driven piles 2. Bored piles 3. Driven piles in till with $L &lt; 10 \cdot d$</td>
</tr>
<tr>
<td>API (1987)*</td>
<td>1. $\alpha = 1.0$ for $s_u &lt; 25$ kPa 2. $\alpha = 1.25 - 0.01 \cdot s_u$ for $25 &lt; s_u &lt; 75$ kPa 3. $\alpha = 0.50$ for $s_u &gt; 75$ kPa</td>
<td>Driven piles in clay other than Gulf of Mexico</td>
</tr>
<tr>
<td>Kulhawy &amp; Jackson (1989)</td>
<td>$\alpha = 0.21 + 0.26 \cdot (\sigma_{atm}/s_u) \leq 1$</td>
<td>Analyses of 106 drilled shaft foundations</td>
</tr>
<tr>
<td>API (1989)</td>
<td>For $s_u/\sigma_{vo}' \leq 1$: $\alpha = \frac{0.5}{\sqrt{(s_u/\sigma_{vo}')}}$ For $s_u/\sigma_{vo}' &gt; 1$: $\alpha = 0.5 \cdot (s_u/\sigma_{vo}')^{-0.25}$</td>
<td>Driven steel pipe piles</td>
</tr>
<tr>
<td>Kulhawy &amp; Jackson (1989)</td>
<td>$\alpha = (1 - \sin \phi') \cdot \tan \phi' \cdot OCR^{\sin \phi'} \cdot 0.5 \cdot \sin \phi' \cdot OCR \cdot \Lambda$</td>
<td>$\Lambda = (1 - C_s/C_c) = 0.80$ for insensitive clays</td>
</tr>
<tr>
<td>Nowacki et al. (1996)</td>
<td>For $s_u/\sigma_{vo}' \leq 0.7$: $\alpha = 0.5 \cdot (s_u/\sigma_{vo}')^{-0.5}$ For $s_u/\sigma_{vo}' &gt; 0.7$: $\alpha = 0.55 \cdot (s_u/\sigma_{vo}')^{-0.2}$</td>
<td>Driven pile foundations</td>
</tr>
<tr>
<td>Kolk and van der Velde (1996)</td>
<td>$\alpha = 0.9 \cdot F_L \cdot (s_u/\sigma_{vo}')^{0.3} \leq 1.0$ $F_L = [(L - z)/d]^{-0.2} = \text{length term}$ $L = \text{pile length}$ $z = \text{depth at point considered}$ $d = \text{outside diameter of pile}$</td>
<td>Note: $s_u$ obtained from UU lab tests Applicable to driven piles in clays</td>
</tr>
<tr>
<td>Karlsrud et al. (2005, NGI Method)</td>
<td>$\alpha = \text{function (} s_u/\sigma_{vo}' \text{ and plasticity index) }$</td>
<td>Driven pilings</td>
</tr>
</tbody>
</table>
For driven piles in clay where \((s_u/\sigma_{vo'})_{NC}\) is the normalized shear strength for normally-consolidated clay

Brown, et al. (2010)

\[ \alpha = \begin{cases} 0 & \text{for } 0 < z \leq 5 \text{ feet} \\ 0.55 & \text{for } z > 5 \text{ feet and } (s_u/\sigma_{atm}) \leq 1.5 \\ 0.55 - 0.1 \times (s_u/\sigma_{atm} - 1.5) & \text{for } 1.5 \leq (s_u/\sigma_{atm}) \leq 2.5 \end{cases} \]

Drilled Shafts and Bored Piles

Knappett & Craig (2012)

\[ \alpha = \begin{cases} 1 & \text{for } s_u \leq 30 \text{ kPa} \\ 1.16 \times (s_u/185) & \text{for } 30 \leq s_u \leq 150 \text{ kPa} \\ 0.35 & \text{for } s_u > 150 \text{ kPa} \end{cases} \]

Non-displacement piles in fine-grained soils

Knappett & Craig (2012)

\[ \alpha = 0.55 \times \left( \frac{40}{(L/d)} \right)^{0.2} \times (s_u/\sigma_{vo'})^{-0.3} \]

Displacement piles in fine-grained soils

Karlsrud (2012)

\[ \alpha = \text{function } (s_u/\sigma_{vo'}) \text{ and plasticity index} \]

Driven pilings

*Note: as reported by Karlsrud (2012)

Table 2. Beta Methods for Axial Pile Side Friction where \(f_p = \beta \cdot \sigma_{vo'}\)

<table>
<thead>
<tr>
<th>Reference Source</th>
<th>Beta Equation</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Burland (1973)</td>
<td>(\beta = (1-\sin\phi') \cdot \tan \phi')</td>
<td>Piles in NC clays</td>
</tr>
<tr>
<td>Meyerhof (1976)</td>
<td>(\beta = K \cdot \tan \delta)</td>
<td>where (K = K_0) for NC clays, (K = 1.5 \cdot K_0) for OC clays (K_0 = (1-\sin\phi') \cdot \sqrt{OCR})</td>
</tr>
<tr>
<td>Poulos &amp; Davis (1980)</td>
<td>(\beta = (1-\sin\phi') \cdot \tan \phi' \cdot OCR^{0.5})</td>
<td>Piles in OC stiff clays</td>
</tr>
<tr>
<td>Kulhawy &amp; Jackson (1989)</td>
<td>(\beta = (1-\sin\phi') \cdot \tan \phi' \cdot OCR^{\sin\phi'}/\delta)</td>
<td>Piles in quartz sands and insensitive clays</td>
</tr>
<tr>
<td>O’Neill (2001)</td>
<td>(\beta = (1-\sin\phi') \cdot \tan \theta' \cdot OCR^{\sin\phi'}/\delta)</td>
<td>Drilled shafts where (\theta = (\delta/\phi')\cdot\phi') and (\delta) = interface friction between pile and soil</td>
</tr>
<tr>
<td>Fleming et al. (2009)</td>
<td>(\beta = K \cdot \tan \delta)</td>
<td>(K = ) lateral stress coefficient and (\delta = ) friction angle between soil and pile material</td>
</tr>
<tr>
<td>Karlsrud (2012)</td>
<td>(\beta = \text{fctn (OCR and PI)})</td>
<td>(\text{PI} = ) plasticity index of the clay (%)</td>
</tr>
<tr>
<td>Mayne and Niazi (2017)</td>
<td>(\beta = C_M \cdot C_k \cdot K_0 \cdot \tan\phi') where (K_0 = (1-\sin\phi') \cdot OCR^{\sin\phi'}/\delta) for soils that are virgin loaded then unloaded</td>
<td>(C_M = ) pile material factor = 1 (drilled; augered); 0.9 (prestressed or precast concrete); 0.8 (timber); and 0.7 (steel); and (C_k = ) pile installation factor = 0.9 (bored or augered); 1.0 (low displacement, e.g. H-pile or open end pipe); and 1.1 (driven high displacement, e.g. prestressed concrete, closed-end pipe)</td>
</tr>
</tbody>
</table>
As the beta approach applies to all types of soils (gravels, sands, silts, and clays) and more or less has remained unchanged since its advent circa 1970, it has been selected for further discussion herein. In the direct form for calculation of unit pile side friction, the expression is given by:

\[ f_p = C_M \cdot C_K \cdot (1 - \sin \phi') \cdot OCR^{\sin \phi'} \cdot \tan \phi' \cdot \sigma_v' \]

where \( C_M \) is a soil-pile interface friction coefficient and \( C_K \) = pile installation factor. Values of \( C_M \) are in the range: \( 0.7 < C_M < 1.0 \) depending upon pile material (steel, wood, concrete) and ranges for \( C_K \) vary: \( 0.9 < C_K < 1.1 \) depending upon method of installation (auger, drill, driven), as detailed in Table 2.

The value of \( K_0 \) is limited to the passive stress coefficient which for the simple Rankine case is given by: \( K_P = (1+\sin \phi')/(1 - \sin \phi') \). Thus there is a maximum value of overconsolidation ratio for which equation (5) applies, given by: \( OCR_{\text{limit}} = \left((1+\sin \phi')/(1 - \sin \phi')\right)^{2/(1+\sin \phi')} \)

The corresponding graph in Figure 3 shows the relationship for beta in terms of \( \phi' \) and OCR.

![Figure 3. Pile side friction coefficient \( \beta \) in terms of effective friction angle and overconsolidation ratio](image)

**PILE UNIT END BEARING**

*Theoretical Considerations*

The evaluation of the end bearing resistance of pile foundations is commonly determined using limit plasticity theory where:

\[ q_b = N_q \cdot \sigma_v' \] (drained loading)

\[ q_b = N_c \cdot s_u \] (undrained loading)
where the value of $\sigma_{vo}'$ is calculated at the depth $z = L$ and the value of $s_u$ is the average undrained shear strength from $z = L$ to a depth $z = L + d$ beneath the pile tip. For a circular pile, the limit plasticity solution for undrained loading gives (Vesic 1977) a value $N_c = 9.33$ while a deep strip foundation would employ a value of $N_c = 8.24$. For drained loading, the expression for $N_q$ for a deep foundation is given by (Vesic 1977):

$$N_q = \exp(\pi \cdot \tan \phi') \frac{1 + \sin \phi'}{1 - \sin \phi'} \left\{1 + \tan \phi' \left(\frac{B}{A}\right)\right\} \cdot \left[1 + 2 \tan \phi' (1 - \sin \phi')^2 \arctan(L / B)\right]$$

where $A$ and $B$ are the pile plan dimensions (for a circular pile, $A = B$) and $L =$ pile length. For a circular pile, this can be approximated by:

$$N_q \approx 0.77 \cdot \exp(\psi'/7.5^\circ)$$

over a range of effective friction angles: $20^\circ \leq \psi' \leq 45^\circ$.

**Practical Considerations**

For drained loading of sands, the full calculated end bearing capacity will never be realized because it would require the pile to move a distance equal to its diameter. This is beyond practical use and therefore the limit plasticity solutions must be clipped to a fraction of the calculated value. Another reason for using a reduced end-bearing capacity is due to strain incompatibility since the side capacity is mobilized early while the end-bearing is engaged much later. So, to have compatible values of $Q_s$ and $Q_b$, the $q_b$ must be reduced using the following guidelines (Randolph 2003; Mayne 2007):

$$q_b = N_q \cdot \sigma_{vo}' \cdot f_x'$$

where $f_x'$ = strain incompatibility factor
- $f_x' = 0.10$ for drilled shafts, augered cast-in-place, and bored piles
- $f_x' = 0.20$ for driven low displacement piles (opened-ended steel pipe and H-piles)
- $f_x' = 0.30$ for driven high-displacement piles (i.e., solid piles, PSC, and closed-ended pipe)

For undrained loading of circular piles in compression, no reduction of end-bearing is necessary, therefore:

$$q_b = 9.33 \cdot s_u$$

**DIRECT CPT METHODS FOR PILE CAPACITY**

In the direct CPT method, the penetrometer readings are scaled directly via specified algorithms to obtain the pile unit side friction and end-bearing, as depicted in Figure 4. As many as 40 different direct CPT methods have been developed over the past five decades, as summarized by Niazi & Mayne (2013). Starting circa 1970, many of these early methods relied on hand-recorded data from field mechanical CPTs where only $q_c$ data were obtained at 20 cm intervals, or later with mechanical readings of both $q_c$ and $f_s$ using special sets of inner and outer rods that recorded vertical load for tip and loads for tip plus sleeve in alternating increments. Also, early pile load test data were often obtained from top-down measurements of load-displacement.
Beginning in the mid-1990’s, as the modern electric piezocone (CPTu) was implemented, the newer equipment offered improved data and better resolution because of the additional reading of porewater pressures and correction of raw measured $q_s$ to total resistance $q_t$. In addition, the use of electronic digital data collection and field computers proved superior in field measurements and recordings. As a consequence, several reliable CPT methods for axial pile capacity have been developed. Moreover, parallel improvements in full-scale pile load testing occurred and now include modern strain gage instrumentation, digital data recording, and automated testing procedures. Also, the testing can be single direction (compression or tension) or bi-directional, as in the Osterberg cell.

Table 3 provides a selection of recent direct CPT methods that have become available over the past two decades. A number of these (namely ICP, NGI, UWA, and Fugro) were funded by the offshore industry because of the growth of oil & gas reserves and windfarm installations, thus necessitating increased concerns on risk, probability, and reliability in the site investigations for offshore platforms and design of large driven monopile foundations. These direct CPT methods are often statistically based on large datasets compiled from full-scale load tests made worldwide (e.g., Schneider et al. 2008).
Table 3. Selection of Direct CPT Methods for Axial Pile Capacity

<table>
<thead>
<tr>
<th>Method</th>
<th>Pile Types</th>
<th>Soil Types</th>
<th>References</th>
<th>CPT data</th>
<th>Additional parameters needed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unicone Method</td>
<td>all types</td>
<td>Sands, silts, clays</td>
<td>Eslami and Fellenius (1997); Fellenius (2009)</td>
<td>$q_t$, $f_s$, $u_2$</td>
<td></td>
</tr>
<tr>
<td>KTRI = Kajima Technical Research Institute</td>
<td>all types</td>
<td>Sands, mixed, clays</td>
<td>Takesue et al. (1998)</td>
<td>$f_s$ and $u_2$</td>
<td>No guidance given on end bearing resistance</td>
</tr>
<tr>
<td>Imperial College Procedure (ICP)*</td>
<td>OE and CE</td>
<td>Sands</td>
<td>Chow et al. (1997; PhD) Jardine et al. (2005)</td>
<td>$q_t$</td>
<td>Interface friction angle ($\delta$) from ring shear tests; Correlated to mean grain size ($D_{50}$)</td>
</tr>
<tr>
<td></td>
<td>OE and CE</td>
<td>Clays</td>
<td>Jardine et al. (2005)</td>
<td>$q_t$</td>
<td>Interface friction angle ($\delta$); correlated to PI</td>
</tr>
<tr>
<td>NGI Method (Norwegian Geotechnical Institute)</td>
<td>OE and CE</td>
<td>Sands</td>
<td>Clausen et al. (2005)</td>
<td>$q_t$</td>
<td>$D_R$ from $q_t$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clays</td>
<td>a. Alpha method (Karlsrud et al. 2005; 2012)</td>
<td>$q_t$</td>
<td>$s_u$ for $q_t$; $q_t$ for $OCR$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>b. Beta method (Karlsrud 2012)</td>
<td></td>
<td>$PI = \text{plasticity index (%)}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$PI = \text{plasticity index (%)}$</td>
</tr>
<tr>
<td>Fugro Method</td>
<td>OE and CE</td>
<td>Sands</td>
<td>Kolk et al. (2005)</td>
<td>$q_t$</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clays</td>
<td>Van Dijk and Kolk (2011)</td>
<td>$q_t$</td>
<td></td>
</tr>
<tr>
<td>UWA (Univ. of Western Australia)</td>
<td>OE and CE</td>
<td>Sands</td>
<td>Lehane et al. (2005)</td>
<td>$q_t$</td>
<td>IFR = infilling ratio related to amount of plugging $R_s = \text{pile roughness}$</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Clays</td>
<td>Lehane et al. (2012)</td>
<td>$q_t$</td>
<td>Interface friction angle ($\delta$) from ring shear tests; and also method without $\delta$</td>
</tr>
<tr>
<td>HKU Method (Hong Kong Univ.)</td>
<td>OE and CE (end resistance only)</td>
<td>Sands</td>
<td>Yu and Yang (2012)</td>
<td>$q_t$</td>
<td>End bearing only. PLR = plug length ratio Note: PLR can be estimated from OE inner diam.</td>
</tr>
<tr>
<td>Purdue LFRD</td>
<td>Drilled shafts</td>
<td>Sands</td>
<td>Basu &amp; Salgado (2012)</td>
<td>$q_t$</td>
<td>LRFD = load resistance factored design</td>
</tr>
<tr>
<td>Enhanced Unicone Method</td>
<td>Various</td>
<td>Sands, silts, clays, and mixed soils</td>
<td>Niazi and Mayne (2015, 2016)</td>
<td>$q_t$, $u_2$, and $f_i$</td>
<td>Based on 330 load tests, including bored, augered, jacked, and driven piles</td>
</tr>
</tbody>
</table>

*Note: previously called "Marine Technical Directorate" (MTD)
Many of the offshore CPT methods relate to driven piles, either in sand or clay, as that is common deep foundation for those purposes. Of particular interest are the Unicone and Modified Unicone Methods, since they use all three readings of the piezocone (qt, fs, and u2) and address a variety of pile foundation types.

**Modified UniCone Method**

The modified UniCone Method was developed based on a total 330 pile load tests which were associated with SCPTu data during their site investigations (Niazi and Mayne, 2015, 2016). This represents a threefold increase over the original UniCone database that was built upon data from 106 pile load tests (Eslami & Fellenius 1997).

For the original UniCone algorithms, use is made of the effective cone resistance (qE):

\[
q_E = q_t - u_2
\]

and a chart of qE vs f, provided an approximate soil classification in five distinct groups, as shown by Figure 5a. Later, in the modified approach, a better delineation of the larger dataset gave soil subclassifications, as indicated by Figure 5b.

![Figure 5. Soil behavior type using CPT via (a) original UniCone; and (b) Modified Unicone charts](image)

In the modified approach, the 9-zone normalized soil behavioral type (SBTn) is ascertained using CPT data in conjunction with the Robertson (2009) charts, as shown in Figure 6. As discussed in Tasks 1 and 2, the SBTn method uses the normalized cone resistance (Qtn), normalized sleeve friction (F), and CPT material index, Ic. This permits a much wider range in the calculated pile side friction because fp is related as a continuous curve with Ic, rather than only 5 values that are assigned in the original scheme.

The pile unit side friction (fp) is obtained from qE and the CPT material index, Ic, using the following expression at each elevation along the sides of the pile:

\[
f_p = q_E \cdot \theta_{PT} \cdot \theta_{TC} \cdot \theta_{RATE} \cdot 10^{\left(0.732 \cdot Ic - 3.605\right)}
\]
where $\theta_{PT} = \text{coefficient for pile type (0.84 for bored piles; 1.02 for jacked piles; 1.13 for driven piles), }$ $\theta_{TC} = \text{coefficient for loading direction (1.11 for compression and 0.85 for tension); and } \theta_{RATE} = \text{is rate coefficient applied to soils in SBT zones 1 through 7 (1.09 for constant rate of penetration tests and 0.97 for maintained load tests).}$ Since CPT provides data at regular intervals of 2 cm to 5 cm along the sides of the pile, the average $f_p$ from $z = 0$ to $z = L$ can be used directly in equation (1) to obtain the shaft capacity $Q_s$.

The pile end bearing resistance is obtained from:

$$q_b = q_E \cdot 10^{0.325 \cdot I_c - 1.218}$$

where $q_E$ is averaged in the vicinity of the pile tip. Figure 7 shows the Modified Unicone Method in graphical format. For sensitive clays of zone 1, please see additional discussions by Niazi & Mayne (2016).
AXIAL PILE DISPLACEMENTS

The movement of pile foundations can be assessed using elastic continuum theory (Poulos & Davis 1980; Randolph 2003; Mayne & Niazi 2017). These relationships have been developed using finite element analyses, boundary elements, and analytical closed-form solutions. For the latter approach, the top displacement of a rigid pile subjected to a vertical force is shown in Figure 8. This gives the movement at the top of a pile subjected to either compression or tension (uplift) loading. Also, the percentage of axial load transferred to the pile toe is determined.

For piles extending through various soil layers, the elastic solution can be implemented by using a set of stacked pile segments, each with its own stiffness, as represented by a soil Young's modulus.

For pile groups, the use of computer software is recommended. Several available programs can handle pile groups under axial and lateral / moment loading, such as DEFPILG (Univ. Sydney) and PIGLET (Univ. Western Australia). A full listing of pile foundation software is given at the Geotechnical & GeoEnvironmental Service Directory: www.ggsd.com
**Rigid Pile Response**

Randolph Elastic Solution

Figure 8. Elastic continuum solution for axial pile response under compression and tension loading

**REFERENCES:**


Identification of soft organic soils from piezocone penetration soundings

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INTRODUCTION

The identification of soil type using cone penetration tests (CPT) is usually done on the basis of empirical soil behavioral type (SBT) charts that use the measured readings \(q_t\), \(f_s\), and/or \(u_2\). While there are at least 25 different sets of charts available (Hegazy 1998; Eslami et al. 2017; Valsson 2016), one of the most widely used and popular is the 9-zone SBT system that uses normalized cone readings \(Q\), \(F\), and \(B_q\) that is detailed by Robertson & Cabal (2007) and Lunne et al. (1997), as documented in MnDOT CPT manual Chapters 1 and 2. The system is denoted as SBTn and plotted on charts in terms of \(Q\) vs. \(F\) and \(Q\) vs \(B_q\), as shown below.

![9-Zone Soil Behavioral Type Chart](image)

Of particular concern in geotechnical site characterization is the proper identification of soft organic soils, such as organic clays and silts (OL and OH) and peats (Pt). These often have difficulties and problems during construction and performance of civil engineering works because of bio-degradation of the geomaterials, long-term creep, excessive settlements, and/or other issues.

The SBTn system has a category labelled Zone 2 that intends to recognize organic soils. However, several recent studies have noted that data from CPTu soundings do not always fall within the bounds delineated using Zone 2, even though laboratory tests and field inspections clearly note the presence of organic soils. Lab tests to classify organic soils includes loss on ignition (LOI > 5%) and two pairs of Atterberg limits testings per ASTM standards, as well as the visual-manual identification of strong organic odor and smell, and dark color, notably black and dark gray.
Results by Mlynarek et al. (2014) show that organic soils in Poland are not adequately identified by the Q-F chart, as seen in Figure 2. Moreover, studies by Zawrzykraj et al. (2017) found that neither the Q-F and Q-B\textsubscript{g} plots worked well to recognize organic soils and peats in Poland. Nejaim et al. 2016 found that Brazilian soft organic clays were mis-classified as Zone 3 (clays), rather than Zone 2 organic clays.

Similarly, a recent study on CPTu data from organic clays located at 24 sites in the USA, Sweden, Mexico, Brazil, and Australia have been compiled (Agaiby 2018). These data are shown to generally avoid falling with the Zone 2 boundaries in either Q-F or Q-B\textsubscript{g} charts, thus are not recognized during these soundings, as indicated by Figure 3. The exception in this case is the Mexico City clay which is correctly identified, however the other 23 sites are generally not properly recognized and categorized. Additional findings by Missiaen et al. (2015) found that the CPTu results in Belgian soils also did not identify organic clays and peats in the non-normalized version of these charts that uses q\textsubscript{t} versus friction ratio (R\textsubscript{f} = 100·f\textsubscript{s}/q\textsubscript{t}), as detailed by Robertson & Cabal (2007).

Figure 2. Soil behavioral chart with superimposed CPTu data from Polish organic soils (from Mlynarek et al. 2014).
SIMPLIFIED YIELD STRESS EVALUATION IN CLAYS BY CPTu

From the development of a hybrid formulation of spherical cavity expansion (SCE) theory with critical-state soil mechanics (CSSM), the effective preconsolidation stress, or yield stress \( \sigma_p' \), of clays can be expressed in terms of three piezocone parameters: (a) net cone tip resistance: \( q_{\text{net}} = q_t - \sigma_{v0} \), (b) measured excess porewater pressure: \( \Delta u_2 = u_2 - u_0 \); and (c) effective cone resistance: \( q_E = q_t - u_2 \). Details on the SCE-CSSM solutions are given elsewhere (Mayne 1991; 2017; Agaiby & Mayne 2018).

For "normal" and "well-behaved" clays, which include inorganic fine-grained soils such as clays and silts of low sensitivity, a set of simple relationships can be derived. By adopting characteristic default values for effective friction angle \( \phi' = 30\degree \) and rigidity index \( I_R = 100 \), linear expressions for the effective preconsolidation stress are obtained:

\[
F_1 = 100 \cdot \frac{f_{\text{net}}(q_t - \sigma_{v0})}{(a)} \quad \%
\]

\[
B_3 = \frac{\Delta u_2}{(q_t - \sigma_{v0})} \quad \%
\]

where \( F_1 \) is the normalized friction and \( B_3 \) is the normalized porewater.
Application to soft Chicago clay

An example of a common geomaterial in this category includes the infamous soft Chicago clays that were deposited in a glacio-lacustrine environment (Cho & Finno 2010). The soft clay has a mean water content of 20%, liquid limit of 38% and plasticity index PI= 12%. Results of CPTu tests conducted by GT at the national geotechnical experimentation site (NGES) at Northwestern University are shown in Figure 4 (Ouyang & Mayne 2017). As seen in the profile, a thin soft clay resides between depths of 9 to 10.5 m with a thicker soft clay layer in the range of 12 to 18 m.

![Figure 4. Results of CPTu sounding at the NGES at Northwestern University, Illinois](image-url)
Post-processing of the CPTu data using equations [1], [2], and [3] show good agreement in evaluating the profile of effective yield stress in this clay layer compared with results from one-dimensional consolidation tests made on undisturbed samples taken at the site.

![Effective Yield Stress, $\sigma_p'$ (kPa)](image)

Figure 5. Evaluation of effective yield stresses at NWU using consolidation tests and CPTu

**Application to soft Bay Mud, California**

Another example of a "well-behaved" fine-grained soil is the San Francisco Bay Mud which is a soft clay deposit in northern California. Results of a representative CPTu are shown in Figure 6 and indicate the soil profile at a site in eastern San Francisco (Hunt et al. 2002). For this site, index values include: $70 < w_r < 90\%$, $60 < LL < 80\%$, and $35 < PI < 45\%$. Post-processing the CPTu data using [1], [2], and [3] show good agreement with each other, as well as with the results of consolidation tests, as seen in Figure 7. The consolidation tests were performed using a CRS device as reported by Pestana et al. (2002).
Figure 6. Representative CPTu in San Francisco Bay Mud (data from Hunt et al. 2002)

Figure 7. Evaluation of effective yield stresses in Bay Mud from consolidation tests and CPTu.
CPTu SCREENING OF SOFT ORGANIC CLAYS

For soft organic clays, the aforementioned equations will not apply, thus can serve as a means to screen such geomaterials from the soil profile. Two examples of CPTu soundings in soft organic clays are presented to illustrate the approach. Additional case records and documentation are given in Agaiby (2018).

Soft organic alluvial soils in Washington, DC

Results of in-situ tests including CPTu soundings in soft organic clayey silts along the Potomac River at the Anacostia Naval Air Station are presented by Mayne (1987). Here the soft soils are categorized as organic clayey silts (OH) per the Unified Soils Classification Systems (USCS) with mean values (and plus and minus one standard deviations) of $w_{n} = 68 \pm 16 \%$, $LL = 83 \pm 25 \%$, and $PI = 37 \pm 17 \%$. A representative piezocone sounding is depicted in Figure 8. For the post-processing of CPTu data, the use of the $Q$-$F$ and $Q$-$Bq$ graphs do not find any points within the zone 2 category for organic soils. When the data are evaluated to determine the profile of effective yield stress, the three expressions do not agree, as shown by Figure 9.

Figure 8. Representative CPTu in soft organic clay along the Potomac River, DC
Figure 9. Post-processing of CPTu data from Anacostia-Bolling: (a) SBTn zones using CPT material index, \(I_c\); (b) CPTu screening equations for yield stress

Note that the hierarchy of the results from equations [1], [2], and [3] show that the CPTu evaluations for preconsolidation stress in soft organic clays indicates:

\[ \sigma_p' = 0.53 \Delta u_2 < 0.33 q_{net} < \sigma_p' = 0.60 q_E \]

**Soft organic peaty clay in Saint Paul, MN**

Results of CPTu tests were collected during a training exercise underneath I-35E just northeast of Saint Paul, MN in 2007. All three MnDOT CPT rigs available at the time were used to obtain in-situ test data at the site. The site is well known as having soft organic soils and samples were obtained from three borings made at the site (Boring ID # T523, T542, and T518). Boring logs indicate the presence of highly organic silt loams, with marl and spongy organic clayey silts. From 12 samples, the natural water contents ranged from 30% to 191%, with a mean of 112 ± 58%.

A representative piezocone sounding from the site (MnDOT CPTu ID #F22Y0703C) is used for this illustration and is presented as Figure 10. Post-processing these data using the SBTn algorithms and CPT material index show that the soils classify primarily as Zone 3 (clays) and Zone 4 (silty mix), thus do not identify the geomaterials properly as Zone 2 (organic soils).
Figure 10. MnDOT CPTu sounding at the I-35E test site near St. Paul, Minnesota

Figure 11. Post-processing St. Paul sounding using the 9-zone SBTn classification system
Using the recommended approach, Figure 12 shows the CPT-evaluated yield stress profiles from equations [1], [2], and [3]. Here again, the three estimates of $\sigma_p'$ do not agree but show the same hierarchy as detailed in equation [4].

![Figure 12. CPTu screening of soft organic soils at St. Paul test site](image)

**CPTu EVALUATIONS OF YIELD STRESS IN SOFT ORGANIC SOILS**

Once the presence of soft organic clays and silts is identified using the aforementioned CPT screening algorithms, the evaluation of effective yield stress is done using the procedure outlined in Chapter 2 of this MnDOT manual. For soft organic soils, an exponent of $m' = 0.90$ is recommended (Mayne, et al. 2009). Therefore, the preconsolidation stress is obtained from:

$$[5] \quad \sigma_p' = 0.33 \cdot (q_{\text{net}})^{0.90} \quad \text{(units of kPa)}$$

The algorithm can be expressed in dimensionless form by:

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

where $\sigma_{\text{atm}} = $ reference stress ($= 100 \text{ kPa} = 14.7 \text{ psi}$) and $m' = 0.90$ for soft organic silts and clays.

The approach is applied to the two former example case studies in Figure 13 (Washington DC) and Figure 14 (St. Paul, MN), respectively, showing good agreement with benchmark results taken from laboratory consolidation tests on undisturbed samples of these sites in both cases.
Figure 13. Profiles of yield stress from consolidation tests and CPTu method for organic clayey silts at Anacostia-Bolling site in Washington, DC.

Figure 14. Profiles of yield stress from consolidation tests and CPTu method for organic clayey silts at MnDOT test site near Saint Paul, Minnesota.
REFERENCES


Direct CPT Method for 130 Footings on Sands, including Rectangular Foundations

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ABSTRACT

Using the concept of a characteristic load-displacement curve for shallow foundations, a direct design method based on cone penetration testing (CPT) is applied to a database compiled from a total 130 footings on sands. Performance results are based solely on field load test data from large foundations (B > 0.5 m) in order to avoid known scaling effects from small model size footings. The footing responses are presented in terms of applied stress (pF) versus square root of normalized displacements (s/B) so that a single parameter represents the behavior of all size foundations on the particular sand. Furthermore, footing stresses are normalized by the cone tip resistance (qc) to obtain a unique relationship for shallow foundations on sands. While the method was originally developed from field load tests on square and circular footings, it is now extended to include rectangular foundations via elasticity solutions using a dimensionless length-to-width ratio. Limited data from 8 footings on calcareous sands also show compatibility with the derived algorithm.

INTRODUCTION

Many methods are available for assessing the bearing capacity and settlement behavior of shallow foundations situated on sands, including theoretical, analytical, numerical, statistical, and empirical approaches (e.g., Lutenegger & DeGroot 1995; Briaud & Gibbens 1999). Theoretical procedures are often taught in university curricula and detailed in geotechnical textbooks, mainly promoting limit plasticity for bearing capacity and elastic continuum solutions for footing settlement calculations. For implementation of in-situ tests in geotechnical practice, a two-step process is then required as the field data are first used to assess input parameters of the sand (i.e., unit weight γt, effective friction angle φ', dilatancy angle ψ', Poisson's ratio ν', and elastic modulus E') that are then utilized in the elasticity and plasticity solutions.

Yet in recent times, interest has grown towards the adoption of direct in-situ methods (Briaud 2007, 2013), including techniques that employ the results of standard penetration tests (SPT), dilatometer (DMT), cone penetration (CPT), pressuremeter...
Figure 1. Individual measured load versus displacement responses for five large footings on natural sand at TAMU (data from Briaud & Gibbens 1994, 1999)

(PMT), and/or shear wave velocity ($V_s$). Herein, a direct CPT design method, derived on the basis of statistical analyses of full-scale load tests on square and circular footings, is extended to rectangular foundations.

CHARACTERISTIC LOAD-DISPLACEMENT CURVES

For footings situated on a given sand formation, individual records of vertical load ($Q$) versus displacement ($s$) show separate curves that increase with foundation size, as illustrated in Figure 1. This shows load test results from five large footings at the national test site for sands located at Texas A&M University (TAMU), as detailed by Briaud & Gibbens (1994). Details on the characteristics of the sand layers at this site are given by Briaud (2000), including information from laboratory, in-situ testing, and geophysical measurements.

It has been observed that the response of footing load tests can be unified via a "characteristic load-displacement curve" (Briaud & Gibbens 1999), where the data are presented in terms of applied stress ($p = Q/B^2$) versus normalized displacement ($s/B$), where $B =$ foundation width of a square footing. Figure 2 presents the unified trend for the TAMU test site to illustrate the normalized behavior for all five footings in a single relationship.

One method to establish a reference "capacity" of foundation response is selecting the magnitude of stress ($p_{\text{capacity}}$) corresponding to a settlement of 10% of the
Figure 2. Characteristic load-displacement curve for all five TAMU footings

foundation width, or \( (s/B) = 0.10 \). At the TAMU site, a "capacity" of 1.55 MPa can be chosen in this fashion.

An additional simplification is afforded by plotting the data in terms of square root of \( (s/B) \), since the derived slope is a single parameter (designated \( r_s \)) that is characteristic of that sand formation (Mayne & Woeller 2014):

\[
r_s = \frac{p_F}{\sqrt{(s/B)}}
\]

Figure 3 presents the TAMU footing responses in this manner and determines \( r_s = 4.86 \) MPa as the characteristic value for the deltaic Pleistocene sands at the site. This single value provides a means for representing the load-displacement response for all size footings on this sand.

**DIRECT CPT METHOD FOR NORMALIZED FOOTING RESPONSE**

In order to extend the value of characteristic curves to other sand deposits and geologic formations, Briaud (2007) presents the idea of normalized load-displacement relationships, where the aforementioned applied foundation stress axis is normalized to a representative strength parameter for the soil.

In a recent study of 32 large scale footings on 13 sands, Mayne et al. (2012) show the application of this approach using the cone tip resistance (\( q_c \)) from cone penetration tests (CPT). Foundations were primarily square or circular with equivalent
Figure 3. Stress versus square-root plotting of pseudo-strain (s/B) for the five TAMU footings on sands (data from Briaud & Gibbens 1999)

square widths (B) that ranged in size from 0.5 to 6 m. For sands of primarily quartz to silica mineralogy, the derived trend for square footings is shown in Figure 4, which is expressed:

\[ p_F = 0.58 \cdot q_c \cdot \sqrt{\frac{s}{B}} \]  

(2)

where the value of \( r_s = 0.58 \cdot q_c \) has been substituted so that all footings have been normalized to a common sand strength parameter (Mayne & Illingworth 2010).

Using the "capacity" criterion of \( (s/B) = 0.1 \) in equation (2) indicates a general mean reference stress equal to 0.18\( \cdot q_c \) for all 13 sands which is very similar to the value of 0.23\( \cdot q_c \) noted for the TAMU sand site by Briaud & Gibbens (1999).

For CPT interpretations, the use of net cone resistance \( (q_{net} = q_t - \sigma_{vo}) \) from total cone resistance \( q_t \) is preferred over measured \( q_c \) (Mayne 2007). However, for clean sands, the correction of \( q_c \) to \( q_t \) is negligible and since overburden stresses are small, especially for shallow foundations, then: \( q_c \approx q_t \approx q_{net} \).

SAND MINERALOGY

The aforementioned relationship was developed primarily for sands of common mineralogy, mainly quartz to silica particles. Geologic origins of these sand formations included: alluvial, glaciofluvial, marine, aeolian (dune), and residuum, as well as three tests on compacted sand fill (Mayne et al. 2012; Uzielli & Mayne 2012).
In contrast, footing load test data from a calcareous sand formation at Ledge Point, a site about 100 km north of Perth, Australia are reported by Lehane (2011). Here, a total of 8 circular and square footings and plates were used, with sizes in the range of $0.27 \text{ m} < B < 0.6 \text{ m}$. The characteristic stress versus normalized displacement curves give a representative $r_s = 3.14 \text{ MPa}$ for this carbonate sand and the normalized stress uses an overall average $q_c = 5.09 \text{ MPa}$ from 8 CPT soundings. The ratio $r_s/q_c = 0.61$ is quite close to the empirical value of 0.58 in equation (2).

Footing load test data for the calcareous sand series is overlain on the results from the 32 prior reported foundations in Figure 5. Thus, it would seem that the CPT $q_c$ captures the essence of the strength-stiffness behavior of the sands, regardless of sand mineralogy, at least for the 14 different sands discussed herein.

**FOUNDATION SHAPE**

For the detailed load tests in the original 32 foundation database, 80% consisted of square foundations and the remaining 20% were circular. For the latter, an equivalent $B$ was obtained by setting $B^2 = \pi d^2/4$. For rectangular footings, a number of different approaches are available, as discussed by Briaud (2007). Herein, the elastic solution from Giroud (1968) was utilized to cover rectangular distortions $(A/B)$ ranging from 1 (square) to very long foundations with $A/B = 20$, where $A =$ foundation length and $B =$ width. The influence factors for mean displacements of a range of rectangular shaped
Figure 5. Normalized footing stress-displacement responses for 8 footings on calcareous sand at Ledge Point, Australia (data from Lehane et al. 2011)

foundations are given in tabular form in Poulos & Davis (1974), as well as presented graphically in Figure 6. The effect can be approximated by the expression:

\[
I_{A/B} = (A/B)^{0.345} \tag{3}
\]

In linear elastic theory, the solution for shallow foundation displacements is given in the form:

\[
s = \frac{p_F \cdot B \cdot I_{A/B} \cdot (1 - \nu^2)}{E_s} \tag{4}
\]

where \(E_s\) = equivalent Young's modulus for soil and \(\nu\) = Poisson's ratio. For sands, a representative value \(\nu = 0.2\) is valid at working loads (e.g., Lehane & Cosgrove 2000). Rearranging to express the footing response in terms of applied stress:

\[
p_F = \frac{E_s}{(1 - \nu^2) \cdot (s/B) \cdot \frac{1}{I_{A/B}}} \tag{5}
\]

Since soil modulus \(E_s\) is actually nonlinear with pseudo-strain \((s/B)\), the resulting algorithm in equation (2) expresses \((s/B)\) in a power law format to accommodate the fact that the soil modulus decreases with increasing strain. In any event, it can be seen that applied footing stress is affected in terms of the reciprocal of the displacement influence factor and this facet will be explored in a later section of this paper.
The effect of embedment has been shown to be inherently included by adoption of the normalized load-displacement curves (Briaud, 2007). Thus no correction or factor is necessary for the depth of foundation placement below the ground surface. Furthermore, the effect of embedment has been shown to be small for shallow foundations from both a numerical viewpoint (Burland, 1970; Christian & Carrier, 1978; Mayne & Poulos, 1999), as well as from full scale load tests on footings (Fellenius & Altaee, 1994).

**ADDITIONAL FOUNDATION PERFORMANCE DATABASES**

In addition to the 32 full scale load tests already described, results from an additional 90 shallow foundations have been reported in prior research studies. These include large spread footings with their performances at service loads, i.e., measured settlements and applied bearing stresses. In these cases, only a single datum is available for each case study in the paired set \((s, p_F)\). The measured CPT \(q_c\) resistances are also available for each of these case studies, generally reported as the average value over the range from the foundation bearing elevation to a depth of about \(2B\), as suggested by Schmertmann (1970). Table 1 presents a summary of the available datasets that have been compiled for this complete study.
Table 1. Sources of Shallow Foundation Performance and CPT Data

<table>
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<th>Source of Data</th>
<th>No. of Footings</th>
<th>Mean B (m)</th>
<th>Max. B (m)</th>
<th>Min. B (m)</th>
<th>Mean A (m)</th>
<th>Max. A (m)</th>
<th>Min. A (m)</th>
</tr>
</thead>
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<td>32</td>
<td>1.49</td>
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<td>6.09</td>
<td>0.46</td>
</tr>
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<td>Lehane (2011)**</td>
<td>8</td>
<td>0.41</td>
<td>0.60</td>
<td>0.27</td>
<td>0.41</td>
<td>0.60</td>
<td>0.27</td>
</tr>
<tr>
<td>Gifford et al. (1987)*</td>
<td>17</td>
<td>8.46</td>
<td>15.88</td>
<td>5.27</td>
<td>15.91</td>
<td>35.96</td>
<td>7.01</td>
</tr>
<tr>
<td>Schmertmann (1970)*</td>
<td>31</td>
<td>9.45</td>
<td>56.08</td>
<td>0.61</td>
<td>12.88</td>
<td>86.68</td>
<td>0.61</td>
</tr>
<tr>
<td>Papadopoulos (1992)*</td>
<td>29</td>
<td>8.70</td>
<td>36.00</td>
<td>1.00</td>
<td>13.00</td>
<td>72.90</td>
<td>1.00</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>130</td>
<td>6.71</td>
<td>56.08</td>
<td>0.27</td>
<td>10.12</td>
<td>86.68</td>
<td>0.27</td>
</tr>
</tbody>
</table>

Notes: *Redundant case studies removed;  **Calcareous sand deposit.

For the total set of footing data on noncalcareous sands (n = 122), the range of length to width ratios varied from 1 < (A/B) < 23, with an average value (A/B) = 2.38 ratio and corresponding standard deviation (SD) = 2.85. The embedment depth (z_e) to footing width ratio ranged from 0 < ze/B < 2.22, and averaged 0.46 with a SD = 0.49.

Measurements of applied stresses and corresponding settlements were obtained over a large range of values during the execution of the full scale load tests. For the case studies at working loads, only a single value of stress and displacement were available. For the complete dataset (n = 452), the mean applied foundation stress = 359 kPa (± 329 kPa) with a maximum value of 1708 kPa. The mean measured settlement was 25 mm (± 32 mm) with an attained maximum value 269 mm.

For the CPT results from all 130 footing case studies, the full range of cone tip resistances varied from 0.88 MPa < q_c < 21.56 MPa, indicating a wide range of consistencies and relative densities of these sands from very loose to dense. The overall CPT data indicate a mean value q_cAVE = 9.45 MPa with SD = 5.39 MPa.

EXTENDED DIRECT CPT APPROACH FOR RECTANGULAR FOOTINGS

Combining the direct CPT equation developed from 32 full scale load tests (Equation 2) together with the elastic influence factor solution (Equation 3), the extended expression for foundation load-displacement-capacity of rectangular footings becomes:

\[ p_F = 0.58 \cdot q_c \cdot \sqrt{\left(\frac{s}{B}\right)} \cdot (A/B)^{-0.345} \]  

which is presented in Figure 7. The performance data from the compiled 90 foundation case studies are superimposed on the graph, showing general agreement with the expression. As these case studies provide data only at service loads, they are clustered towards the origin of the graph, thus masking their trending. Figure 8 shows the same data but plotted in log-log form of normalized stress versus normalized displacement in order to enhance the results at the lower end of the scale.
Figure 7. Normalized response of 122 footings on sands using arithmetic scales

Figure 8. Normalized response of 122 footings on sands using logarithmic scales in order to show small displacement region
CONCLUSIONS

A direct CPT method for square footings on quartz and silica sands is extended to rectangular foundations by incorporating an influence factor term which considers the foundation length-to-width ratio (A/B) from elastic theory solutions. The method utilizes a normalized stress versus normalized displacement concept that accounts for foundation size and embedment effects. The results are validated with full-scale in-service performance data from 32 footings on 13 sands that are primarily quartz and silica types, as well as performance settlement data from an additional 90 case studies that were summarized in prior research efforts. Finally, the derived approach also appears applicable to uncemented carbonate sands, based on load test data from 8 footings and plates situated at an Australian calcareous sand site.
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REFERENCES


