GeoCharacterization for Shallow and Deep Foundations Using Hybrid Geotechnical-Geophysical In-Situ Tests

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Purposes: Geotechnical Site Investigation

- Required for all geotechnical projects
- Determine geostratigraphy for site development
- Information for foundation design
- Data for geotechnical parameter evaluation
- Input to analytical models and numerical FEM
- Minimize problems during construction
- Mitigate potential for legal involvement
GeoCharacterization

Initial Conditions

**INDICES**
- Geologic Origin
- Age, $A_g$
- Grain Sizes, $D_{50}$
- Mineralogy
- Plasticity, PI
- Shape (fractals)
- Sphericity, $S_{ph}$
- Roundness, $R_n$
- Angularity, $A_{ng}$
- Packing limits: $e_{max}$ and $e_{min}$
- Specific Surface, $S_s$
- Particle characteristics for DEM:
  - crushing strength
  - modulus
  - roughness
  - friction

**STATE**
- Void Ratio, $e_0$
- Unit Weight, $\gamma_t$
- Relative Density, $D_R$
- State Parameter, $\Psi$
- Vertical Stress, $\sigma_{vo}$
- Hydrostatic Pressure, $u_o$
- Yield Stress Ratio, YSR
- Saturation, S (%)
- Geostatic $K_0 = \sigma_{ho} / \sigma_{vo}$
- Stiffness, $G_0 = G_{max}$
- Degree of cementation
- Fabric and void index, $I_{vo}$
- Continuity
  - intact
  - fissured

**Geoengineering Parameters**

**CONDUCTIVITY**
- Hydraulic: $k_r, k_h$
- Thermal: $k_C$
- Electrical: $\Omega, \zeta$
- Chemical: $D_t$
- Transmissivity, $T_m$
- Permittivity, $P_m$

**COMPRESSIBILITY**
- Recompression index, $C_r$
- Yield Stress, $\sigma_y$ (and YSR)
- Preconsolidation, $\sigma_0'$ (and OCR)
- Coefficient of Consolidation, $c_y$
- Virgin Compression index, $C_v$
- Swelling index, $C_s$

**STIFFNESS**
- Stiffness: $G_0 = G_{max}$
- Shear Modulus, $G'$ and $G_u$
- Elastic Modulus, $E'$ and $E_u$
- Bulk Modulus, $K'$
- Constrained Modulus, $D'$
- Tensile Stiffness, $K_t$
- Poisson's Ratio, $\nu$
- Effects of Anisotropy ($G_{xy}/G_{xx}$)
- Nonlinearity ($G/G_{max} vs \gamma_s$)
- Subgrade Modulus, $k_s$
- Spring Constants, $k_z, k_x, k_{xy}, k_0$

**STRENGTH**
- Drained and Undrained, $\tau_{max}$
- Peak ($s_u, c', \phi'$)
- Post-peak, $\tau'$
- Remolded strength
- Softened or critical state, $s_u$ (rem)
- Residual ($c', \phi'$)
- Cyclic Behavior ($\tau_{cyc}/\sigma_{vo}$)
Quote from Lord Kelvin (1883):
“...when you can measure what you are speaking about and express it in numbers, you know something about it; but when you cannot express it in numbers, your knowledge is of a meager and unsatisfactory kind"
GeoCharacterization in 2014

- This talk is part **State-of-the-Art (SOA)** = What we COULD be doing
- This talk also part **State-of-the-Practice (SOP)** = What we ARE doing
- Limited time, so focus on **geocharacterization**
- Intro GeoCourse needs more balance of geophysics + in-situ + lab testing
- This talk = part **SOA** + part **SOP** → betterment

Mayne ≠ SOB
Evolution of Geotechnical Site Characterization
Modified after Lacasse (1985)

Time period covered by your favorite geotextbook

Therefore Missing Topics of Great Importance to Modern Geotechnical Education

Level of Reliance for Design (%)

In-Situ Tests

Probabilistic Models

Analytical Models

Numerical Simulation

Geophysics

Judgment

Year
Geotechnical Laboratory Testing Devices

- Grain size analyses
- Hydrometer
- Water content by oven
- Liquid limit cup
- Electron microscopy
- Plastic limit thread
- Fall cone device
- Pocket penetrometer
- Torvane
- Unconfined compression
- Miniature vane
- Digital image analysis

- Mechanical oedometer
- Consolidometer
- Constant rate of shear (CRS)
- Falling-head permeameter
- Constant-head permeameter
- Flow permeameter
- Direct shear box
- X-ray diffraction
- Ring shear
- Unconsolidated undrained Tx
- Simple shear
- Directional shear cell

- Triaxial apparatus (iso-consols, CIUC, CKoUC, CAUC, CIUE, CAUE, CKoUE, stress path, CIDC, CKoDC, CIDC, CKoDE, constant P’)
- Plane strain apparatus (PSC, PSE)
- True triaxial (cuboidal)
- Hollow cylinder
- Torsional Shear
- Resonant Column Test device
- Non-resonant column
- Bender elements
Methods for Geomaterial Characterization

Soils Laboratory

Lab Rat
Laboratory Soils Testing

- Limited number (discrete points)
- Lengthy test durations
- Affected by sample disturbance
- Expensive: Cost per specimen:
  - Oedometer = $600 (2 weeks)
  - Automated Consolidation = $800 (2-3 days)
  - CIUC Triaxial = $600 (2 to 3 days)
  - CK\textsubscript{0}UC Triaxial = $1500 ea (5 days)
  - Resonant Column= $2000 ea (1 week)
  - Permeability = $800 (1 to 2 weeks)
Field In-Situ Geotechnical Test Methods

SPT = Standard Penetration Test
TxPT = Texas Penetration Test
VST = Vane Shear Test
PMT = Pressuremeter Test
CPMT = Cone Pressuremeter
DMT = Dilatometer Test
SPLT = Screw Plate Load Test
K₀SB = Iowa K₀ Stepped Blade
SWS = Swedish Weight Sounding
HF = Hydraulic Fracture
BST = Borehole Shear Test

TSC = Total Stress Cell (spade cell)
FTS = Freestand Torsional Shear
PV = Piezovane
MPT = Macintosh Probe Test
CPT = Cone Penetration Test
CPTu = Piezocone Penetration
RCPTu = Resistivity Piezocone
SCPTu = Seismic Cone
SDMT = Seismic Flat Dilatometer
TBPT = T-Bar Penetrometer Test
BPT = Ball Penetrometer

Full Flow Penetrometers

PPT = Plate Penetration Test
PLT = Plate load test
HPT = Helical Probe Test
PBPT = piezoball penetration test
RapSochs = Rapid soil characterization system
CPTù = piezodissipation test
DMTà = Dilatometer with A-reading dissipations
SPTT = Standard Penetration Test with Torque
LPT = Large Penetration Test
DEPPT = Dual Element PiezoProbe Test
SCPMTu = Seismic Piezocone Pressuremeter
Geotechnical Methods for Site Investigation

Soils Laboratory
Lab Rat

In-Situ Testing
Field Mouse

Ball penetrometer testing
Home Appliances

- Telegraph
- Stove
- Sewing Machine
- Horse & Buggy

Office Equipment

- Casagrande Cup
- Split-Spoon
- Abacus

Note: Not to scale
1950

Home Appliances

- Telephone
- Refrigerator
- Gas Automobile
- Television
- Washer

Office Equipment

- Casagrande Cup
- Split-Spoon
- Slide Rule

Note: Not to scale
Home Appliances | 2014 | Office Equipment

- Smartphone
- Refrigerator
- 3-d LED Television
- Washer
- Electric Automobile

- Liquid Limit
- Tablet
- Split-Spoon

Note: Not to scale
## Calibration of SPT Energy - Auto Hammers

<table>
<thead>
<tr>
<th>Manufacturer Type</th>
<th>ID No.</th>
<th>Mean Energy Ratio (%)</th>
<th>Reference</th>
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<td>Diedrich D-50</td>
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<td>BK-81 w/ AW-J rods</td>
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<td>68.6</td>
<td>ASCE</td>
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<td>Mobile B-80</td>
<td>ID 18</td>
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<td>UDOT</td>
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<td>72.9</td>
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<td>76</td>
<td>UF</td>
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<td>CME 85</td>
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<tr>
<td>CME 75 rig</td>
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<td>UDOT</td>
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</table>

**Factor of 2.1**
$c_u$ = undrained strength  
$\gamma_T$ = unit weight  
$I_R$ = rigidity index  
$\phi'$ = friction angle  
$OCR$ = overconsolidation  
$K_0$ = lateral stress state  
$e_o$ = void ratio  
$V_s$ = shear wave  
$E'$ = Young's modulus  
$C_c$ = compression index  
$q_b$ = pile end bearing  
$f_s$ = pile skin friction  
$k$ = permeability  
$q_a$ = bearing stress

$D_R$ = relative density  
$\gamma_T$ = unit weight  
$LI$ = liquefaction index  
$\phi'$ = friction angle  
$c'$ = cohesion intercept  
$e_o$ = void ratio  
$q_a$ = bearing capacity  
$\sigma_p'$ = preconsolidation  
$V_s$ = shear wave  
$E'$ = Young's modulus  
$\Psi$ = dilatancy angle  
$q_b$ = pile end bearing  
$f_s$ = pile skin friction
Need a Variety of Different Methods to Truly and FullyAscertain Soil Parameters
Cone Penetration Test (CPT)

- ASTM D-5778 Field Test Procedures
- Continuous push at 20 mm/s
- Add rods at 1-m vertical intervals

Readings taken every 1 or 5 cm

- \( q_t \) = measured tip resistance
- \( f_s \) = sleeve friction resistance
- \( u_m \) = porewater pressure
- \( q_c \) = total cone tip resistance
- \( i_c \) = inclination

Electronic Penetrometer
CPT

- Current Phase Transformer
- Cross Product Team
- Cellular Paging Teleservice
- Chest Percussion Therapy
- Crisis Planning Team
- Consumer Protection Trends
- Computer Placement Test
- Current Procedural Terminology
- Cost Per Treatment
- Choroid Plexus Tumor
- Cardiopulmonary Physical Therapy
- Corrugated Plastic Tubing
- Cumulative Price Threshold
- Cell Preparation Tube
- Central Payment Tool
- Certified Proctology Technologist
- Cockpit Procedures Trainer
- Cone Penetration Test
- Color Picture Tube
- Critical Pitting Temperature
- Certified Phelbotomy Technician
- Control Power Transformer
- Cost Production Team
- Channel Product Table
- Conditional Probability Table
- Command Post Terminal
Cone Penetrometer Testing

- Electronic Steel Probes with 60° Apex Tip
- ASTM D 5778 Procedures
- Hydraulic Push at 20 mm/s
- No Boring, No Samples, No Cuttings, No Spoil
- Continuous readings of stress, friction, pressure
Cone Penetration Vehicles

InDOT
Cone Penetration Vehicles
Geostratigraphy by CPTu in Portsmouth, Virginia

Cone Resistance, $q_t$ (kPa)

Sleeve Friction, $f_s$ (kPa)

Porewater Pressure, $u_2$ (kPa)
Georgia Tech Anchored Cone Rig

- 6-tonne CPT truck with 20-tonne hydraulic pushing system
- No special license
- Twin earth anchors
- Has been used at sites in GA, VA, NC, SC, IL, FL, AR, MO, TN, KY, and AL

Downtown Atlanta, Georgia
Quick Estimate of Unit Weight by CPTu

\[
\text{Gamma (kN/m}^3\text{)} = 12 + 1.5 \ln(f_s + 1)
\]

Note: \( f_s \) (kPa)

- Sand
- Clay
- Diatomaceous Mudstone
- Organic Peat

\( n = 950 \)
Friction Angle of Undisturbed Sands

\[ \phi' = 17.6^\circ + 11.0 \cdot \log (q_{t1}) \]

\[ \phi' = 25.0^\circ (q_{t1})^{0.10} \]

Effective Friction Angle, \( \phi' \) (deg)

Normalized Cone Resistance, \( q_{t1} = \frac{q_t}{\sigma_{atm}} \left( \frac{\sigma_{vo}'}{\sigma_{atm}} \right)^{0.5} \)

Uzielli et al (2013)
Kulhawy & Mayne (1990)
Triaxial Dataset
McDonald Farm
Stockholm Sand
Blessington
Friction Angle $\phi'$ from CPTU for clays & silts (OCR < 2)

Norwegian Institute of Technology: Senneset et al (1989); Sandven (CPT'95)

$\phi' = 29.5 \, B_q^{0.121} \left[ 0.256 + 0.336 \, B_q + \log Q \right]$ where $B_q = (u_2 - u_0)/(q_t - \sigma_{vo})$

Notes for NTNU Method:

1. Define Cone Resistance Number: $N_m = (q_t - \sigma_{vo})/(\sigma_{vo}' + a')$

2. Attraction: $a' = c' \cot \phi'$ where $\phi'$ = effective friction angle and $c'$ = effective cohesion intercept.

3. For case where $a' = c' = 0$: $N_M = Q = (q_t - \sigma_{vo})/\sigma_{vo}'$

4. Define Porewater Pressure Parameter: $B_q = \Delta u_2/(q_t - \sigma_{vo})$

5. Approximate Expression Given for Ranges: $0.1 < B_q < 1.0$ and $20^\circ < \phi' < 45^\circ$

6. Plastification angle, $\beta_p = 0$
Boston Blue Clay
LL = 45; PI = 20
\(\omega_n = 47\%\)

\(\phi' = 36.7^\circ\)
\(c' = 0\)

\[ y = 0.5979x \]
\(R^2 = 0.9142\)
\(n = 16\)

Shear Stress, \(q = (\sigma_1 - \sigma_3)/2\) (kPa)

Effective Stress, \(p' = (\sigma_1' + \sigma_3')/2\) (kPa)
Newbury, Massachusetts
(Landon, 2007, U-Mass-Amherst)
Yield stress in soils from CPT

General Trend:
\[ \sigma_y' = 0.33(q_t - \sigma_{vo})^{m'} \]

Fissured Clays: \[ m' = 1.1 \]
Intact clays: \[ m' = 1.0 \]
Sensitive clays: \[ m' = 0.9 \]
Silt Mixtures: \[ m' = 0.85 \]
Silty Sands: \[ m' = 0.80 \]
Clean Sands: \[ m' = 0.72 \]

Units: kPa

Net Cone Resistance, \( q_t - \sigma_{vo} \) (kPa)

Yield Stress, \( \sigma_y' \) (kPa)
Blessington Sand Site
University of College Dublin (UCD)

Doherty et al. (2012)
Tolooiyan & Gavin (2011)
Blessington Sands, Ireland (Doherty et al 2012)

Depth, \( z \) (m)

Cone Resistance, \( q_t \) (MPa)

Friction Angle, \( \phi' \) (deg)

Dense OC Quartz Sands
\( D_{50} \approx 0.12 \text{mm} \)

CPT-Bk
CPT-Be
CPT-Gn
CPT-Rd

Triaxials
CPTs: $\sigma_p' = 0.33 \ q_{\text{net}}^{0.75}$

and $\text{OCR} = \frac{\sigma_p'}{\sigma_{\text{vo}}'}$
Field Geophysics - Mechanical Wave Methods

- SRFS = Surface Refraction Survey
- SFLS = Surface Reflection Survey
- SASW = Spectral Analysis of Surface Waves
- MASW = Modal Analysis (Rayleigh Waves)
- CSW = Continuous Surface Waves
- PSW = Passive Surface Wave Testing
- ReMi = Reflection MicroSeis
- SLP = Suspension Logger ProblNg
- CHT = Crosshole Test
- RCHT = Rotary Crosshole
- DHT = Downhole Test
- UHT = Uphole Test
- SCPTu = Seismic Piezocone Test
- SDMT = Seismic Flat Dilatometer Test
- BTSD = Borehole Torsional Shear Device

**Seismograph + Source**
- Vertical Source
- Torsional Source

**Oscilloscope + Source**
- Cased Boreholes

**V_p**, **V_s HH**, **V_p**, **V_s HV**, **V_s VH**, **V_s HH**, **V_s VH**, **V_s VV**

**Rayleigh Wave Methods**
- SASW
- MASW
- CSW
- PSW
- ReMi

**High frequencies**
- medium frequency content
- low frequency content

**V_sRW**

**SLP**

**BTSD**

**Spectral Analyzer + Source**
Shear Wave Velocity, $V_s$

- Fundamental measurement in all solids (steel, concrete, wood, soils, rocks)

- Initial stiffness represented by the small-strain shear modulus ($G_{dyn} = G_{max} = G_0$):

  $$G_0 = \rho_t \, V_s^2$$

  where total mass density $\rho_t = \gamma_t / g_a$

- Applies to all static & dynamic problems at small strains ($\gamma_s < 10^{-6}$)

- Applicable to both undrained & drained loading cases in geotechnical engineering
Geotechnical Methods for Site Investigation

Soils Laboratory  In-Situ Testing  Geophysics

Lab Rat  Field Mouse  Fruit Bat

Field mouse with ball penetrometer
Hybrid Geotechnical - Geophysical Test

Seismic Cone Penetration Test (SCPT)

ASTM D 5778 and ASTM STP 1213

- Cone Truck
- Surface Seismic Source (parallel with geophone axis)
- Horizontally-polarized and vertically-propagating shear waves
- Electronic Penetrometer
  - horizontal geophone
  - inclinometer
  - $f_s = \text{sleeve friction resistance}$
  - $u_2 = \text{porewater pressure}$
  - $q_t = \text{total cone tip resistance}$
- Penetrometer Readings taken every 1 or 2 seconds
- Shear Wave Arrivals taken at 1-m rod intervals
Hybrid Geotechnical - Geophysical Test
SCPTu Sounding – Memphis, TN

- $q_t$ (MPa)
- $f_s$ (kPa)
- $u_2$ (kPa)
- $V_s$ (m/sec)

$d = 35.7$ mm
# Costs of Methods to profile $V_s$ to 30 m

<table>
<thead>
<tr>
<th>METHOD</th>
<th>TYPICAL COST</th>
</tr>
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<tr>
<td>Suspension Logging (PSSL)</td>
<td>$35,000 #</td>
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<tr>
<td>Crosshole Testing (CHT)</td>
<td>$20,000</td>
</tr>
<tr>
<td>Downhole Testing (DHT)</td>
<td>$ 9,000</td>
</tr>
<tr>
<td>Surface Waves (SASW, MASW)</td>
<td>$ 4,500</td>
</tr>
<tr>
<td>ReMi  Passive Surface Waves</td>
<td>$ 2,500</td>
</tr>
<tr>
<td>Seismic Piezocone*</td>
<td>$ 2,000</td>
</tr>
</tbody>
</table>

**NOTE:**

# Typically only economical for profiling $z > 60$ m

* Includes 4 separate readings with depth: $q_t$, $f_s$, $u_2$, and $V_s$
SCPTu for Foundation Analyses

• Shallow Foundations
  ▪ spread footings
  ▪ mats or rafts

• Deep Foundations:
  ▪ augered piles
  ▪ driven pilings
  ▪ drilled shafts
  ▪ bored piles
Shallow Foundation Response

Calculated Bearing Capacity, \( q_{\text{ult}} \)

\[
q_{\text{ult}} = N_c \cdot c + (B/2) \cdot \gamma \cdot N_y + \sigma_{vo} \cdot N_q
\]

Limit Plasticity

Applied stress

\[
q_{\text{allow}} = \frac{q_{\text{ult}}}{FS} \text{ with } FS = 3
\]

Elastic Continuum Theory

\( G_{\text{MAX}} = \text{small-strain modulus} \)
Displacement Influence Factors

- Homogeneous Case: Soil Modulus constant with depth
- Uniform Flexible Loading
- Footing resting on a semi-infinite elastic half-space

\[
\text{Displacement: } s = \frac{q \cdot B \cdot I}{E_s}
\]

- \( q \) = applied surface stress
- \( B \) = foundation width (smaller dimension)
- \( E_s \) = equivalent elastic soil modulus
- \( I \) = displacement influence factor from elastic theory (Poulos & Davis 1974)

www.usucger.org
Equivalent Modulus for Monotonic Load Response

\[ E_{\text{max}} = 2G_{\text{max}}(1+\nu) \]

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

\[ \rho_t = \gamma_t/g \]
Modulus Reduction from TS and TX Data

Open = Drained
Closed = Undrained

Mobilized Strength, $\tau/\tau_{\text{max}}$ or $q/q_{\text{max}} = 1/FS$
Modulus Reduction Scheme (Fahey & Carter 1993)

\[ \frac{E}{E_{\text{max}}} = 1 - f \left( \frac{q}{q_{\text{max}}} \right)^{g} \]

Note: \( f = 1 \)
Equivalent Modulus for Foundation Response

- **Initial stiffness from small-strain shear modulus**
  - \( G_{\text{max}} = \rho V_s^2 \)
  - \( E_{\text{max}} = 2G_{\text{max}} (1+\nu) \)

- **Modulus reduction factor (Fahey & Carter 1993):**
  - \( \frac{E}{E_{\text{max}}} = 1 - f \left( \frac{Q}{Q_{\text{ult}}} \right)^g = 1 - (FS)^{-g} \)
  - where \( FS = \frac{Q_{\text{ult}}}{Q} \)
  - Operational \( E = \left( \frac{E}{E_{\text{max}}} \right) \cdot E_{\text{max}} \)
  - for "well-behaved" soils: \( f = 1 \) and \( g \approx 0.3 \)
    i.e., "vanilla clay" and "hourglass sand"
Nonlinear Foundation Displacement Analyses

\[ S_{\text{center}} = \frac{q \cdot B \cdot (1 - \nu^2)}{E_{\text{MAX}} \left[1 - \left(\frac{q}{q_{\text{ult}}}\right)^{0.3}\right]} \]

where

- \( s \) = centerpoint displacement
- \( q \) = applied surface stress;
- \( q_{\text{ult}} \) = ultimate bearing stress;
- \( B \) = footing width
- \( \nu \) = Poisson’s ratio
- \( E_{\text{max}} \) = initial elastic modulus = \( 2G_{\text{max}}(1+\nu) \)
- See Mayne & Poulos (ASCE J. Geot. Engrg. - Jan 2001)

Geotechnical Special Publication GSP No. 41

Measured and Predicted Behavior of Five Spread Footings on Sand
Class “A” Prediction – Texas A&M
ASCE and FHWA Symposium (1994)

Deltaic Sands
ASCE - FHWA Symposium at Texas A&M

Five Footings on Sand:
$B \text{ (meters)} = 1.0, 1.5, 2.5, 3.0 - \text{South, 3.0 - North}$
Class “A” Prediction – Texas A&M
ASCE and FHWA Symposium (1994)

Applied Axial Load, Q (MN)
0 2 4 6 8 10 12

Displacement, $\delta$ (mm)
0 20 40 60 80 100

South Footing
B = 3.0 m

$V_s = 210 \text{ m/s}$

$\gamma_T = 15.5 \text{ kN/m}^3$

$\phi' = 39^0$

$Q_{ult} = f(q_t)$

$K_i = f(V_s)$
FHWA-ASCE Load Tests at Texas A&M

MEASURED FOOTING RESPONSES

PREDICTED LOAD-DISPLACEMENTS

ASCE Settlement '94
Spread Footing Load Tests on Sand

Axial Load, Q (MN)

Vertical Displacement, w (mm)

TAMU Footing Load Tests (GSP 41, 1994)

Predicted Loads, Q (kN)

Measured Loads, Q (kN)

- B = 3 m (South)
- B = 3 m (North)
- B = 2.5 m
- B = 1.5 m
- B = 1.0 m
- Perfect Prediction

Note: The diagram compares measured loads with predicted loads for different footing widths, showing a close alignment with the perfect prediction line.
Class “A” Prediction

Belfast Footing Load Test (June 2001)
European Foundation Prediction Symposium
Belfast Test, Ireland (Data by Lehane, 2001)
Prediction Rating

- 1. Mayne - USA
- 2. Orr - Ireland
- 3. Murry - NZ
- 4. 22 (anonymous)
Predicted $q_{ult} = 92$ kPa

Measured $q_{ult} = 96$ kPa
cone penetrometer = mini-pile

Scaling relationships for force-displacement-capacity response of axial pile foundation

Full-Scale Deep Foundation

$P_t$

$P_s$

$P_b$

$V_s$

$f_s$

$u_2$

$q_T$
AXIAL PILE CAPACITY FROM CONE PENETROMETER

**Method One**
“Direct” CPT Method (Scaled Pile)

- \( q_b = \text{unit end bearing} \)
- \( q_b = \text{func} (\text{soil type}, q_t - u_b, \text{and degree of movement, } s/B) \)
- \( q_b = fctn (\text{soil type, } q_t, \text{ or } f_s \text{ and } \Delta u) \)

**Method Two: Rational or “Indirect” Method**

- \( Q_{\text{Total}} = Q_s + Q_b - W_p \)
- \( Q_{\text{side}} = \sum (f_p \, dA_s) \)
- \( Q_{\text{base}} = q_b \, A_b \)

- \( f_p = \text{func} (\text{soil type, pile type, } q_t, \text{ or } f_s \text{ and } \Delta u) \)

- \( f_p = c_m c_k K_o \, \sigma_{vo}' \, \tan\phi' \)

**Drained:** \( q_b = N_q \, \sigma_{vo}' \)

**Undrained:** \( q_b = N_c \, s_u \)
Unicone CPTu Method

Eslami & Fellenius (1997 CGJ)
www.fellenius.net

A. At each elevation, determine effective cone resistance:
\[ q_E = q_t - u_2 \]

B. Plot \( q_E \) vs \( f_s \) for soil type (see chart).

C. Unit Side Resistance:
\[ f_p = C_s \cdot q_E \]

D. Unit Tip Resistance:
\[ B < 0.4 \text{m}: \quad q_b = q_E \]
\[ B > 0.4 \text{m}: \quad q_b = \frac{q_E}{3B} \]

- **Soil Type**
  - 1. Very soft sensitive soils
  - 2. Soft Clay
  - 3. Stiff clay to silty clay
  - 4. Silt-Sand Mix
  - 5. Sands

- **C_s Value**
  - 0.080
  - 0.050
  - 0.025
  - 0.010
  - 0.004

**Diagram Details**
- Chart axes: 
  - \( q_t \) vs \( u_2 \) (MPa) on the y-axis
  - \( f_s \) (kPa) on the x-axis
- Graph includes lines for each soil type:
  - 1. Very soft clays, sensitive soils
  - 2. Soft clays
  - 3. Silty clays - Stiff clays
  - 4. Silty sands - Sandy silts
  - 5. Sands, Gravelly Sands

**Notes**
- \( B \) = pile width (m)
NCHRP Synthesis 368: Cone Penetration Test

Chapter 8 on Pile Foundations

- www.trb.org
- webforum.com/tc16
- geosystems.ce.gatech.edu
**RIGID PILE RESPONSE**

*Randolph Solution*

\[ Q_{tu} = Q_s + Q_b \]

\[ Q_{su} = \sum (f_p \, dA_s) \]

\[ Q_{bu} = q_b \, A_b \]

Top Displacement, \( w_t \)

\[ w_t = \frac{Q_t \cdot I_\rho}{d \cdot E_{\text{max}} \left[1 - \left(\frac{Q_t}{Q_{tu}}\right)^{0.3}\right]} \]

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

Load Transfer

\[ \frac{Q_b}{Q_t} = \frac{I_\rho}{1 - \nu^2} \]

\( q_b \) = unit end bearing

**Diagram Notes:**
- Direct Unicone Method or Traditional Limit Plasticity + Beta Side Resistance
- \( E_{\text{max}} = 2\rho_t \, V_s^2 \, (1 + \nu) \)
- \( V_s \)
- \( f_s \)
- \( u_2 \)
- \( q_t \)
University of Houston NGES Texas

Situated in stiff overconsolidated Beaumont clay
Augered Cast-In-Place (ACIP) Piles at University of Houston

O'Neill, Ata, Vipulanandan, & Yin (2002)
ACIP Concrete Piles at UH
(O'Neill et al. 2002)

Profile from J. Benoit (2000, GSP 93)

- Beaumont Clay
- Montgomery Formation

\[ d = 0.46 \text{ m} \]
\[ L = 15.2 \text{ m} \]
Load Test of Augercast Pile in Beaumont Clay
Seismic Piezocone Sounding, University Houston

Stiff Beaumont clay
Fissured Clay

Very stiff sandy clay (Montgomery Formation)

\[ d = 0.46 \text{m} \]
\[ L = 15.2 \text{m} \]
**ACIP Pile, University of Houston**

**Input Parameters**

Length $L$ = 15.20 m  
Diam. $d$ = 0.456 m  
$E_{\text{max}}$ = 363,855 kPa  
$Q_{\text{cap.}}$ = 1800 kN

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<th>$Q/Q_{\text{ult}}$</th>
<th>$E/E_{\text{max}}$</th>
<th>$Q_t$ (kN)</th>
<th>$Q_b$ (kN)</th>
<th>$Q_s$ (kN)</th>
<th>$E$ (kPa)</th>
<th>$s$ (m)</th>
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Elastic Influence Factor:

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

Pile Displacements:

$$s = \frac{Q_t \cdot I_\rho}{d \cdot E_s}$$

Load Transfer:

$$\frac{Q_b}{Q_t} = \frac{I_\rho}{1 - \nu^2}$$

(O'Neill, 2000)
ACIP Concrete Piles at UH  (O'Neill, 2000)

Rigid Elastic Pile Solution

Axial Load, Q (kN)

Top Deflection (mm)

\[ Q_{total} = Q_s + Q_b \]

- Predicted \( Q_b \)
- Predicted \( Q_s \)
- Measured Total
- Measured Shaft
- Measured Base
Opelika National Geotechnical Experimentation Site, Alabama
Mean SCPTu at Opelika NGES, Alabama

Applied Load, Q (kN)

Displacement, s (mm)

Elastic Solution with SCPTu Data

Shaft S02
Shaft S04
Shaft S07
Shaft S09
Compressible Pile Solution

Influence factor: \( I_p = \frac{x_1}{x_3} \)

\[
x_1 = 4 \cdot (1 + \nu) \cdot \left[ 1 + \frac{1}{\pi \lambda} \cdot \frac{8}{(1 - \nu)} \cdot \eta \cdot \frac{\tanh(\mu L)}{\xi} \cdot \frac{L}{d} \right]
\]

\[
x_2 = \frac{4}{(1 - \nu)} \cdot \frac{\eta}{\xi} \cdot \frac{1}{\cosh(\mu L)}
\]

\[
x_3 = \frac{4}{(1 - \nu)} \cdot \frac{\eta}{\xi} + \frac{4 \pi \rho_E}{\xi} \cdot \frac{\tanh(\mu L)}{\zeta} \cdot \frac{L}{\mu L} \cdot \frac{d}{d}
\]

The proportion of load transferred from the top to base:

\[
P_b/P_t = x_2/x_3
\]

The proportion of load carried in side shear is:

\[
P_s/P_t = 1 - P_b/P_t
\]

The displacement at the pile toe/base is given by:

\[
w_b = w_t / \cosh(\mu L)
\]

NOTES:
- \( \eta = \frac{d_b}{d} = \text{eta factor} \) (Note: \( d_b = \text{base diameter} \), so that \( \eta = 1 \) for straight shaft piles)
- \( \xi = \frac{E_{sL}}{E_b} = \text{xi factor} \) (Note: \( \xi = 1 \) for floating pile; \( \xi < 1 \) for end-bearing pile)
- \( \rho_E = \frac{E_{sm}}{E_{sL}} = \text{rho term} \). The Gibson parameter can be evaluated from: \( \rho_E = \frac{1}{2} (1 + E_{s0}/E_{sL}) \).
- \( \lambda = 2 \cdot (1 + \nu) \cdot \frac{E_p}{E_{sL}} = \text{lambda factor} \)
- \( \zeta = \ln \left\{ \left[ 0.25 + (2.5 \cdot \rho_E \cdot (1 - \nu) - 0.25) \cdot \xi \right] (2 \cdot L/d) \right\} = \text{zeta factor} \)
- \( \mu L = 2 \cdot (2/\xi \cdot \lambda)^{0.5} \cdot (L/d) = \mu \text{m factor} \)
Cone Rig at I-85 Bridge, Coweta Georgia
Load Test at I-85 Bridge, Coweta County, GA

GDOT Drilled Shaft Load Test:

D = 0.91 m
L = 20.1 m

Load Test Directed by Mike O'Neill
Axial Load Response of Coweta Shaft

Axial Load, $Q$ (kN)

Displacement, $w_t$ (mm)

$Q_{total} = Q_s + Q_b$

- Pred. $Q_s$
- Pred. $Q_b$
- Meas. Total
- Meas. Shaft
- Meas. Base

$Q_t$

$Q_s$

$Q_b$
Pile Load Tests

Dead Weight
www.hindu.com

Reaction Frame
www2.dot.ca.gov

Statnamic Load Test
www.statnamiceurope.com

Osterberg Cell
www fhwa dot gov
GDOT Viaduct at International Boulevard near CNN, Atlanta

Drilled Shaft Load Test by multi-stage O-Cells
GT Class “A” Prediction
March 2003
GDOT Load Test for Viaduct at CNN

Residual Soils (ML/SM)

Partially-Weathered Rock (PWR)

Stage 1 O-cell

Stage 2 O-cell

Constructed Dimensions of Drilled Shaft

2.9 m

2.9 m

11.8 m

11.8 m

6.2 m

6.2 m

d = 1.68 m

d = 1.59 m

d = 1.44 m
Class A Prediction - GDOT Bridge at CNN

GDOT International Blvd. at CNN

Axial Load, $Q$ (MN)

Top Deflection, $w_t$ (mm)

- $Q_t$ Predicted
- O-cell top down
- O-cell Creep Limit
Osterberg Load Cell Test

- High capacity sacrificial hydraulic jack
- Originally installed at pile base
- Juxtaposes side resistance of upper pile segment vs. base resistance
- Continue until ultimate skin friction or end bearing are reached, else capacity of the O-cell
- Multiple O-cell can be used at several elevations within test shaft
- Staged O-cell tests have now reached up to 30+ tons on single drilled shaft

(http://www.loadtest.com/loadtest-uk/about/ocell/3%20O-cell.jpg)
O-Cell Elastic Solution

Rigid pile shaft under upward loading

\[
\frac{P_1}{G_{s1} r_{o1} w_1} = \frac{2\pi}{\zeta_1} \cdot \frac{L_1}{r_{01}}
\]

Rigid pile under compression loading

\[
\frac{P_2}{G_{s2} r_{o2} w_2} = \frac{4}{(1-\nu)\zeta} + \frac{2\pi}{\zeta_2} \cdot \frac{L_2}{r_{o2}}
\]

**Parameters:**
- \(P\) = applied force
- \(L\) = pile length
- \(r_o\) = pile radius
- \(E_p\) = pile modulus
- \(G_s\) = soil side shear modulus
- \(G_{sb}\) = soil modulus below pile base/toe
- \(\nu\) = Poisson's ratio of soil
- \(P_1 = P_2\)
- \(r_m = L\{0.25 + \zeta [2.5 (1-\nu) - 0.25]\}\)
- \(\zeta = \ln(r_m/r_o) = \) soil zone of influence
- \(\zeta = G_{s2}/G_{sb}\) (Note: floating pile: \(\zeta = 1\)
Calgary Drilled Shaft O-Cell Load Test by Seismic Piezocone Tests

Drilled Shaft O-Cell Load Test Dimensions
- \( d = 1.4 \text{ m} \)
- \( L = 14 \text{ m} \)
Evaluation of Calgary O-Cell Shaft Response by Seismic Piezocone Tests

Calgary Foothills Medical Center
O-Cell Load Test Results

O-cell load test data App. A, page 3 of 5
LOADTEST Project No. LOT-9121 (Figure 1)

O-Cell Load, $Q$ (kN)
Displacement, $w$ (mm)

- Loading Down
- Measured Below O-Cell
- Measured Above O-Cell
- Loading Up

$d = 1.4$ m
$L = 10$ m
$L = 4$ m
Cooper River Bridge, Charleston, SC

Deep Foundations: 2.5 m- and 3-m diameter drilled shafts with lengths of 45 to 60 m
Arthur Ravenel Bridge over Cooper River, SC

(Camp, ASCE GeoSupport GSP 2004)
Arthur Ravenel Bridge, Charleston, SC

Meas. Stage 1
Lower O-Cell: Load Down

Meas. Stage 2
Upper O-Cell: Load Down

Meas. Stage 3
Upper O-Cell: Load Up

Shaft diameter
\( d = 2.6 \text{ m} \)

Depth
0 m
10 m
20 m
30 m
40 m

Upper O-Cell
L = 16.3 m
L = 14.2 m
L = 14.0 m
L = 2.5 m

Lower O-Cell

Casing

O-Cell Load, Q (MN)
Displacement, w (mm)
Arthur Ravenel Bridge
over the Cooper River, Charleston, SC

Photo courtesy of Sparky Witte
Seismic Piezocone Sounding
Golden Ears Bridge Site, Vancouver, BC

Tip $q_t$ (MPa)
Sleeve $f_s$ (MPa)
Porewater $u_2$ (MPa)
Shear Wave $V_s$ (m/s)

Depth (m)

$d = 2.6$ m
$L = 74$ m
Application to Osterberg Load Testing

75-m long drilled shaft, Golden Ears Bridge, Vancouver, BC

O-Cell Load (MN)

Measured - Stage 3, Upper Segment Displ.
Measured - Stage 2, Middle Segment Displ.
Measured - Stage 1, Lower Segment Displ.
Elastic Solution - Stage 3, Upper Segment Displ.
Elastic Solution - Stage 2, Middle Segment Displ.
Elastic Solution - Stage 1, Lower Segment Displ.

Pile diameter $d = 2.6$ m
21 m Casing

$L = 44$ m
O-cell 2

$L = 27$ m
O-cell 1

$L = 4$ m
$Q_t$ (stage 1)

$Q_s$ (stage 2)

$Q_s$ (stage 3)
Number of Measurements For Each Test Method

1 SPT
2 CPT
2 DMT
3 CPTu
4 PMT
4 SCPTu
4 SDMT
5 SCPTu

Number of Measurements:

- SPT: N_{60}, q_c
- CPT: f_s, p_0, p_1
- DMT: q_t
- CPTu: f_s, p_0, p_1, u_2
- PMT: P_0, t_{\text{max}}, P_L
- SCPTu: q_t
- SDMT: q_t
- SCPTu: q_t
Geotechnical Site Characterization

More Measurements is More Better
Evaluation of $c_{vh}$ from Porewater Dissipations

Monotonic Response

Dilatory Response

25 mm piezocone $c_h = 0.38 \text{ mm}^2/\text{s}$

Rio Sarapui, Brazil at $z = 8.2 \text{ m}$

Hard OC Taranto Clay (Pane, et al. 1995)

CE-CSSM Solution

Measured at 9.1 m

Approx. CE-CSSM

Monotonic Response

Dilatory Response
SCPTù at Atlanta Airport Runway 5

Five Independent Readings of Soil Behavior: $q_t$, $f_s$, $u_2$, $t_{50}$, and $V_s$. 

![Graphs showing five independent readings of soil behavior: $q_t$, $f_s$, $u_2$, $t_{50}$, and $V_s$.]
Seismic Resistivity Dynamic Penetrometer Test (SRDPT) for hard ground

Dynamic Driver Module (Impact, Sonic)

- Shear Wave Velocity, $V_s$
- Lateral Stress, $\sigma_L$
- Resistivity, $\Omega$
- Tip Stress, $q_d$

SRDPT provides 4 continuous readings with depth
**GT Roto AutoSeis**

- Electro-Mechanical off 12-volt
- AC or DC power; variable speed
- Repeatable, Portable, reach 30-m depths
- Can generate shear wavelets every 1 second
- Patent received in February 2010
Continuous $V_s$ profiling to 45 meters

courtesy Dave Woeller - ConeTec
Continuous-Interval Seismic Piezocone, BC

- $q_t$ (kPa)
- $f_s$ (kPa)
- $u_2$ (kPa)
- $V_s$ (m/sec)

Depth (m) vs. $q_t$, $f_s$, $u_2$, and $V_s$.
Continuous-interval SCPTu at Norfolk, VA

Norfolk Formation (Holocene)

Yorktown Formation (Miocene)

Standard using cross-correlation
Continuous Vs (Spectral: 20 order)
In-Situ Common Link

Italy

Soil Test Rig at Treporti Test Embankment Venice
In-Situ Common Link

Peru

CPT Rig at ConeTec Yard Lima
In-Situ Common Link

Ireland

InSitu CPT rig for University Ireland Galway
In-Situ Common Link

Australia

IGS Cone Rig near Coffs Harbour Australia
In-Situ Common Link

South Africa

CPT Rig
built by
Eben Rust
In-Situ Common Link

Brazil

CPT Rig for Univ. Pernambuco and Federal Univ Grand du Sol
thanks