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Enhanced In-Situ Testing and Geotechnical Site Characterization

*(with emphasis on the Cone Penetration Test and
its derivatives: CPT, CPTu, SCPT)*



Paul W. Mayne, PhD, P.E.
Geotechnical Engineering
Georgia Institute of Technology

Spring 2011

Enhanced In-Situ Testing and Geotechnical Site Characterization

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GT CEE: <http://www.ce.gatech.edu/>

GT In-Situ: <http://geosystems.ce.gatech.edu/researchcenters>

2nd International Symposium on CPT: www.cpt10.com

Chair-TC 102/TC16: ISSMGE In-Situ Testing: www.webforum.com/tc16

5th Deformation Characteristics of Geomaterials (Seoul 2011): isseoul2011.org

Fourth Intl. Conf. Site Characterization (Brazil 2012): www.isc-4.com

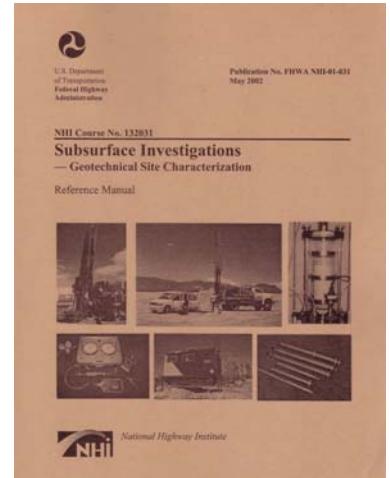
Spring 2011

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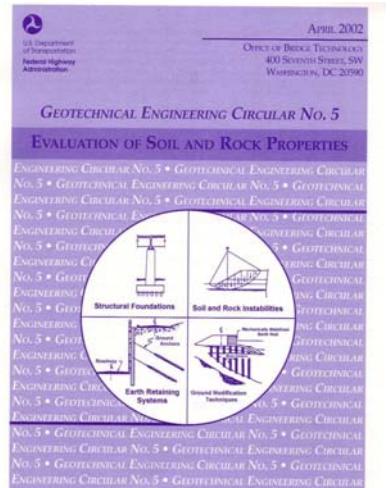
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- **In-Situ Testing Course Notes** on Geotechnical Site Characterization (printable version) and **COLOR PDF** version.
- National Highway Institute Manual (2002): **Subsurface Investigations: Geotechnical Site Characterization**, by Mayne, Christopher, & DeJong, 301 pages



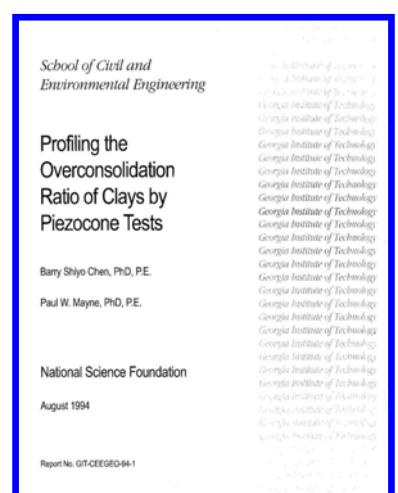
- **FHWA Geotechnical Engineering Circular (GEC 5) (2002): *Evaluation of Soil & Rock Properties***, by Sabatini, Bachus, Mayne, Schneider, and Zettler, Federal Highway Administration, Washington, DC: 385 pages.

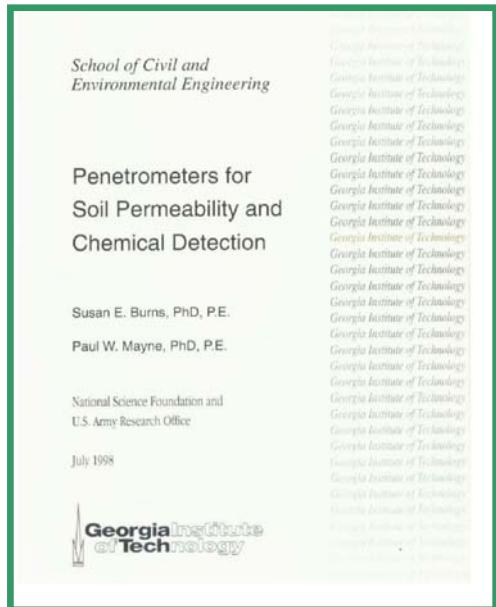


GT In-Situ:

<http://geosystems.ce.gatech.edu/researchcenters/>

- Chen, B. S-Y. and Mayne, P.W (1994). **Profiling Overconsolidation Ratio by Piezocone Penetration Tests**, GTRC Report to National Science Foundation, Washington, DC, 280 pages.





Burns, S.E. and Mayne, P.W. (1998). ***Penetrometers for Soil Permeability and Chemical Detection.*** GTRC Report to Army Research Office and National Science Foundation, Washington, D.C., 144 pages.

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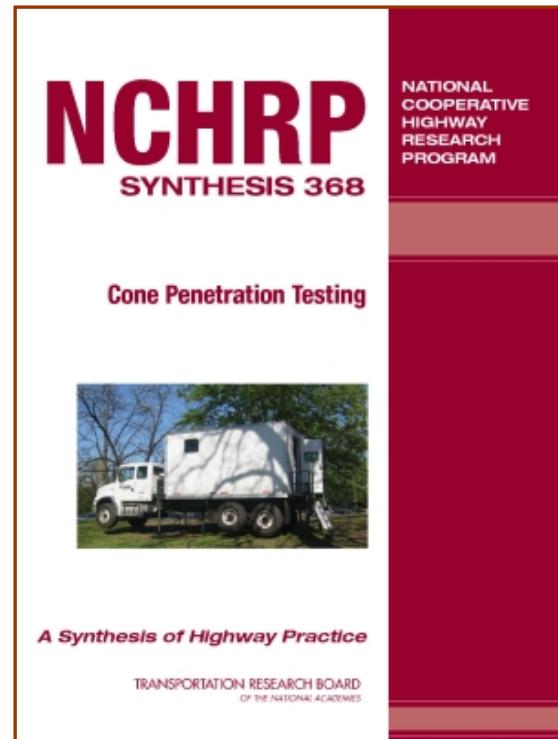
- Schneider, J.A. and Mayne, P.W. (2000). ***Liquefaction Response of Soils in Mid-America Evaluated by Seismic Cone Tests.*** GTRC Report to Mid-America Earthquake Center and National Science Foundation, prepared by Georgia Tech Research Corp: 273 pages.
- Mayne, P.W. and Zavala, G. (2003). ***Cone Penetration Testing for Evaluating Bridge Pile Response.*** GTRC Report E20-H71 to Georgia Dept of Transportation (GDOT Res. Proj. 2021), Forest Park, GA, prepared by Georgia Tech Research Corporation: 250 pages.

- Mayne, P.W. (2007). **NCHRP Synthesis 368 on Cone Penetration Testing**, Transportation Research Board, National Academies Press, National Cooperative Highway Research Program, Washington, D.C., 118 pages:

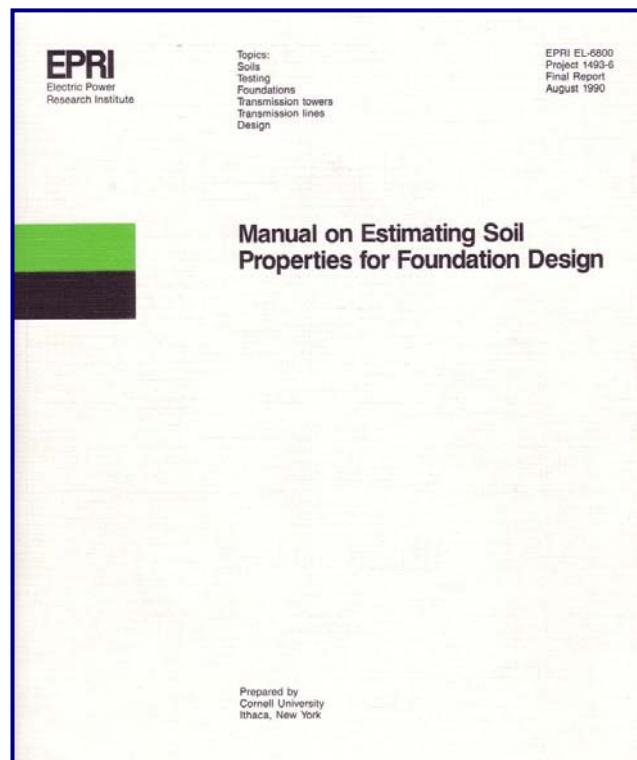
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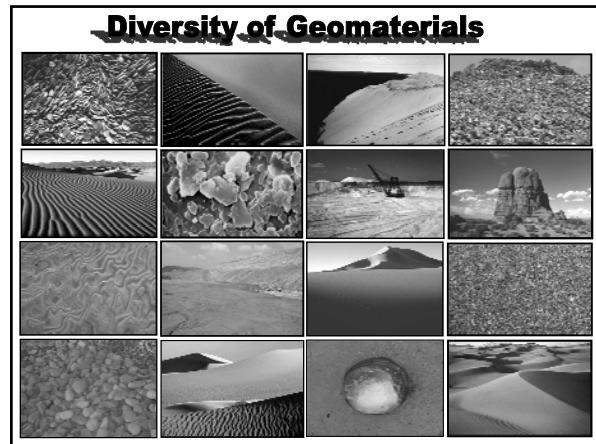
- **Kulhawy, F.H. and Mayne, P.W. (1990). *Soil Properties Manual*, Electric Power Research Institute (EPRI), Palo Alto, CA: 306 p.**
- **Download from:**
www.epri.com
 Search = "EL-6800"



Overview on In-Situ Test Methods

Geotechnical Site Characterization by Enhanced In-Situ Testing

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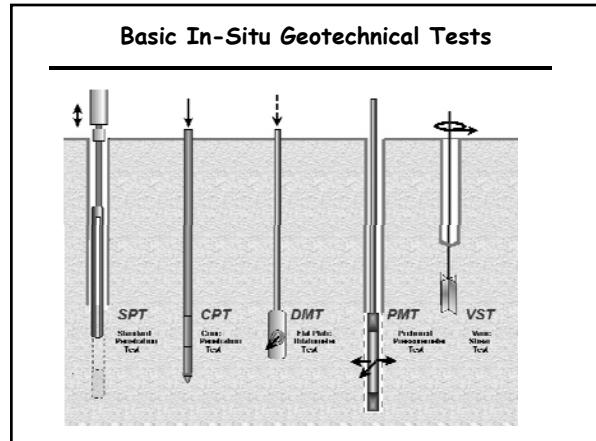


Initial Conditions	
INDICES	
<ul style="list-style-type: none"> ▪ Origin ▪ Geologic Age ▪ Grain Sizes ▪ Mineralogy ▪ Plasticity ▪ Shape ▪ Sphericity ▪ Roundness ▪ Angularity ▪ Packing limits (e_{\max} and e_{\min}) 	STATE <ul style="list-style-type: none"> ▪ Void Ratio, e_0 ▪ Unit Weight, γ_T ▪ Relative Density, D_R ▪ Vertical Stress, σ_v ▪ Hydrostatic, u_o ▪ Saturation, S ▪ Geostatic $K_0 = \sigma_{ho}/\sigma_{vo}$ ▪ Stiffness, $G_0 = G_{\max}$ ▪ Cementation ▪ Fabric, void index I_{vo} ▪ Intact or Fissured

Soil Parameters and Properties	
CONDUCTIVITY	STIFFNESS
<ul style="list-style-type: none"> ▪ Hydraulic: k_v, k_h ▪ Thermal: K_b ▪ Electrical: Ω, ζ ▪ Chemical: D_t 	<ul style="list-style-type: none"> ▪ Stiffness: $G_0 = G_{\max}$ ▪ Shear Modulus, G ▪ Elastic Modulus, E and E_u ▪ Bulk Modulus, K' ▪ Constrained Modulus, D' ▪ Poisson's Ratio, v ▪ Effects of Anisotropy ▪ Nonlinearity
COMPRESSIBILITY	STRENGTH
<ul style="list-style-type: none"> ▪ Recompression, C_r ▪ Preconsolidation, σ_p' ▪ Consolidation, C_v ▪ Virgin Compression, C_c ▪ Swelling index, C_s 	<ul style="list-style-type: none"> ▪ Drained and Undrained ▪ Peak (s_u, c', ϕ') ▪ Post-peak ▪ Remolded/Softened/CS ▪ Residual ▪ Cyclic Behavior
RHEOLOGICAL	
<ul style="list-style-type: none"> ▪ Creep, C_α ▪ Strain rate, $\dot{\epsilon}/\delta t$ ▪ Age (T) 	

Geotechnical Site Characterization

- Drilling & Sampling
 - Rotary drilling & augering; Coring; Sampling
- In-Situ Tests
 - Standard Penetration Test (SPT)
 - Cone Penetration Test (CPT + CPTu)
 - Flat Plate Dilatometer (DMT)
 - Pressuremeter (PMT)
 - Vane Shear (VST)
- Geophysical Methods
 - Mechanical Waves (P-, S-, R-waves)
 - Electromagnetic (radar, resistivity)

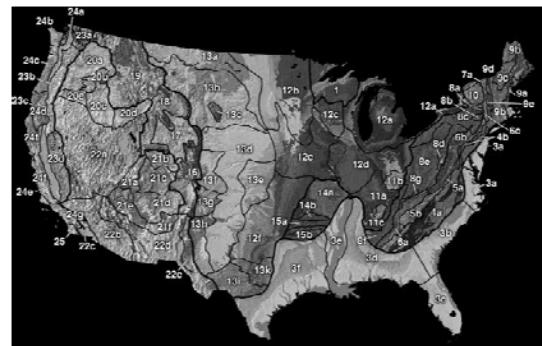


Geology of the Terrestrial World



<http://gsc.nrcan.gc.ca/wmgdb/images/erasep13.gif>

Geology of the United States of America



Drilling & Sampling

Drilling Rig

Methods of Drilling

- Solid Flight Augers
- Hollow Stem Augers
- Rotary Wash Drilling Methods
- Percussive (Hammering) Techniques
- Wireline Drilling
- Continuous Push (Geoprobe)



Truck Mounted Drill Rigs



Soil & Rock Exploration



MoDot Track Rig



CME750 All-Terrain Rig

FHWA-NHI-Subsurface
Investigations

Soil & Rock Exploration



Wireline Rig for Kaolin
Mines, Macon, GA



Water Boring from
Barge for Bridge
Crossing

FHWA-NHI-Subsurface
Investigations

Soil & Rock Exploration



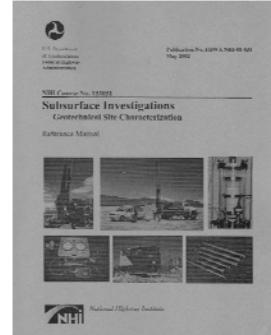
The Explorer
Savannah, GA



SeaCore in
Anchorage

FHWA-NHI-Subsurface

NHI (2002) Subsurface Investigations Geotechnical Site Characterization



National
Highway
Institute

Soil Sampling Methods

- Drive Samples (Split-Spoon; Split-Barrel)
- Hydraulic Push (Undisturbed)
 - Shelby Thin-Walled Tube
 - Piston Sampler
 - Laval Sampler
 - Sherbrooke Type
 - NGI Block Sample
- Rotary Cored Sampling
 - Pitcher
 - Osterberg

} Soft Soils
} Hard Soils

Undisturbed Thin-Walled Tube Sampling



FHWA-NHI-Subsurface Investigations

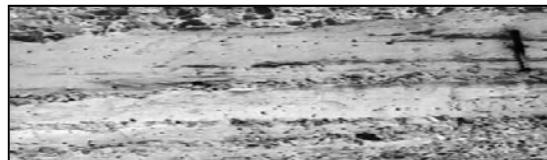
"AN INTRODUCTION TO DRILLING & SAMPLING IN GEOTECHNICAL PRACTICE."

by Jason T. DeJong & Ross W. Boulanger
2nd Edition (2000)
University California-Davis

- Download pictures and full video from:

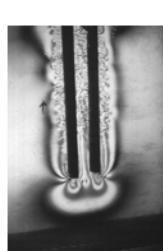
www.usucger.org

Soil Sampling for Geostratification



- Number of Layers
- Depth of Layers
- Soil Type of Layers
- Thickness of Layers
- Soil Consistency
- Groundwater table(s)

Sample Disturbance

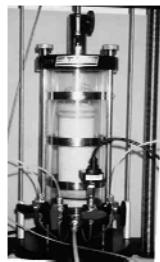


Photoelasticity Studies



Radiography (X-rays) of Tubes

Samples for Laboratory Testing



Oedometers & Consolidometers



Triaxial Apparatus



Direct Shear

Laboratory Soils Testing

- Time-Consuming
- Expensive (Time is Money)
- Tests (per specimen)
 - o Oedometer = \$450 (2 weeks)
 - o AutoConsolidation = \$600 (2 to 3 days)
 - o CIUC Triaxial = \$400 (2 to 3 days)
 - o CKoUC Triaxial = \$1200 each
 - o Resonant Column = \$1800 (3 to 5 days)
 - o Permeability = \$400 (1 to 2 weeks)

Geomaterials

- Natural sands (quartz, silica, feldspar, carbonate, corraline)
- Natural clays (kaolin, illite, smectite, halloysite, montmorillonite)
- Organics, diatoms, shells, ash, salts



DO IN-SITU TESTS

Historical View (Broms & Flodin, 1988)

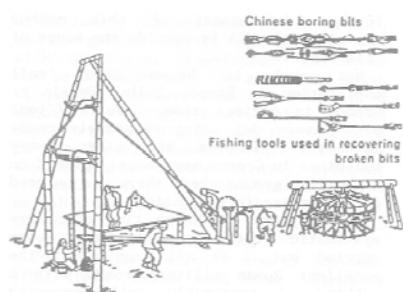


Fig 1a Deep boring 2000 years ago.

Historical View (Broms & Flodin, 1988)

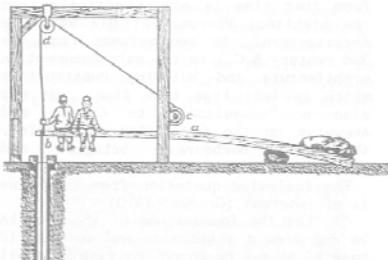


Fig 1b Simple shallow boring, 15th century

Historical View (Broms & Flodin, 1988)

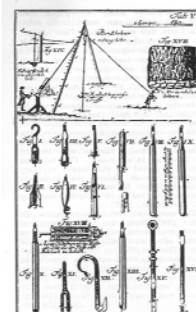


Fig 4 Borning equipment from the 18th century as described by Lehmann in 1714 (Jensen 1990)

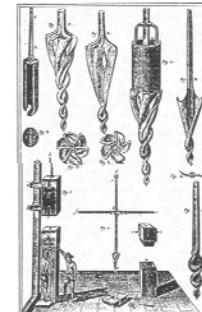


Fig 5 Soil penetration equipment from the 18th century according to Gmelin (1798)

19th Century In-Situ Devices

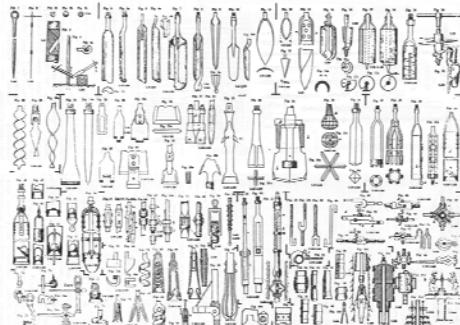
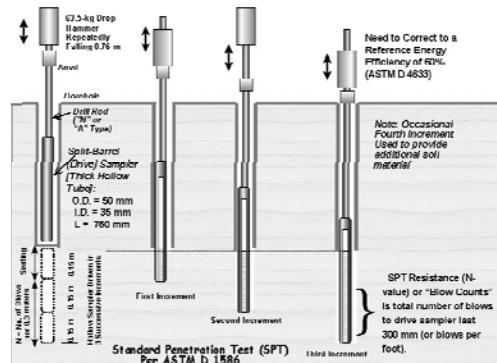


Fig 6 Collection of borning equipment used during the 19th century (Strakosch, 1898)

(Broms and Flodin, Penetration Testing 1988, Orlando)

Standard Penetration Test (SPT)



Standard Penetration Test (SPT)



Split-Barrel Samplers



ENGINEERING SOIL TEST BORING RECORD							November 3, 2001
Elevation	Station	Visual Soil Description	Sample Depth (ft)	Sample Recovery (in)	Soil Sym. K	Penetration N = (blows/ft)	Remarks
1100	0.2	Top soil, grass, and roots				3	Groundwater
	8.0	Soft red-brown fine to medium sandy CLAY (CL)	6.0	18			
1090	14.0	Loose-Firm gray-blue silty medium SAND (SM)	12.0	18		8	at 8.9 ft on 11/02/01
1080	22.5	Firm yellowish slightly silty fine SAND (SP-SM to SP)	20.0	18		28	
	30.0	Firm yellow-white fine to medium SAND, trace silt (SP)	28.5	10		22	
1070	30.0						
<i>Boring Terminated at 30'</i>							
Soil Symbols K (Unified Soil Classification System)				Other Symbols			
Top Soil	CL	CH	SP	Water Level			
Notes:							
N = Penetration in blows per foot (ASTM D-1886)							
N ₆₀ = (ER/0.1 N) ^{1/60} = Energy-Corrected N-value							
ER = Energy Efficiency of Hammer Used							
ER = energy ratio per ASTM D-4633							
Driller: G. Benson				Boring Number: AGB-4			
Date Drilled: November 3, 2001				Job Number: 32335			
Site Location: Tampa, Florida				Test Method: ASTM D-1886			
Hammer Type: Diaphragm Automatic (ER = 82%)				Sampler: Drive (split-barrel)			
Drilling Rig: Rocker (Rocker mounted)				Drilling Rig: CME-850 (truck mounted)			

19th Century In-Situ Devices

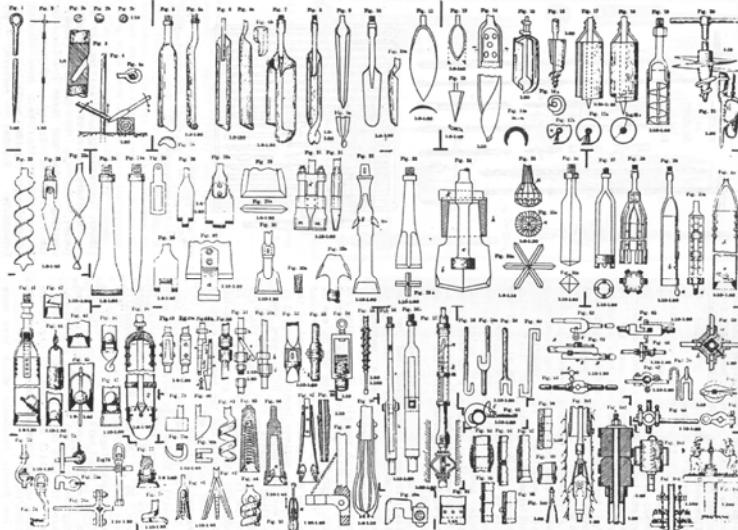
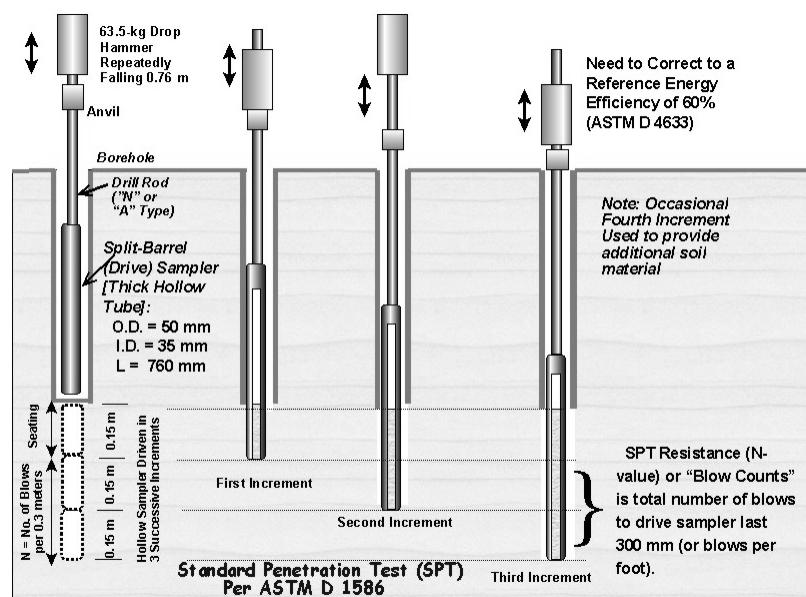


Fig 8 Collection of boring equipment used during the 19th century (Strukel, 1896)

(Broms and Flodin, Penetration Testing 1988, Orlando)

Standard Penetration Test (SPT)

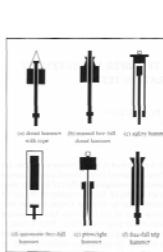


Standard Penetration Test

Advantages	Disadvantages
<ul style="list-style-type: none"> <input type="checkbox"/> Obtain Sample + Number <input type="checkbox"/> Simple & rugged device at low cost <input type="checkbox"/> Suitable in many soil types <input type="checkbox"/> Can perform in weak rocks <input type="checkbox"/> Available throughout the U.S. (worldwide) 	<ul style="list-style-type: none"> <input type="checkbox"/> Obtain Sample + Number <input type="checkbox"/> Disturbed sample (index tests only) <input type="checkbox"/> Crude number for analysis <input type="checkbox"/> Not applicable in soft clays and silts <input type="checkbox"/> High variability and uncertainty

- Obtain Sample + Number
 - Simple & rugged device at low cost
 - Suitable in many soil types
 - Can perform in weak rocks
 - Available throughout the U.S. (worldwide)
- Disturbed sample (index tests only)
 - Crude number for analysis
 - Not applicable in soft clays and silts
 - High variability and uncertainty

SPT Hammer Types

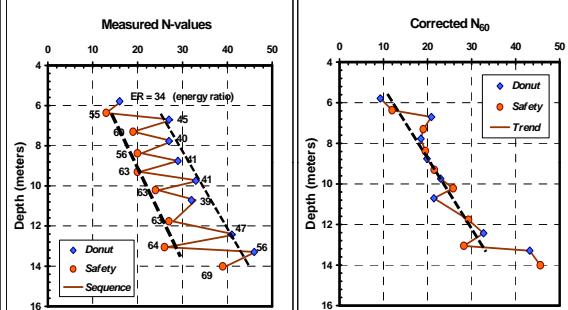


- Pinweight
 - Donut*
 - Donut**
 - Safety*
 - Safety**
 - Auto
- NOTES
*cathead-rope
**trip (free-fall)

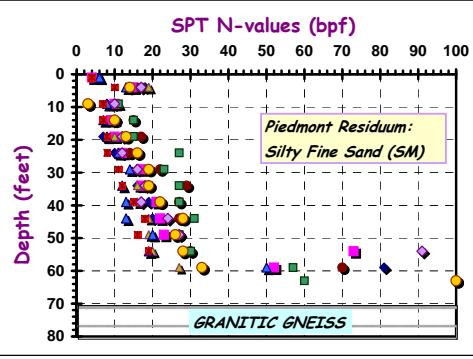
Corrections to SPT N-value

- $N_{\text{measured}} = \text{Raw SPT Resistance (ASTM D 1586)}$.
- $N_{60} = (\text{ER}/60) N_{\text{measured}}$ = Energy-Corrected N Value where ER = energy ratio (ASTM D 4633). Note: $30\% < \text{ER} < 100\%$ with average ER = 60% in the U.S.
- $N_{60} \approx C_E C_B C_S C_R N_{\text{meas}}$ = Estimated corrected N
- For Clean Sands: Normalization of SPT-N value: $(N_1)_{60} = C_N N_{60}$ = Energy-corrected N-value normalized to an effective overburden stress of one atmosphere: $(N_1)_{60} = (N_{60})/(\sigma_{vo}')^{0.5}$ with stress given in atm. (Note: 1 atm = 1 bar = 100 kPa = 1 tsf).

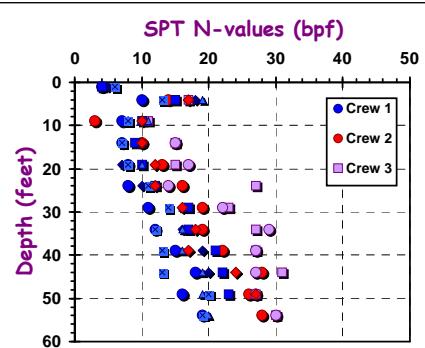
Standard Penetration Test (SPT)

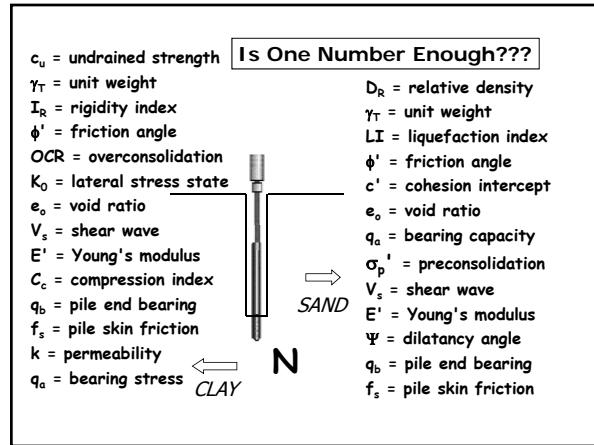
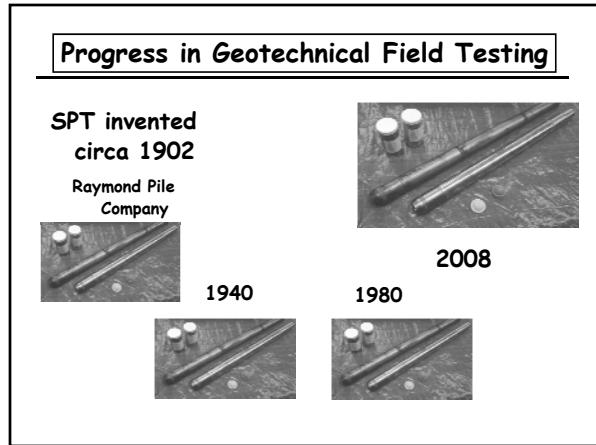
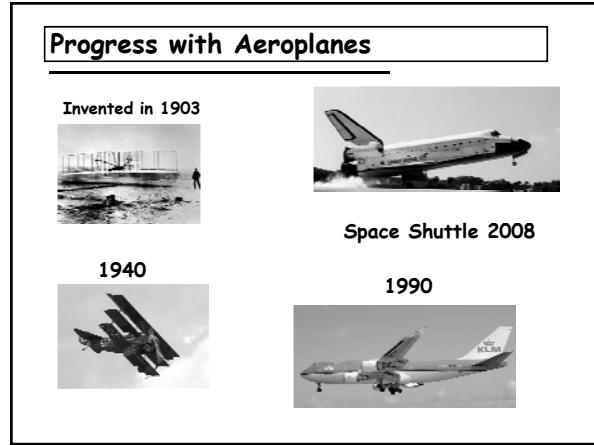
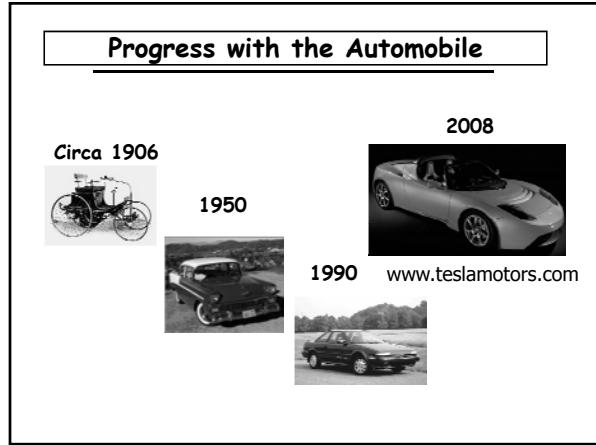
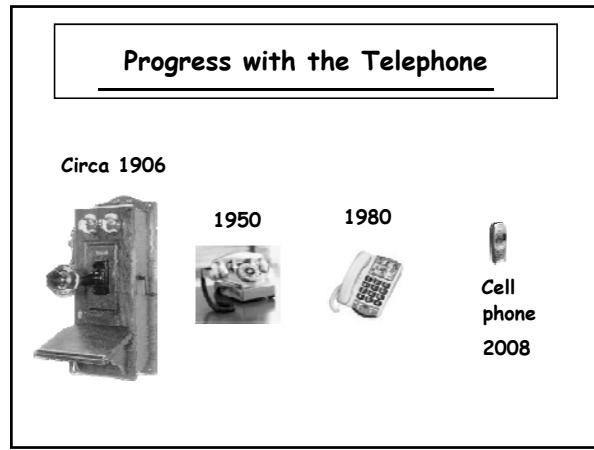
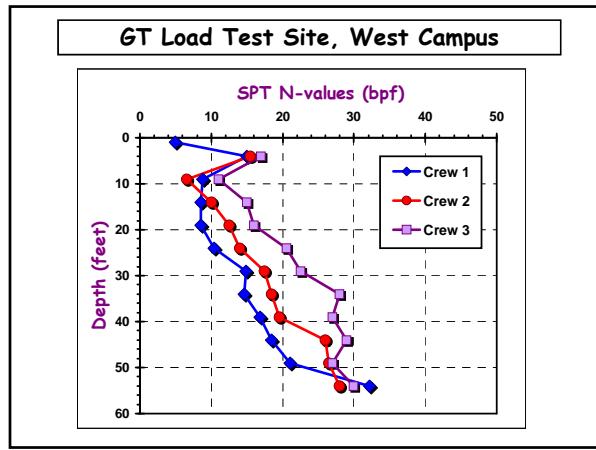


GT Load Test Site, West Campus



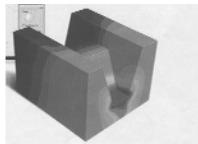
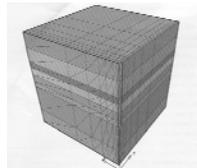
GT Load Test Site, West Campus





Use of In-Situ Test Data

- Numerical Simulations
- Finite Elements
- Strain Path
- Finite Differences
- Discrete Elements



PLAXIS, FLAC, SEEP3d,
ABAQUS, CRISP, ADINA,
GEOSLOPE, FLEA,
Soilvision3d

PLog: Handheld Software Application

- Logs data for:

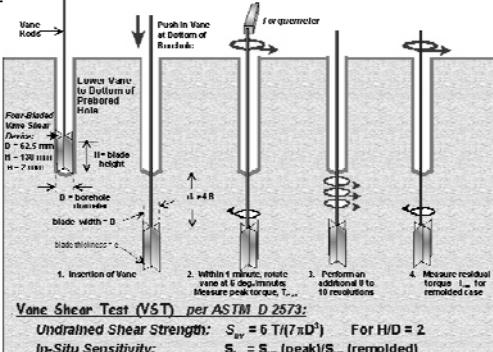
- SPTs
- Split-Spoon Samples
- Stratigraphic Layers
- Groundwater Table
- Rock Coring

- Interfaces directly with gINT for producing soil test boring records

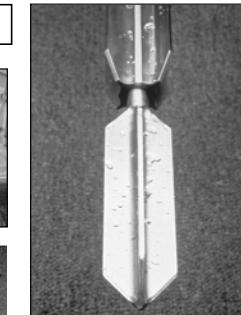
www.dataforensics.net



Vane Shear Test (VST)



Vane Shear Test



Scandinavian Vane

McClelland Offshore Vane

Vane Shear Test



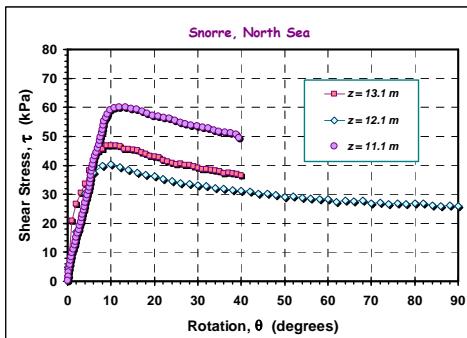
Dutch Vane Equipment, Holland



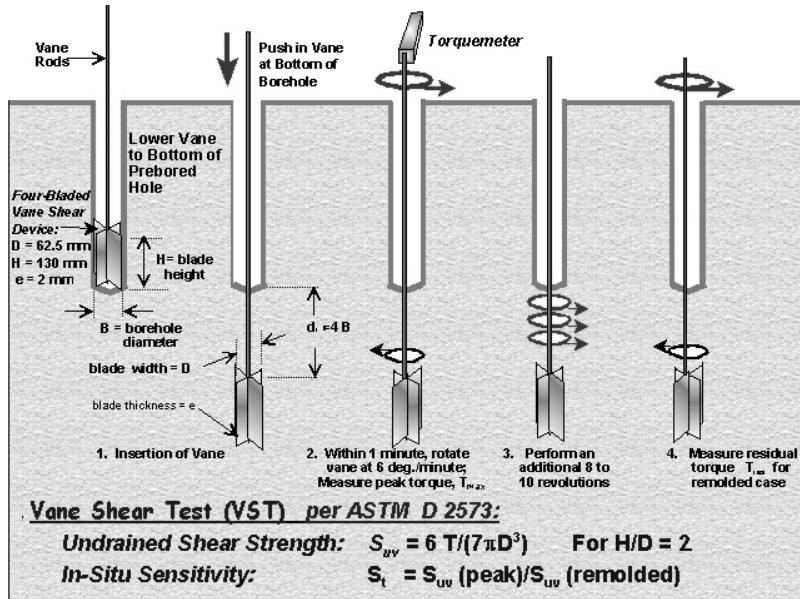
VST in Upstate NY

Remote VST at Snorre, North Sea

(Lunne, Snorrason, and Hauge, 1987)



Vane Shear Test (VST)

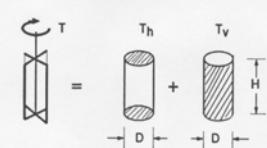


At both ends:

Derivation of s_{uv} for Rectangular Bladed Vane

Derivation of undrained shear strength

Measured torque, $T = T_b + T_v$



$$T_b = 2 \int_0^{(D/2)} (2\pi r^2 \tau) dr$$

For uniform distribution of shear stresses, τ :

$$T_b = 2 (2/3) \pi r^3 \Big|_0^{(D/2)} = (\pi/6) D^3 \tau$$

Along the vertical side shear:

$$T_v = F \cdot r = \tau \pi D H (D/2) = \tau \pi D^2 H / 2$$

Total measured torque (moment):

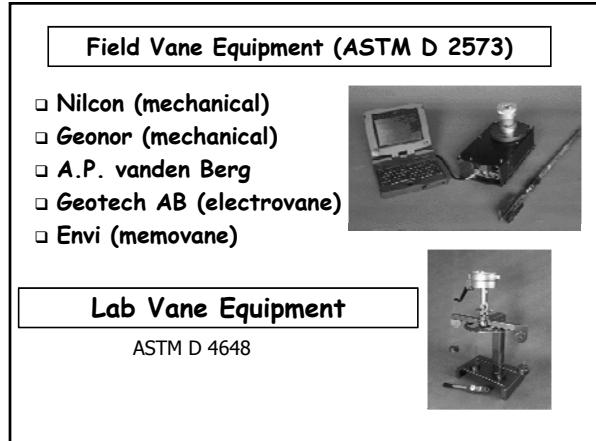
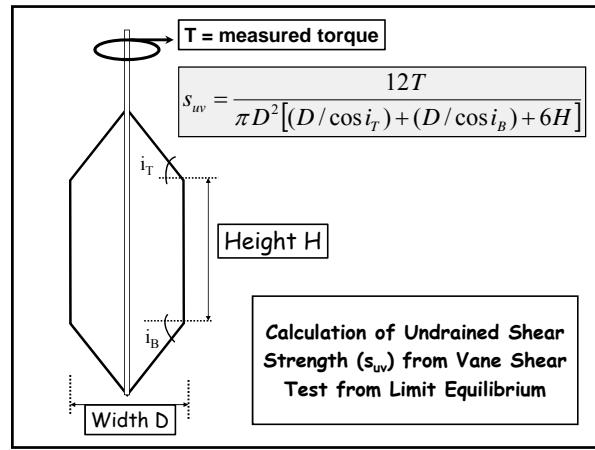
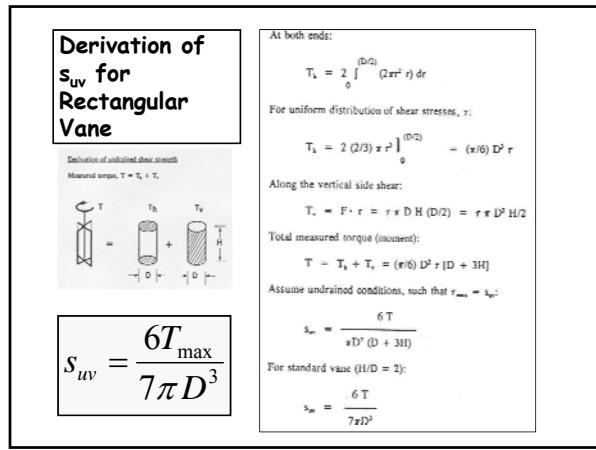
$$T = T_b + T_v = (\pi/6) D^2 \tau [D + 3H]$$

Assume undrained conditions, such that $\tau_{max} = s_{uv}$:

$$s_{uv} = \frac{6 T}{\pi D^2 (D + 3H)}$$

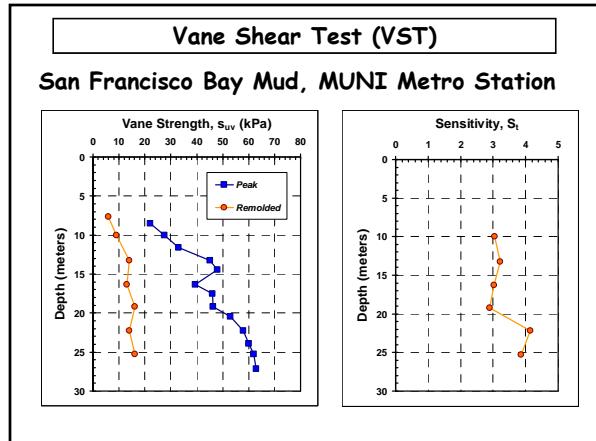
For standard vane ($H/D = 2$):

$$s_{uv} = \frac{6 T}{7\pi D^3}$$



Interpretation of s_{uv} from Vanes with $H/D = 2$ Geometries

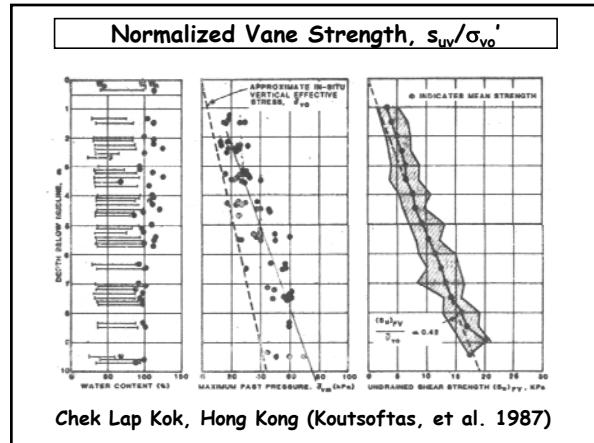
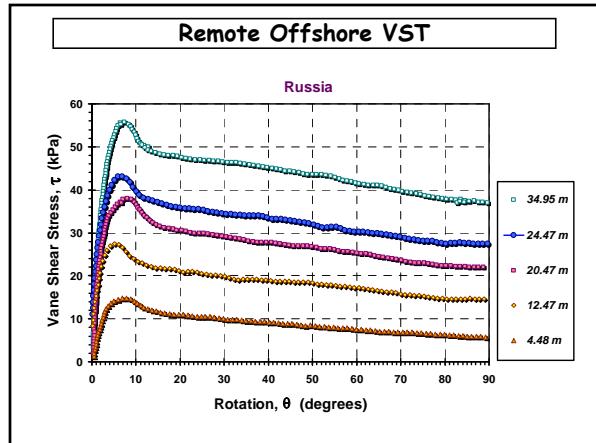
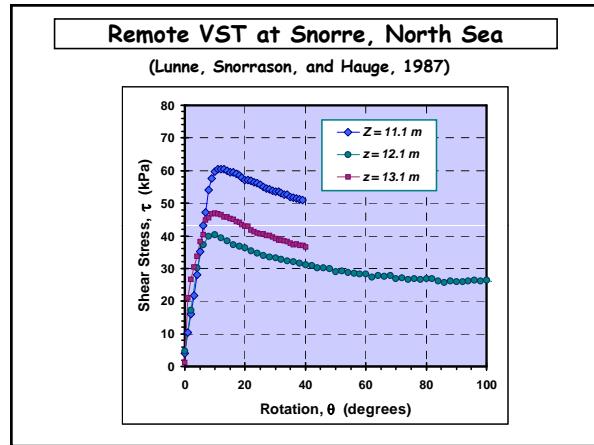
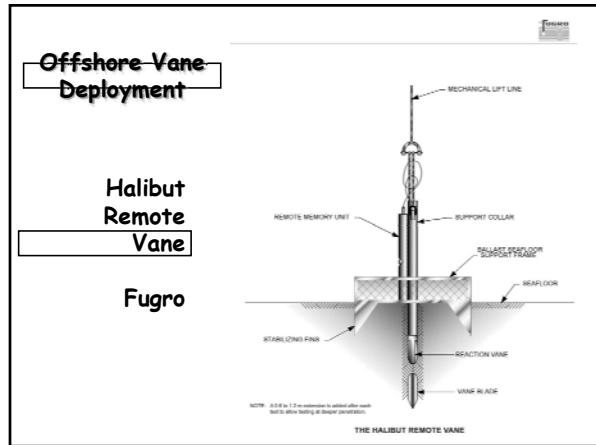
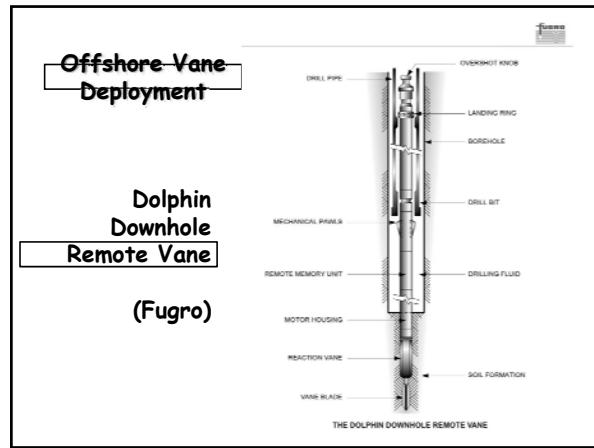
	Rectangular	$S_{uv} = \frac{6T}{7\pi D^3} = 0.273 \frac{T}{D^3}$
	Nilcon	$S_{uv} = 0.265 \frac{T}{D^3}$
	Geonor	$S_{uv} = 0.257 \frac{T}{D^3}$

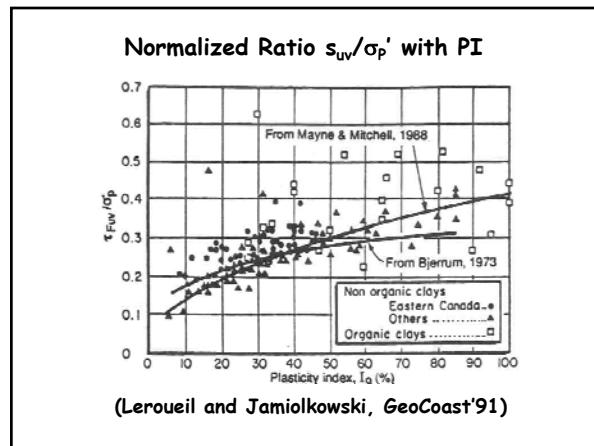
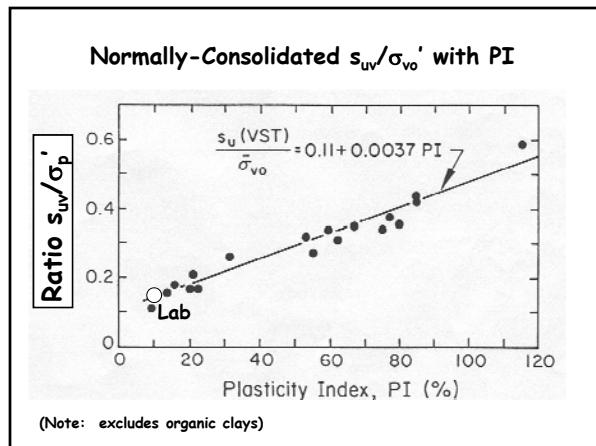
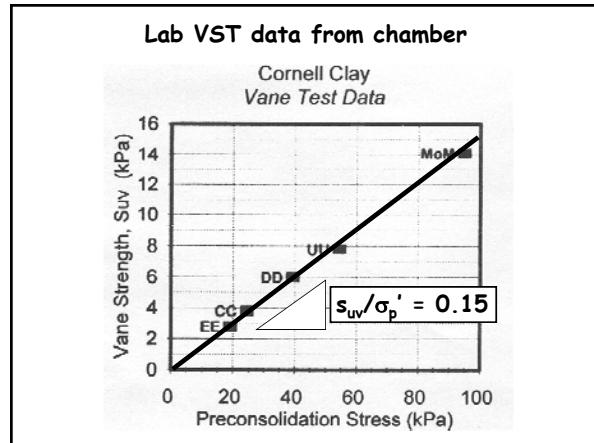
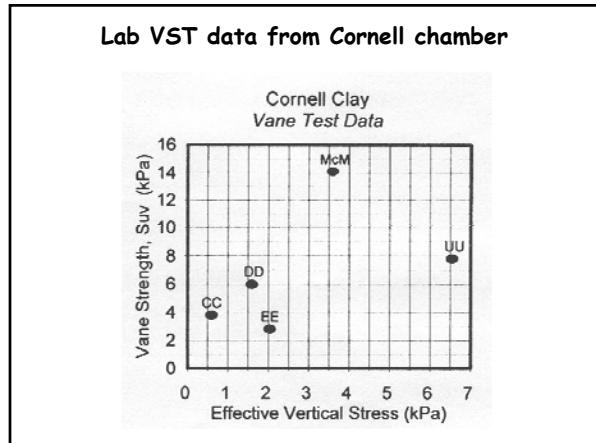
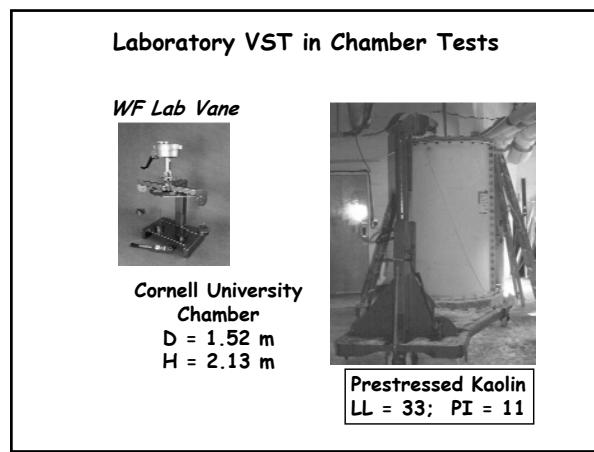
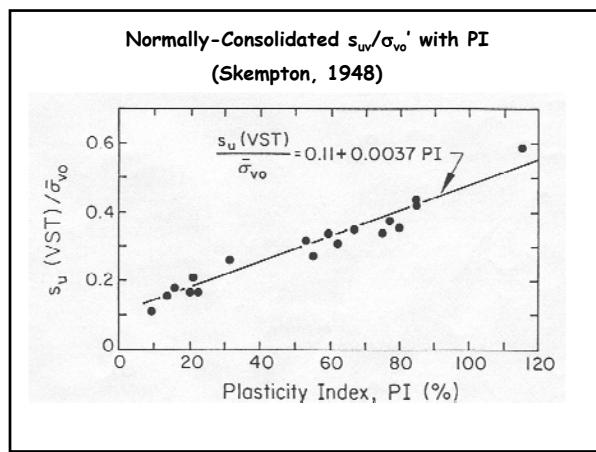


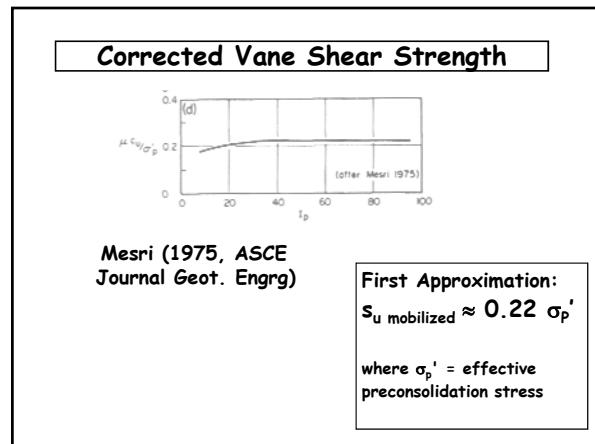
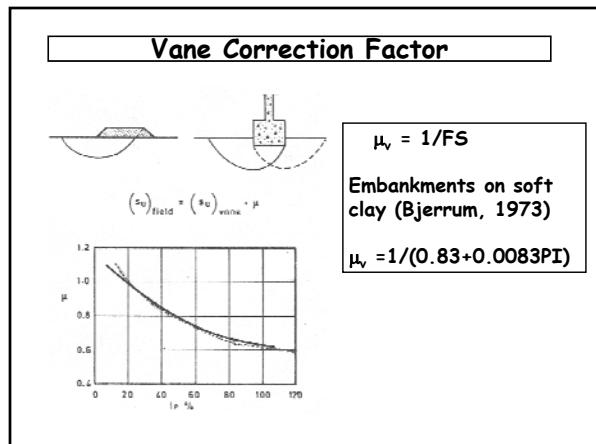
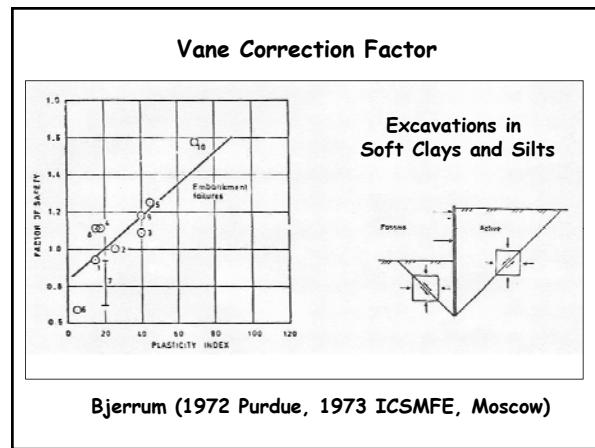
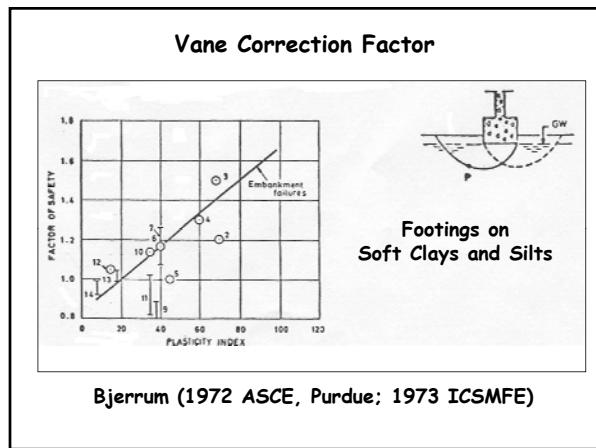
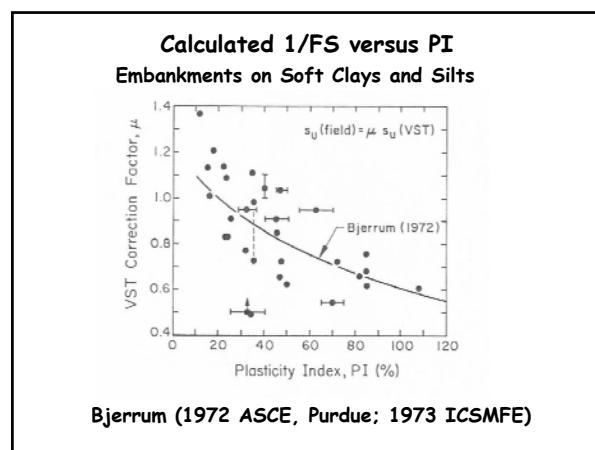
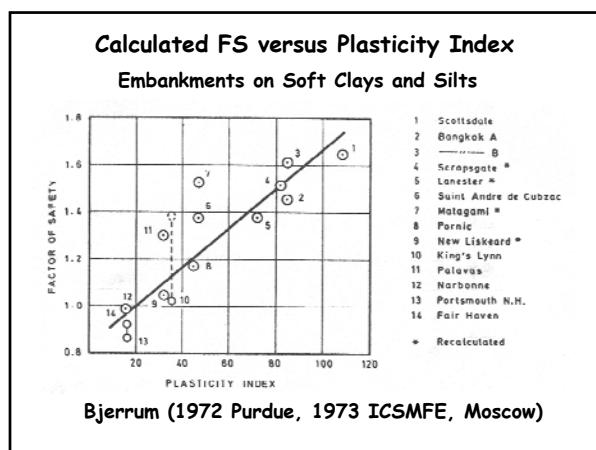
Vane Sensitivity Classification

Category	Sensitivity, S_t
Insensitive	~ 1
Slightly sensitive	1 - 2
Medium sensitive	2 - 4
Very sensitive	4 - 8
Slightly quick	8 - 16
Medium quick	16 - 32
Very quick clay	32 - 64
Extra quick	> 64

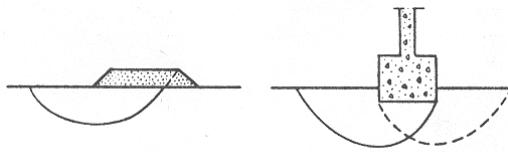
Vane Shear Test	
Advantages	Disadvantages
□ Direct assessment of undrained shear strength of clays	□ Limited to soft to stiff clays & silts with $s_{uv} < 200$ kPa
□ Simple test and equipment	□ Time-consuming in borehole application
□ Measure inplace sensitivity	□ Raw s_{uv} needs empirical correction (onshore ?)
□ Long history of use, particularly for embankment construction, foundations, & cuts	□ Can be affected by sand seams and lenses



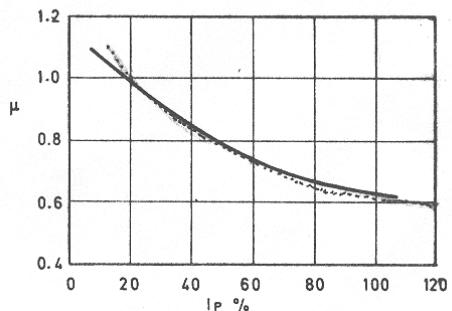




Vane Correction Factor



$$(s_u)_{\text{field}} = (s_u)_{\text{vane}} \cdot \mu$$

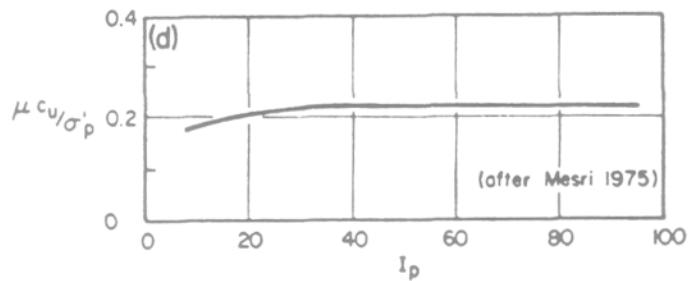


$$\mu_v = 1/FS$$

Embankments on soft clay (Bjerrum, 1973)

$$\mu_v = 1/(0.83 + 0.0083PI)$$

Corrected Vane Shear Strength

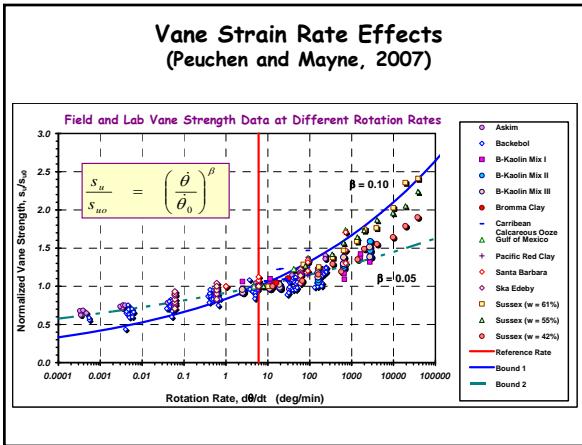
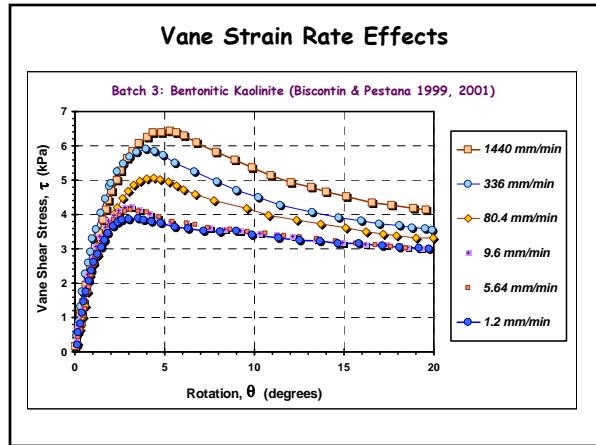
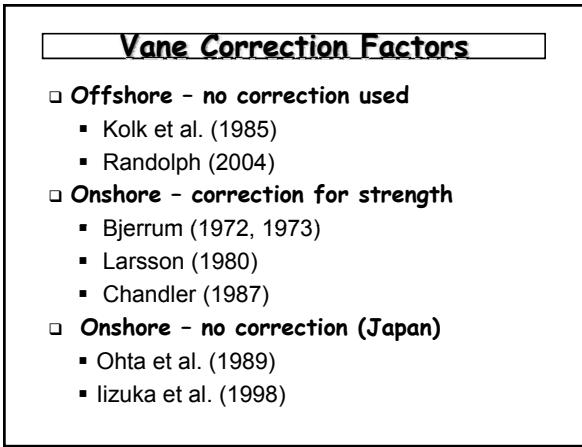
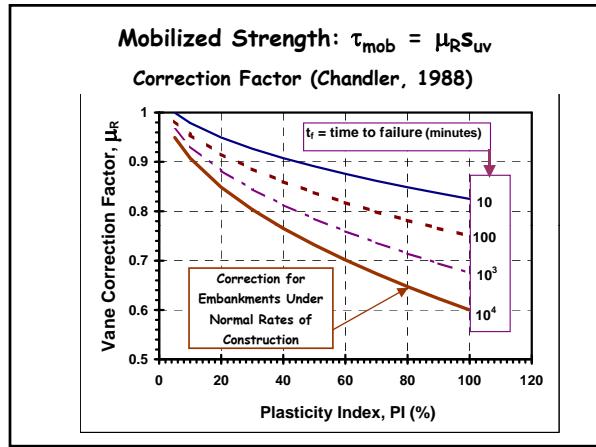
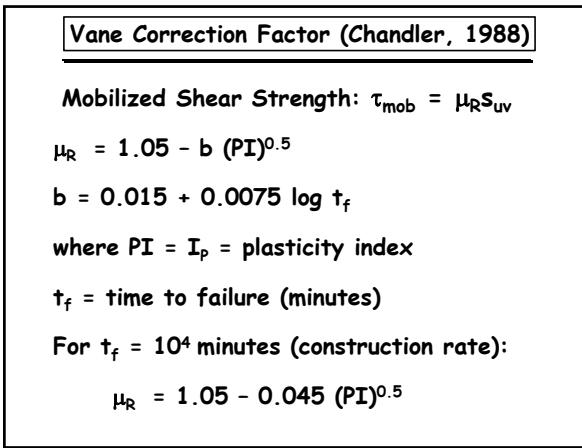
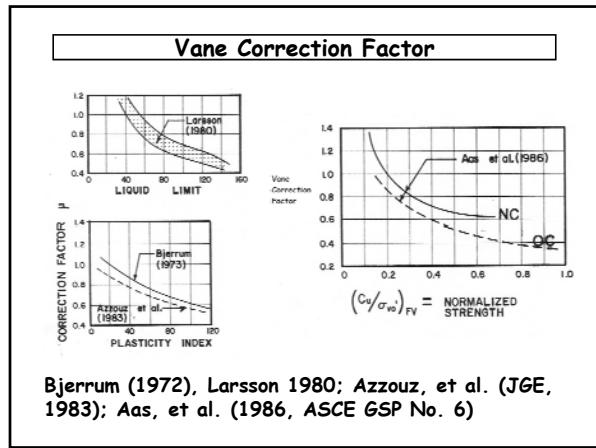


Mesri (1975, ASCE Journal Geot. Engng)

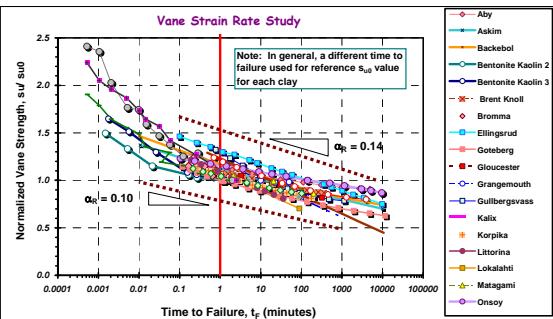
Also see Terzaghi, Peck and Mesri (1996)

First Approximation:
 s_u mobilized $\approx 0.22 \sigma'_p$

where σ'_p = effective preconsolidation stress



Vane Strain Rate Effects (Peuchen and Mayne, 2007)



Vane Strain Rate Effects

(Peuchen & Mayne, SUT 2007)

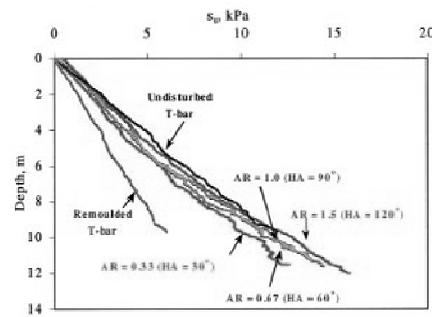
Table 2. Formats Used to Represent Viscous Rate Effects in Vane Shear Data

Rate Measurement	Semi-Logarithmic Form	Power Law Format
Rotation Rate: $\theta = d\theta/dt$	$\frac{s_u}{s_{u0}} = 1 + \alpha \cdot \log\left(\frac{\dot{\theta}}{\dot{\theta}_0}\right)$	$\frac{s_u}{s_{u0}} = \left(\frac{\dot{\theta}}{\dot{\theta}_0}\right)^\beta$
Time to Failure: t_f	$\frac{s_u}{s_{u0}} = 1 + \alpha \cdot \log\left(\frac{t_{f0}}{t_f}\right)$	$\frac{s_u}{s_{u0}} \approx \left(\frac{t_{f0}}{t_f}\right)^\beta$

New Directions in Vane Shear Testing

- Evaluate strength anisotropy using different vane geometries (Silvestri et al. 1993)
- Modulus (E_u or G_u) from τ vs. θ curves
- In-situ K_0 (Wroth 1984)
- Effective stress interpretation (Morris & Williams, 1993, 1994)
- Modified VST with addition readings for self-sufficiency to obtain μ_v ?
- Continuous VST (Univ. Western Australia)

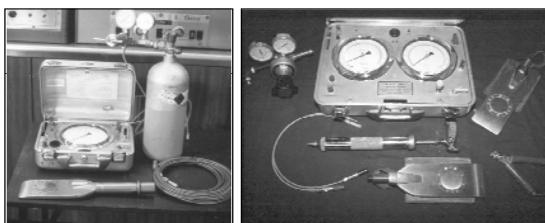
Offshore Continuous Rotating Vane



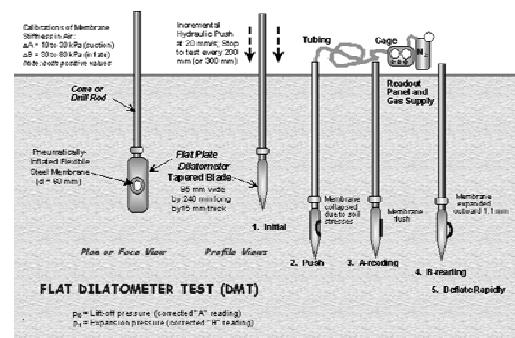
House, Randolph, and Watson (ISOPE 2004)

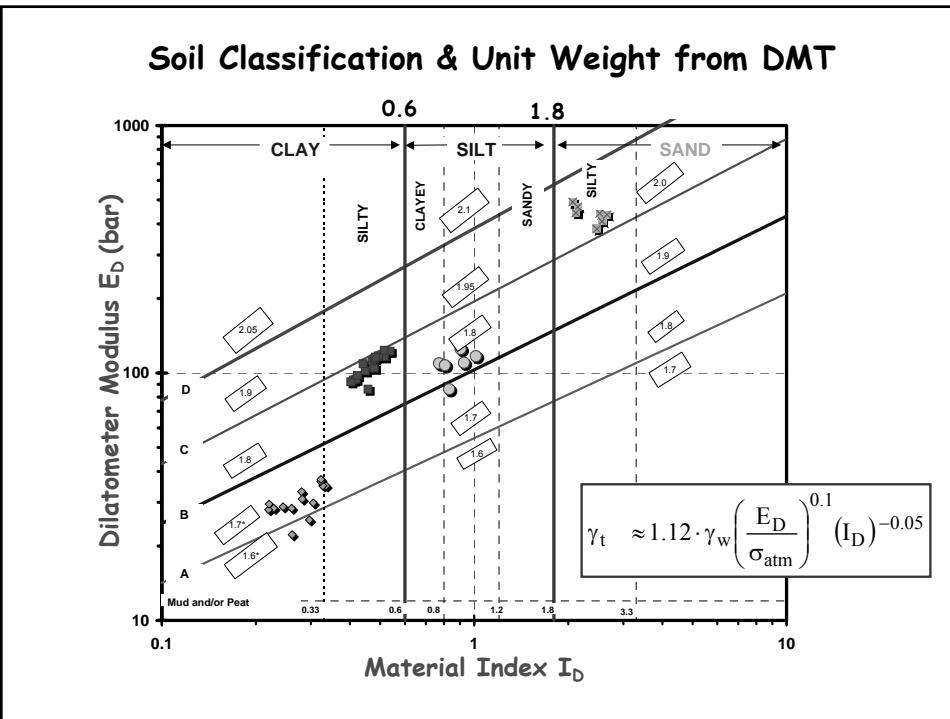
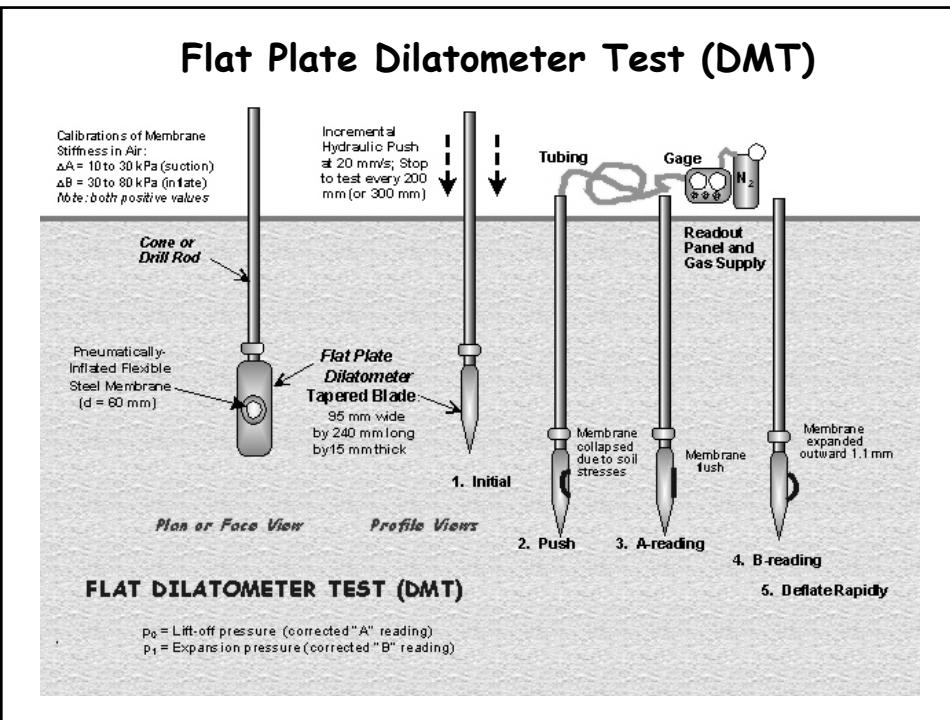
Flat Plate Dilatometer

Marchetti (ASCE JGE, March 1980)
Schmertmann (ASTM GTJ, June 1986)



Flat Plate Dilatometer Test (DMT)



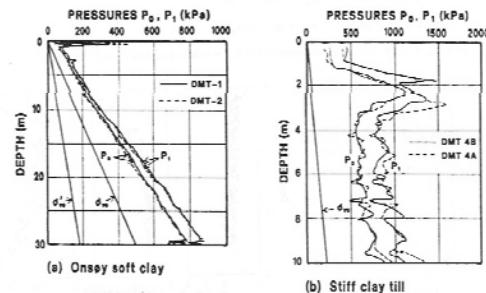


Flat Dilatometer Test

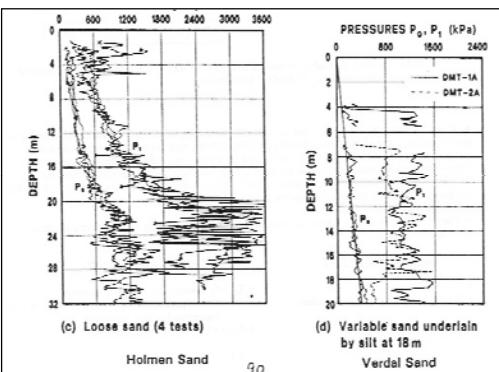
- Calibrations: ΔA , ΔB (positive values)
- Measured Readings: A and B pressures at 20-cm intervals with depth
- Corrections for Membrane Stiffness
- Contact Pressure: $p_0 = 1.05(A+\Delta A)-0.05(B-\Delta B)$
- Expansion Pressure: $p_1 = B-\Delta B$

Examples of DMTs in Different Soil Types

(Lacasse, S. and Lunne, T., 1988, "Calibration of Dilatometer Correlations", Penetration Testing-1988, Vol. 1, Balkema Publishers, Rotterdam).



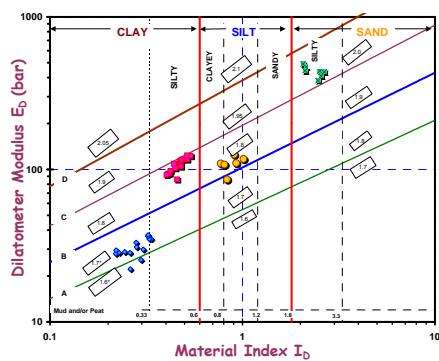
Examples of DMTs in Different Soil Types



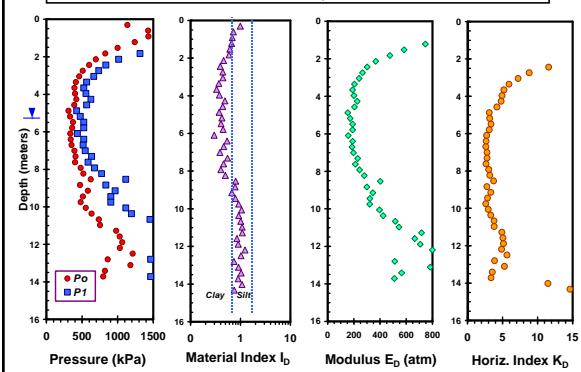
Flat Dilatometer Test

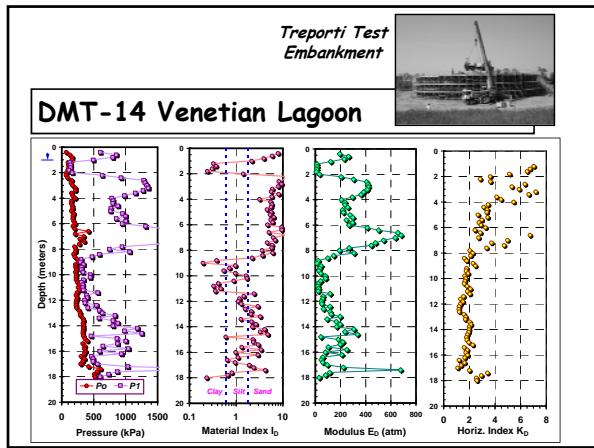
- DMT INDICES from p_0 and p_1 :
- I_D = material index $= (p_1-p_0)/(p_0-u_0)$
 - Clay when $I_D < 0.6$; Sand: $I_D > 1.8$
- E_D = dilatometer modulus $= 34.7(p_1-p_0)$
 - Elastic theory: $E_D = E/(1-v^2)$
- K_D = horizontal stress index $= (p_0-u_0)/\sigma_{vo}'$
 - profiles strength and lateral stress state

Soil Classification & Unit Weight from DMT



DMT Results in Piedmont Residual Silts Charlotte, NC





Flat Dilatometer Test (DMT)

Advantages

- Simple and Robust Equipment
- Repeatable and Operator-Independent
- Quick and Economical
- Theoretical Derivations for elastic modulus, strength, stress history

Disadvantages

- Difficult to push in dense and hard materials
- Primarily established on correlative relationships
- Needs calibration for local geologies

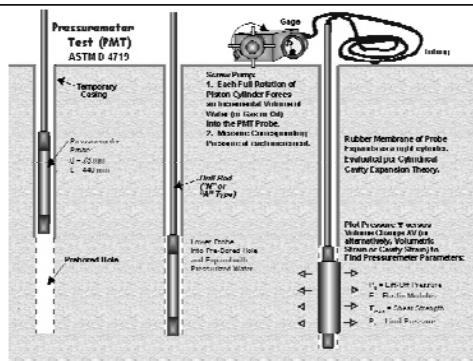
Modifications to conventional DMT

- Measure Thrust Force between pushes → Blade Resistance (q_b)
- Deflation of membrane upon closure ("backward A-reading") = C-reading; corrected for membrane stiffness, designated p_2
- Dissipation of Readings with Time
 - p_0 readings with time (Totani, et al. 1986);
 - p_0 and p_1 and p_2 (Robertson, et al. 1988).
- Offshore Version (Lunne, et al. 1989).

Computerized Flat Plate Dilatometer Systems



Pressuremeter Test (PMT)



Pressuremeter Systems



Pencil Pressuremeter System (Roctest)

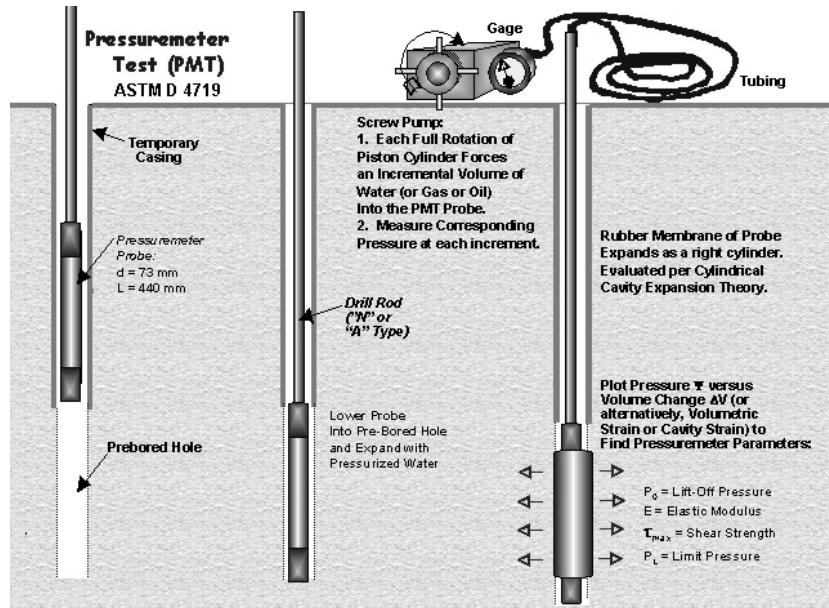


Texam Probe with Screw Pump

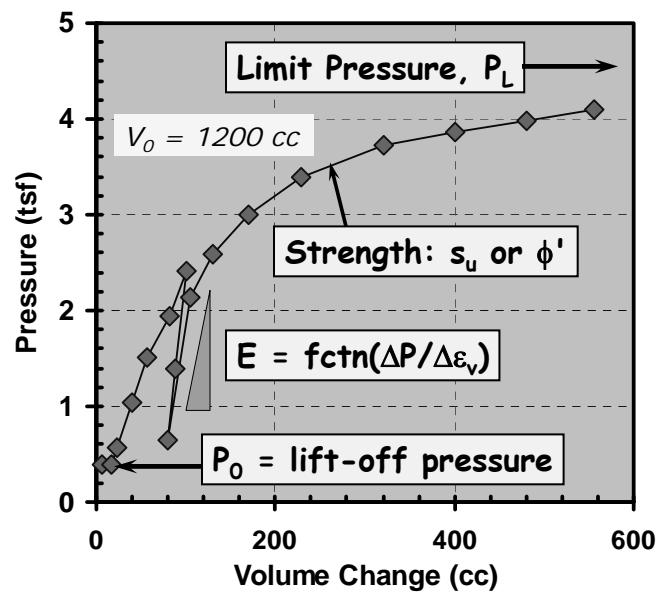


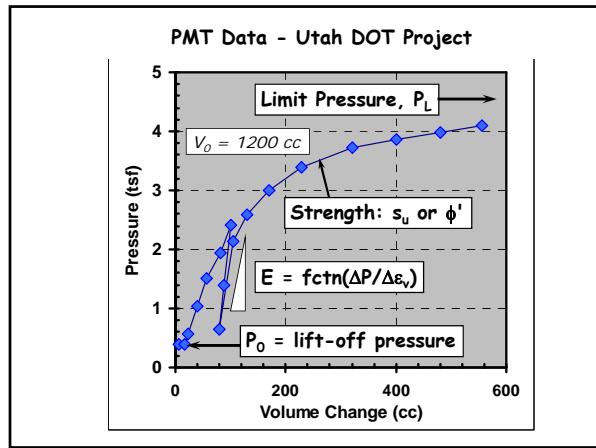
Full-Displacement Type Pressuremeter System (Univ. Florida)

Pressuremeter Test (PMT)



PMT Data - Utah DOT Project



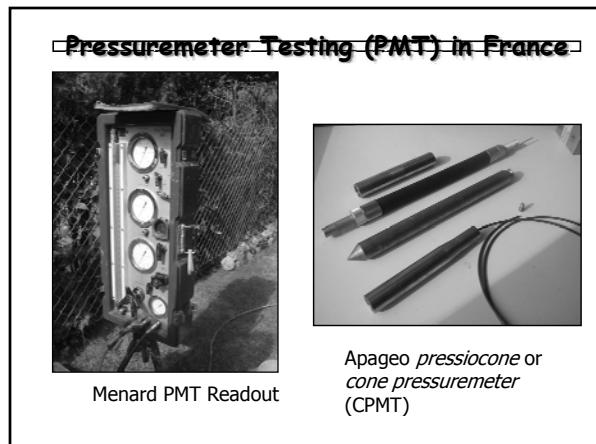
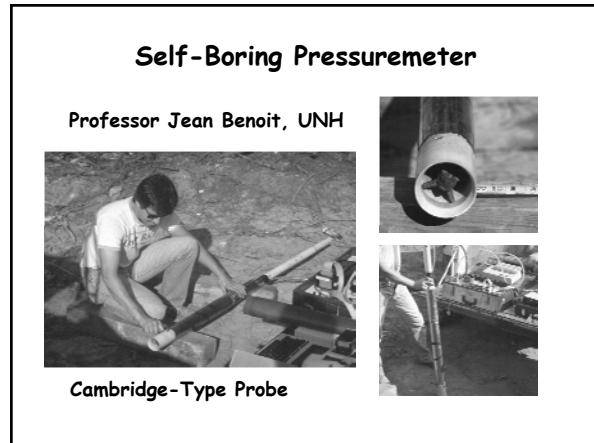
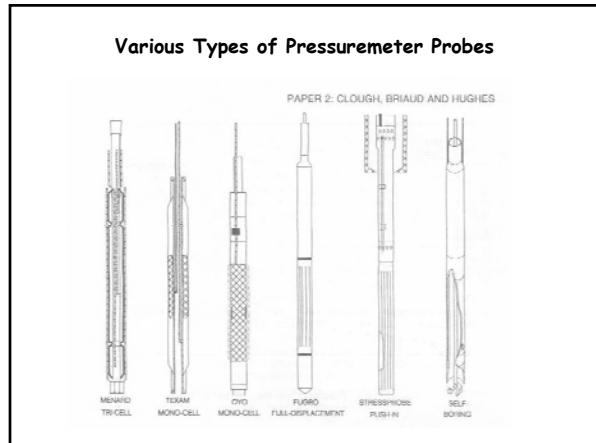


PRESUREMETER TESTS IN CLAY

Cylindrical Cavity Expansion Theory

$$P_L = \sigma_{ho} + s_u [1 + \ln I_R]$$

- Lift-Off Stress: $P_o = \sigma_{ho}$
- Shear Strength s_{uPMT}
- Shear Modulus G [or $E_u = 3G_u$]
- Rigidity Index, $I_R = G/s_u$
- Limit pressure, P_L



Menard Type Pressuremeter Data

PRESUREMETER DATA from Utah

Utah DOT: STA. 13+00 (20 ft left); Depth 5.0 to 7.5 feet
Menard Type PMT Initial Probe Volume
 V_0 (cc)= 1200 Poisson's Ratio
 $\nu = 0.5$

P Measured Pressure (tsf)	ΔV Injected Vol. (cc)	V (cc) Current Creep (cc/min)	V (cc) Current	Volumetric Strains $\Delta V/V$	Cavity Strain ϵ_c	Elastic modulus E (tsf)	Shear modulus G (tsf)	Unload-Reload Cycle E (tsf)	Unload-Reload Cycle G (tsf)
0.39	6.52	4.88	1206.5	0.005	0.005	0.003			
0.40	17.20	4.11	1217.2	0.014	0.014	0.007	84.9	26.0	
0.56	22.40	2.87	1217.2	0.024	0.024	0.010	85.3	23.4	
1.03	40.40	0.01	1240.4	0.033	0.034	0.017	94.8	30.8	
1.50	57.42	0.02	1257.4	0.046	0.048	0.024	98.6	31.7	
1.95	81.90	0.02	1281.9	0.064	0.068	0.034	91.4	29.0	
2.42	101.05	1.43	1301.0	0.078	0.084	0.041	93.5	29.3	
1.96	86.00	0.01	1286.0	0.074	0.074	0.036			312.1 100.7
0.65	69.25	0.01	1289.2	0.053	0.053	0.033			355.3 114.6
2.13	105.50	1.43	1305.5	0.081	0.088	0.043			229.6 73.0
2.58	131.07	3.21	1331.1	0.098	0.109	0.053	78.7	24.3	
3.01	169.72	6.68	1369.7	0.124	0.141	0.068	72.9	22.0	
3.38	229.59	19.52	1428.9	0.161	0.191	0.091	63.2	18.2	
3.73	317.68	42.72	1486.9	0.200	0.220	0.123	52.1	14.9	
3.87	401.52	32.79	1601.5	0.251	0.325	0.165	46.3	12.5	
3.99	476.63	41.21	1679.6	0.286	0.400	0.183	41.9	10.9	
4.09	555.00	51.00	1755.0	0.316	0.463	0.209	38.8	9.8	

PRESSUREMETER TEST (PMT)

- Volumetric Strain: $\Delta V/V_0$
- Current Volume Strain: $\Delta V/V$

Where ΔV = change in volume

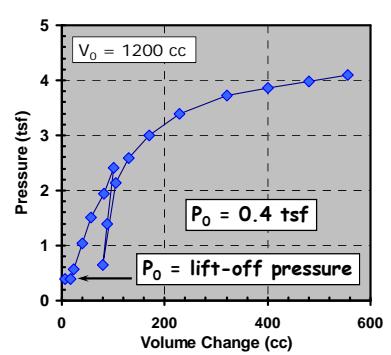
V_0 = initial volume

V = current volume = $V_0 + \Delta V$

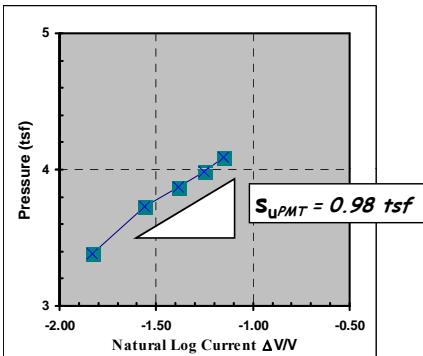
- Define Cavity Strain, $\epsilon_c = \Delta r/r_0$
- Then, $\epsilon_c = (1 - \Delta V/V)^{-0.5} - 1$

(See 2005 In-Situ Notes for more details)

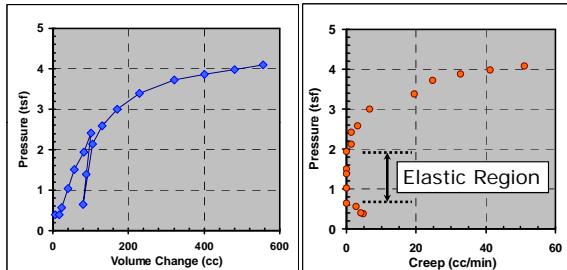
MPMT Data - Utah DOT Project



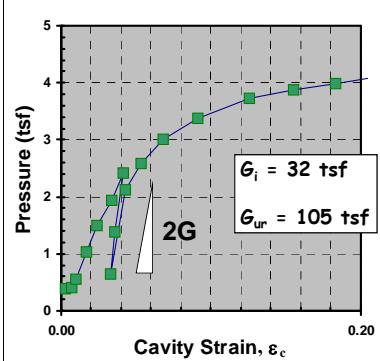
Menard Type Pressuremeter Data



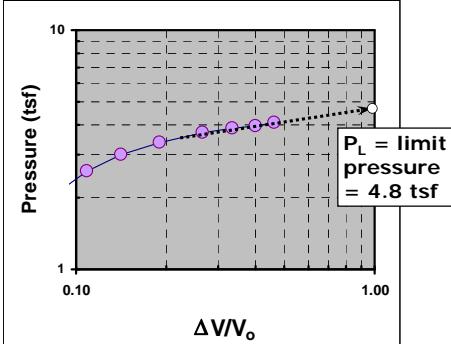
Menard Type Pressuremeter Data



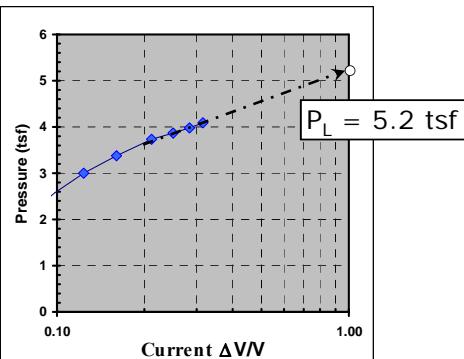
Menard Type Pressuremeter Data



Menard Type Pressuremeter Data



Menard Type Pressuremeter Data



Cylindrical Cavity Expansion Theory

$$P_L = \sigma_{ho} + s_u [1 + \ln I_R]$$

$$4.8 = 0.4 + 0.98 [1 + \ln(32.7)]$$

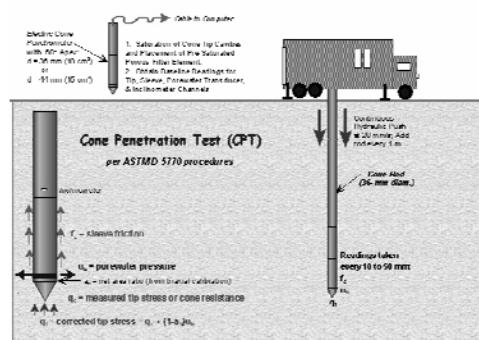
- Lift-Off Stress: $\sigma_{ho} = 0.4$ tsf
- Shear Strength s_u (pmt) = 0.98 tsf
- Shear Modulus $G_i = 32$ tsf
- Rigidity Index, $I_R = G/s_u = 32.7$
- Limit pressure, $P_L = 4.8$ tsf

Unload-Reload G = 105 tsf

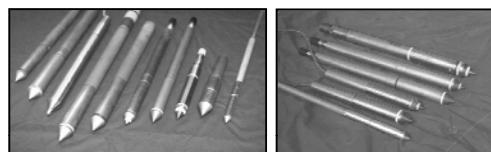
Rigidity Index, $I_R = 107$

Calculated $P_L = 5.9$ tsf versus measured $P_L = 5.2$ tsf

Cone Penetration Testing (ASTM D 5778)



Cone Penetrometer Testing



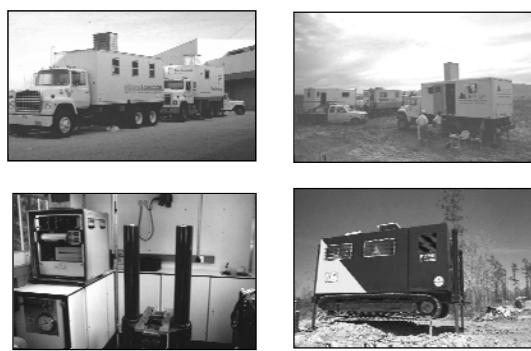
- Electronic Steel Probes with 60° Apex Tip
- ASTM D 5778 Procedures
- Hydraulic Push at 20 mm/s
- No Boring, No Samples, No Cuttings, No Spoil
- Continuous readings of stress, friction, pressure

Cone Penetration Vehicles



Mobile 25-tonne hydraulic pushing rigs

Cone Penetration Vehicles



CPT Track Truck - A.P. van den Berg



Cone Penetration Test (CPT)

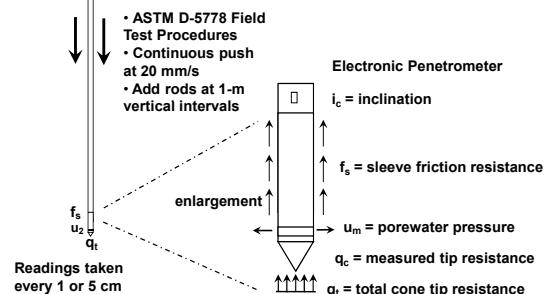


Paul W. Mayne, PhD, P.E.
Geosystems Engineering Group
School of Civil & Environmental Engrg
Georgia Institute of Technology

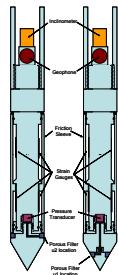
Cone rig with hydraulic pushing system



Cone Penetration Test (CPT)



Cone Penetration Test (CPT)



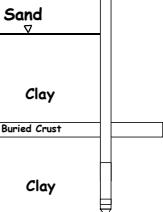
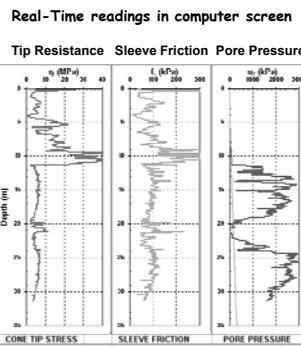
- Electronic Probes with 60° Apex Tip
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- Continuous readings of stress, friction, pressure

Cone Penetration Vehicles



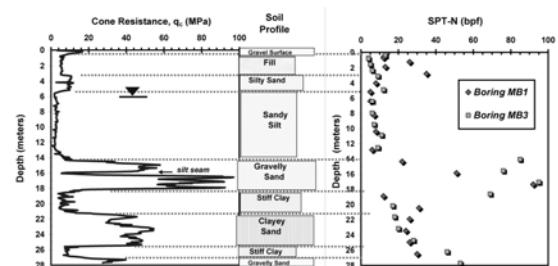
Cone Penetration Testing (CPT)

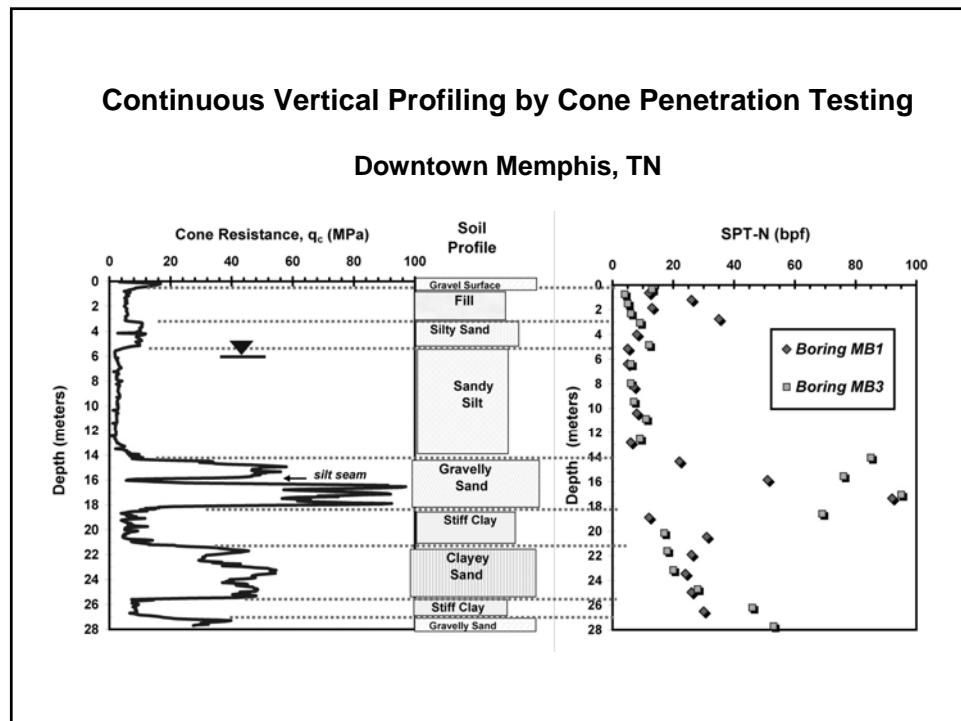
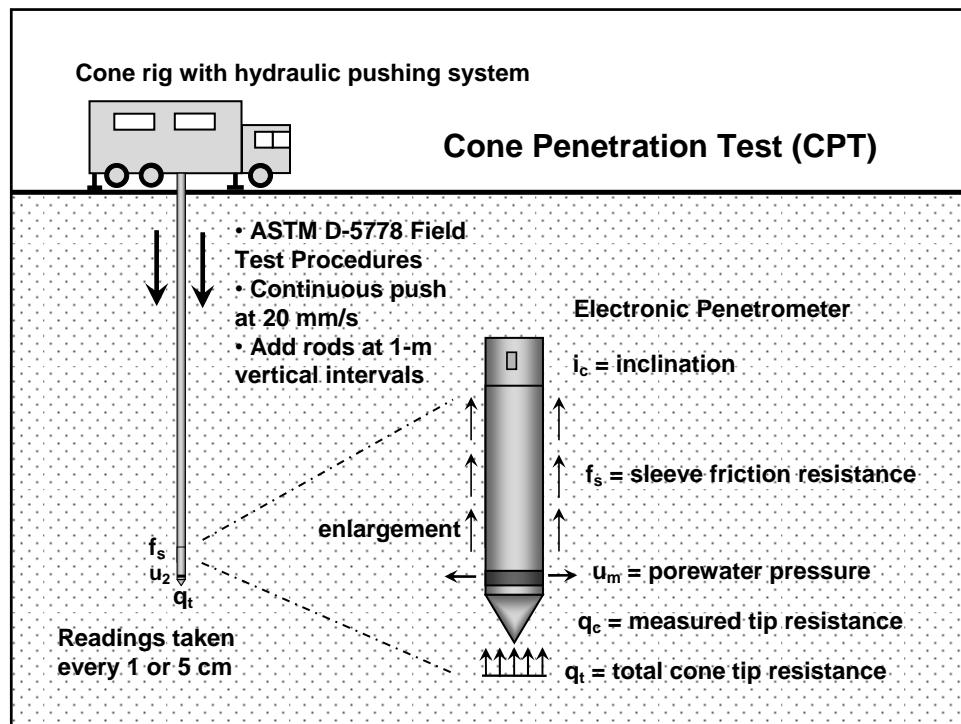
Penetration at 2 cm/s

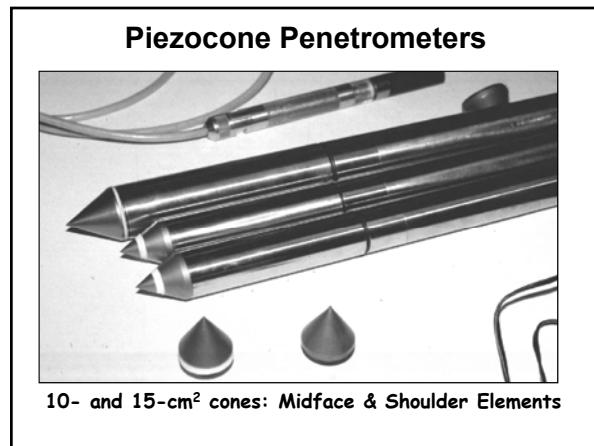
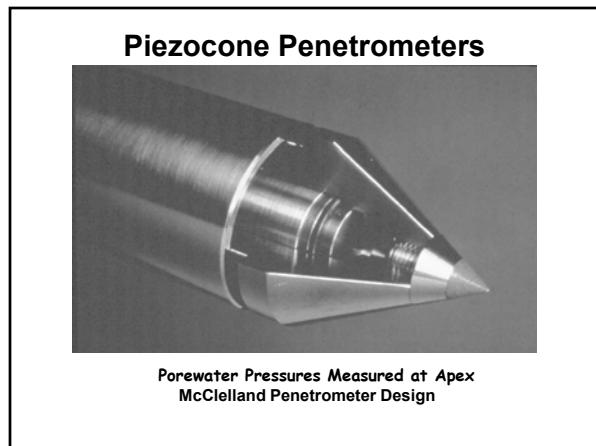
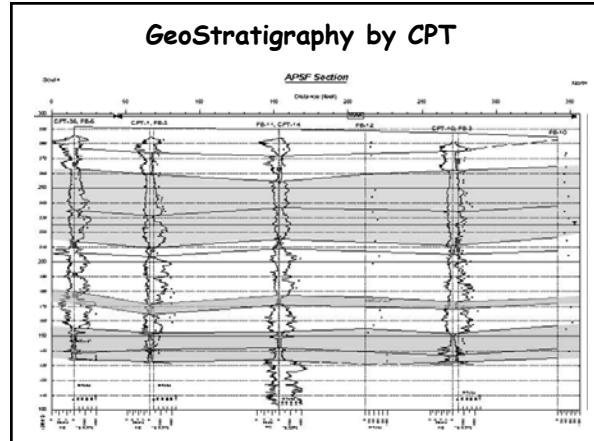
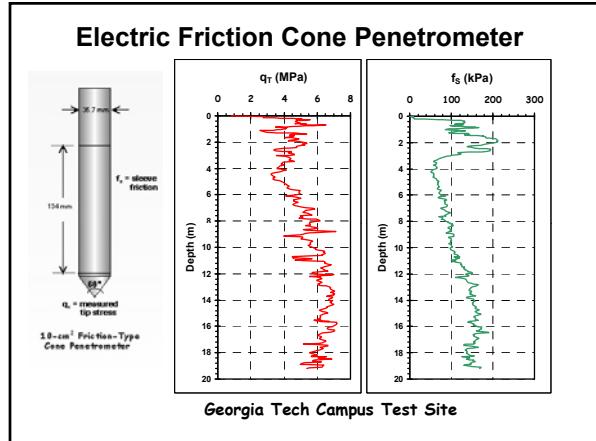
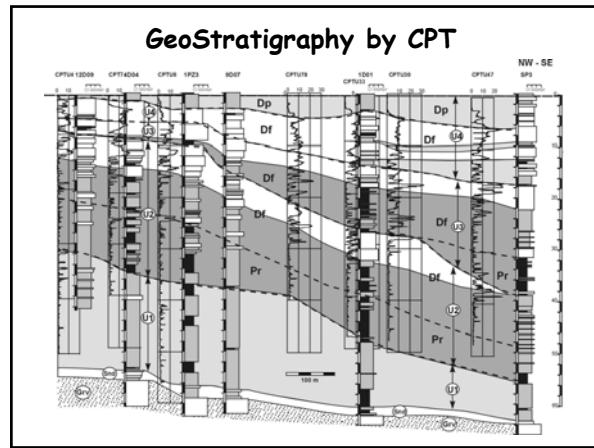
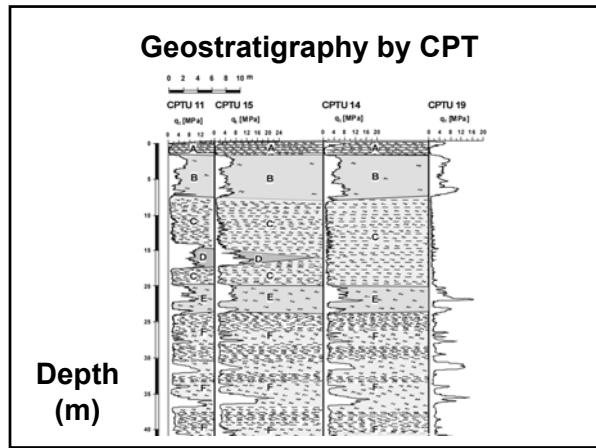


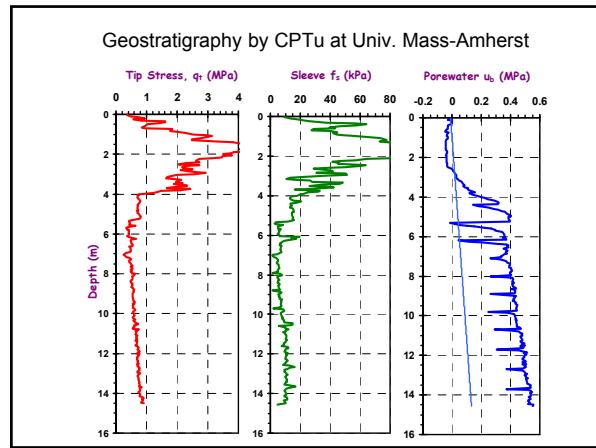
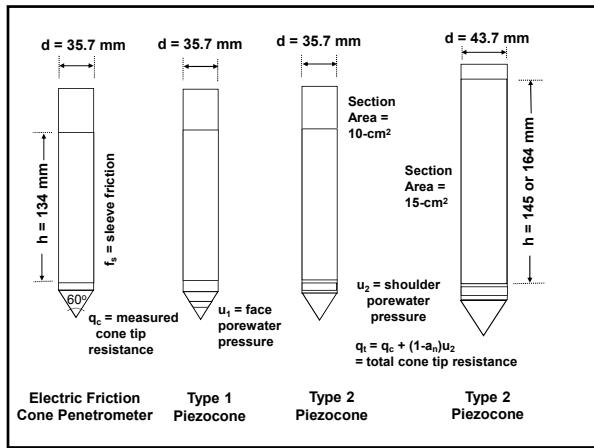
Continuous Vertical Profiling by Cone Penetration Testing

Downtown Memphis, TN









Types of Cone Penetration Tests

Type of CPT	Acronym	Measurements Taken	Applications
Mechanical Cone Penetration Test	MCPT	q_c (or q_t , and f_s) on 20-cm intervals. Uses inner & outer rods to convey loads uphole.	Stratigraphic profiling, Fill control, Natural sands, Hard ground
Electric Friction Cone	ECPT	q_c and f_s (taken at 1- to 5-cm intervals)	Fill placement, Natural sands, Soils above the groundwater table
Piezocene Penetration Test	CPTu and PCPT	q_c , f_s , and either face u_1 or shoulder u_2 (taken at 1- to 5-cm intervals)	All soil types. Note: Requires u_2 for correction of q_c to q_t .
Piezocene with Dissipation	CPTü	Same as CPTu with monitoring of u_1 or u_2 during decay with time	Normally conducted to 50% dissipation in silts and clays.
Seismic Piezocene Test	SCPTu	Same as CPTu with downhole shear waves (V_s) at 1-m intervals	Provides fundamental soil stiffness with depth: $C_{saw} = \rho V_s^2$
Resistivity Piezocene Test	RCPTu	Same as CPTu with electrical conductivity or resistivity readings	Detect freshwater - salt water interface, Index to contaminant plumes.

Procedures for CPTu

Porous Element Materials

- Sintered Metals
- Ceramics
- Plastics (disposable)

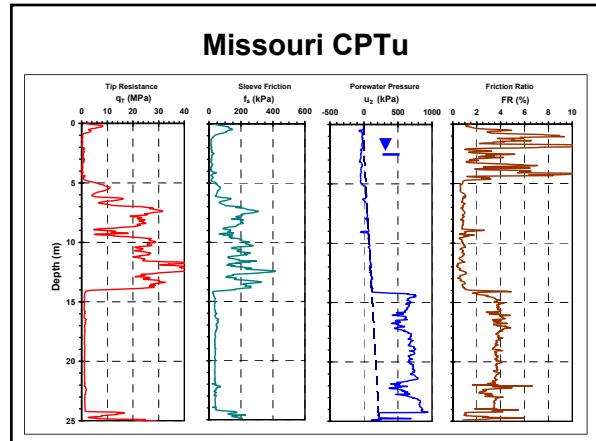
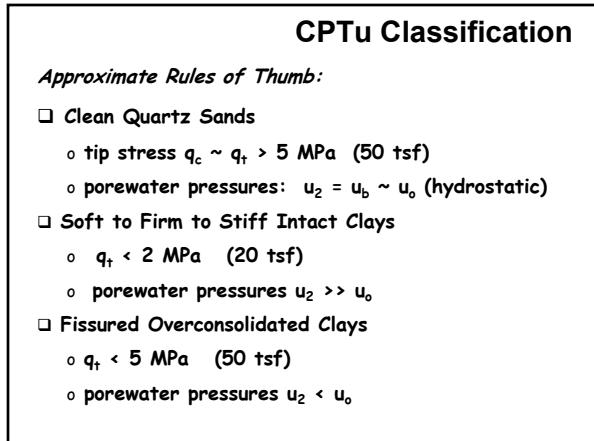
Saturation of Porous Elements:

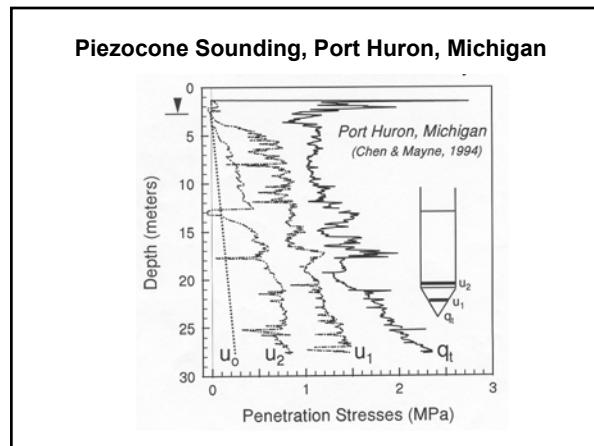
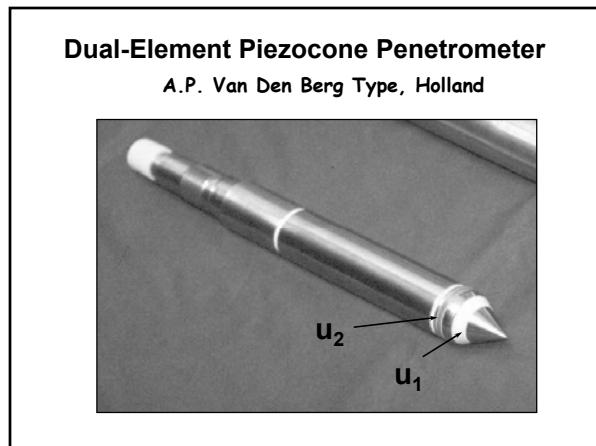
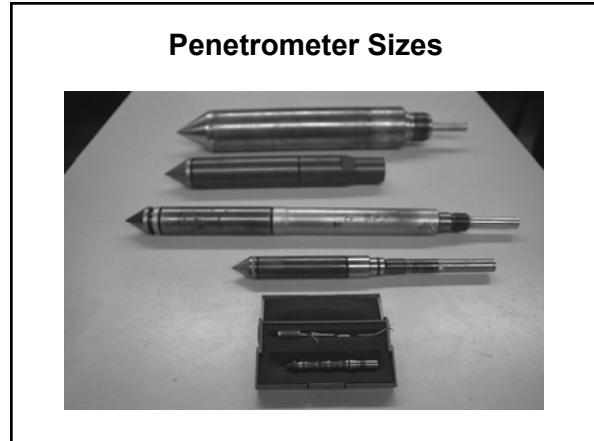
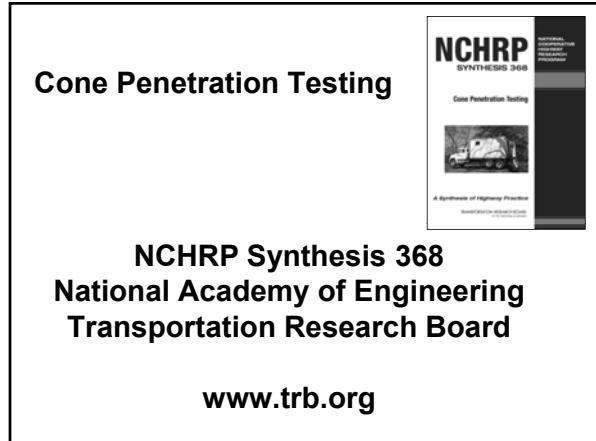
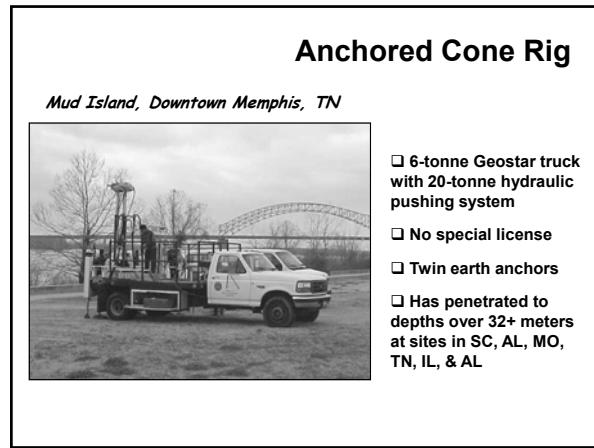
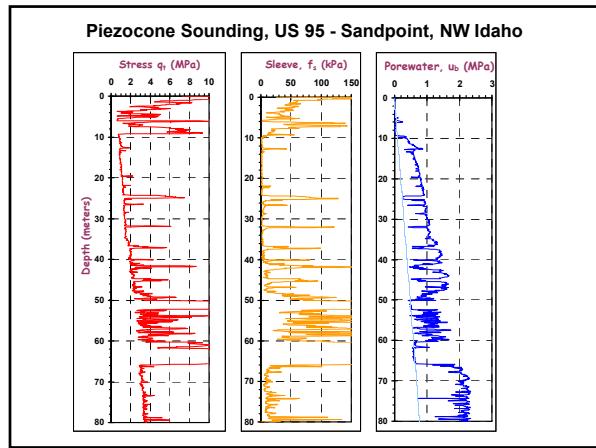
- Water
- Glycerine
- Silicone

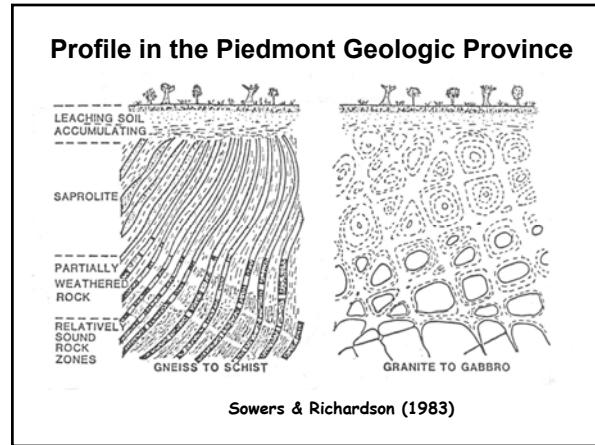
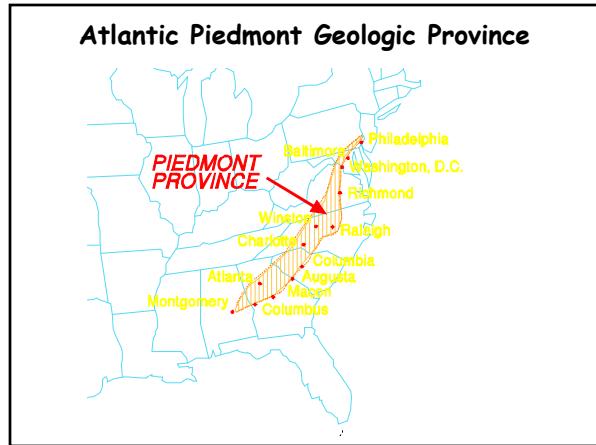
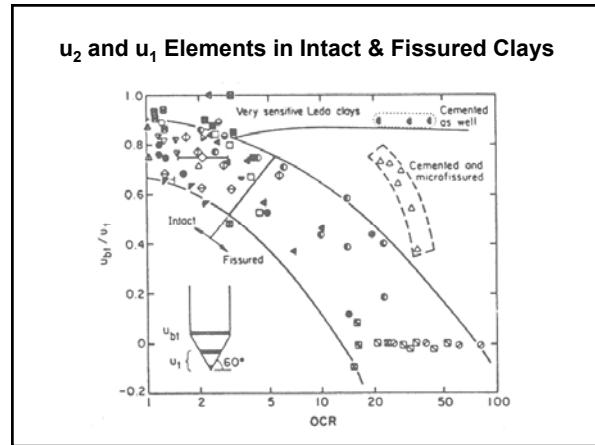
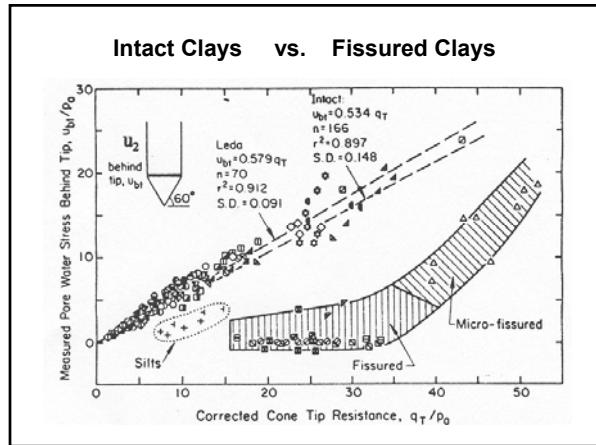
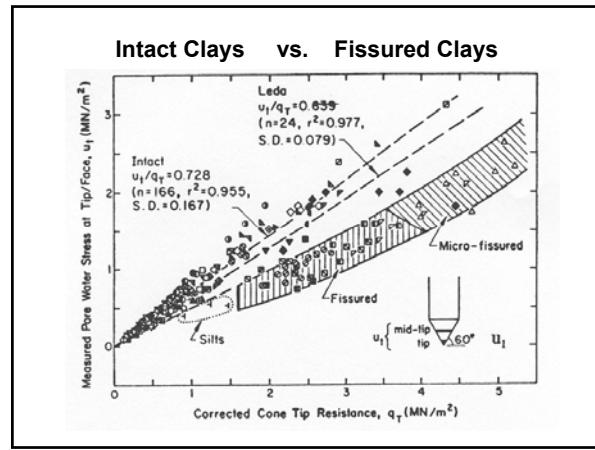
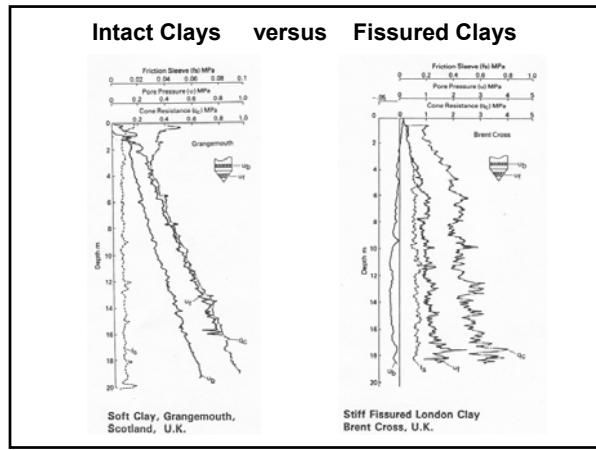
Procedures:

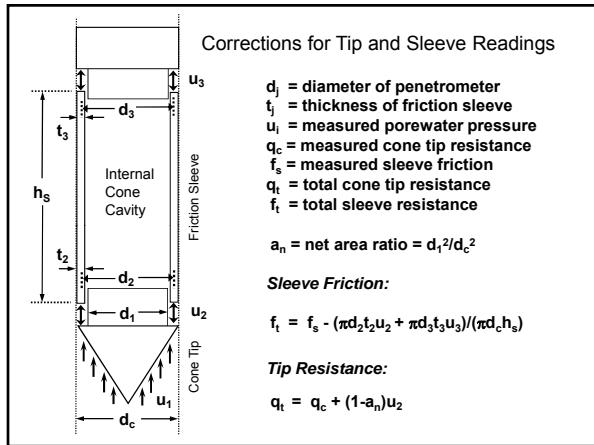
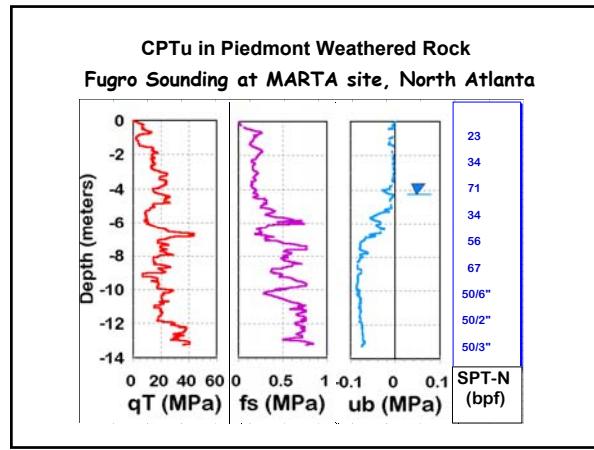
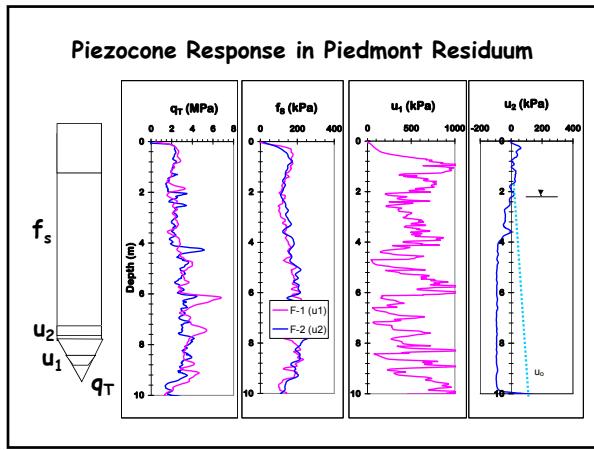
- Vacuum for 24-hours
- Pre-saturated elements
- Prophylactic to maintain fluids

Grease-Filled Slots - (no element)

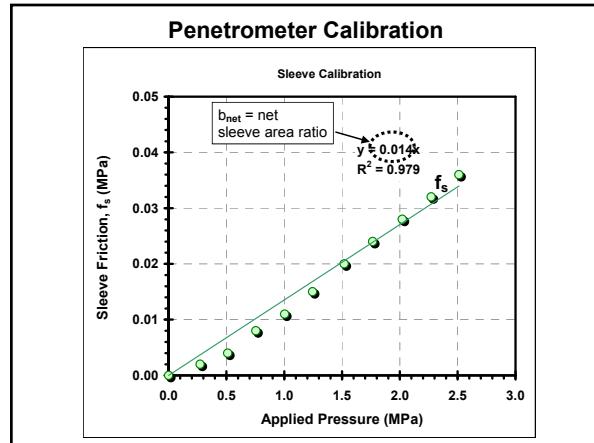
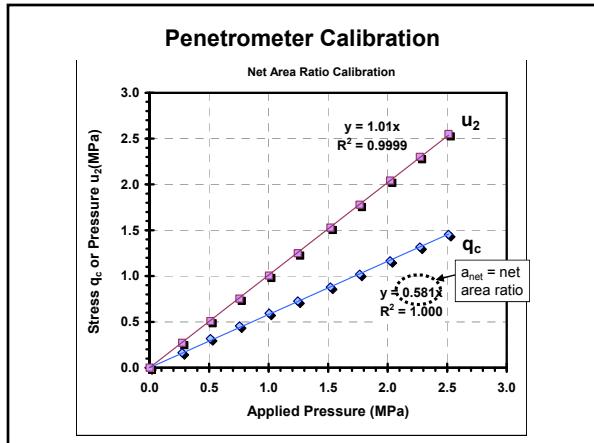








- Calibrations of Penetrometers**
- Two types of calibrations necessary.
 - Load cell verification in compression machine (proving ring) for q_c and f_s
 - Penetrometer in pressurized triaxial cell to detail porewater effects on geometry to obtain q_t and f_t
 - Pressure triaxial calibrations also verify porewater transducer output.



CPT Reading Corrections

- Total Cone Tip Resistance (required):**

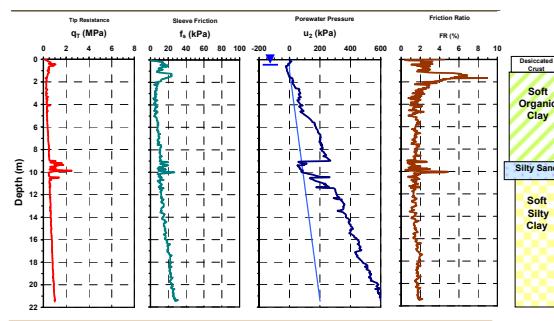
$$q_t = q_c + (1 - a_{net}) u_2$$

- Total Sleeve Resistance (maybe):**

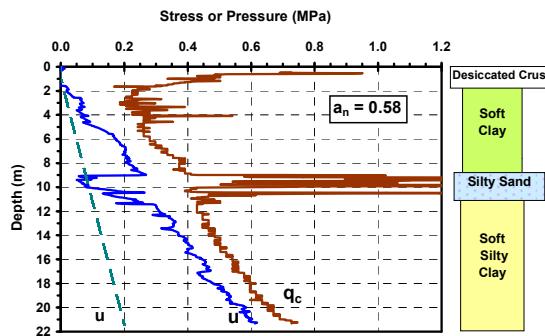
$$f_t \approx f_s - b_{net} \cdot u_2$$

$$f_t = f_s - (\pi d_2 t_2 u_2 + \pi d_3 t_3 u_3) / (\pi d_c h_s)$$

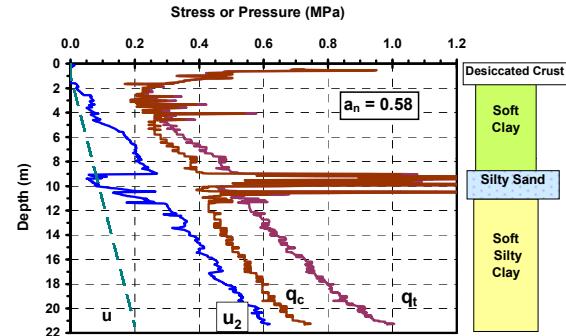
New Orleans Levees - CPTu Sounding



New Orleans Levees - CPTu Sounding



New Orleans Levees - CPTu Sounding

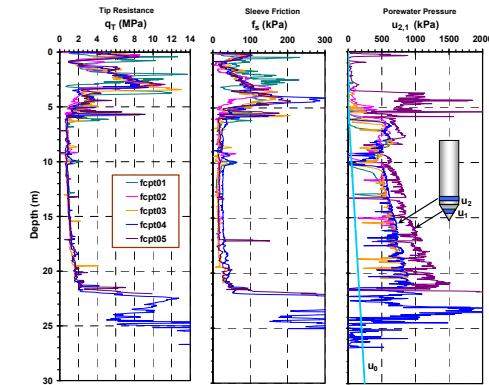


Depth Correction

- Verticality changes with depth
- Measure inclination during penetration
- Actual depth, $z = \int C_j dz = \sum (C_j \Delta z_j)$
- Single axis inclinometer: $C_j = \cos(\alpha_j)$
- Biaxial inclinometer:

$$C_j = \frac{1}{\sqrt{1 + \tan^2(\alpha_j) + \tan^2(\beta_j)}}$$

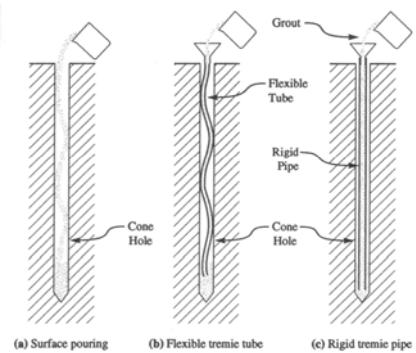
Chicago Soft Clay, Northwestern Univ



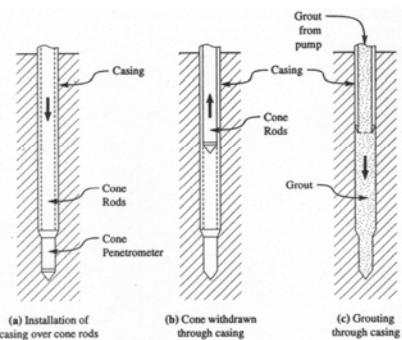
CPT Hole Closure

- Unconfined aquifer vs artesian aquifer
- Groundwater resources
- Need for hole closure (State dependent)
- Grout upon withdrawal
- Separate system for grouting
- Lutenegger et al. (NCHRP Report 378, 1995, Transportation Research Board)
- Lutenegger & DeGroot (Canadian Geot. J. Vol. 32, No. 5, 1995: 880-891)

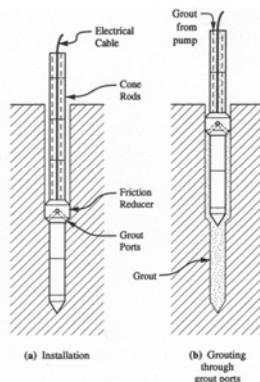
Lutenegger & DeGroot 1995



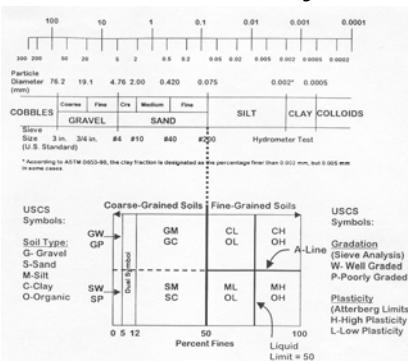
Lutenegger & DeGroot 1995



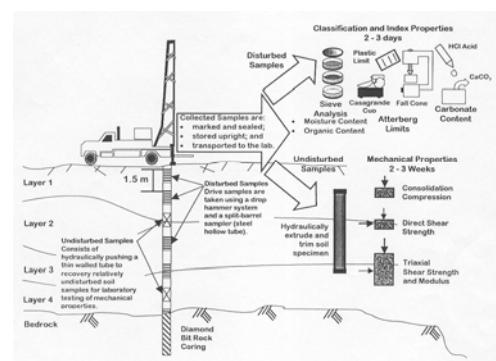
Lutenegger & DeGroot 1995



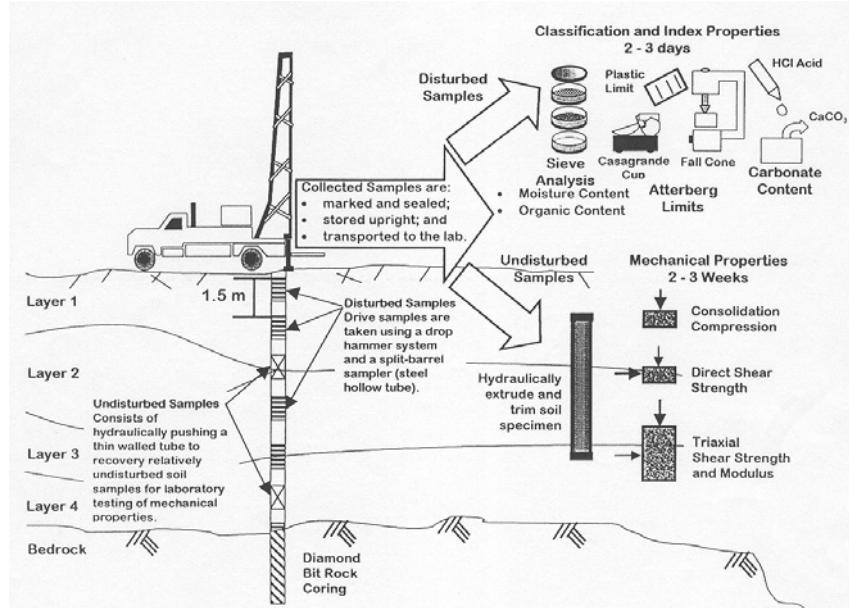
Soil Classification by CPT



Soil Classification by Drilling & Sampling

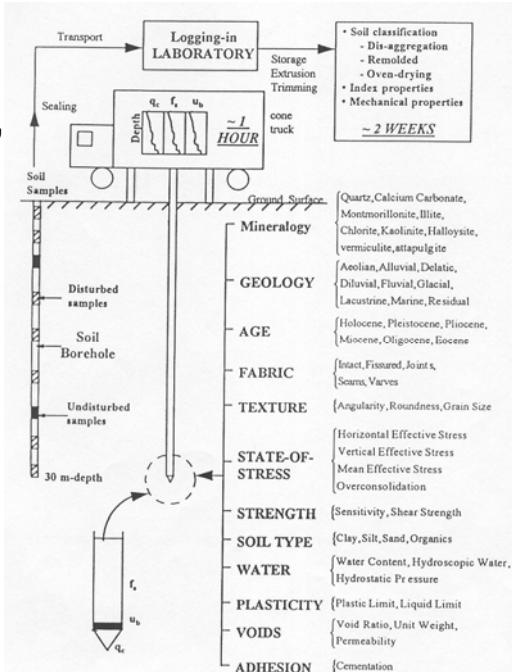


Soil Classification by Drilling & Sampling



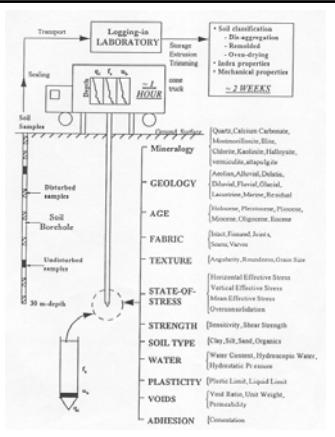
Factors Affecting CPT Measurements, including Soil Classification

(Hegazy, PhD 1998)

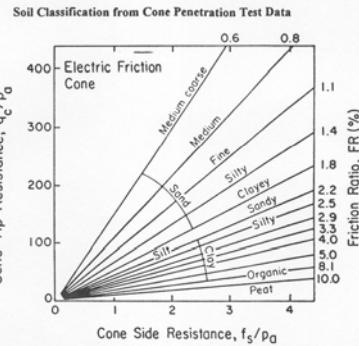


Factors Affecting CPT Measurements, including Soil Classification

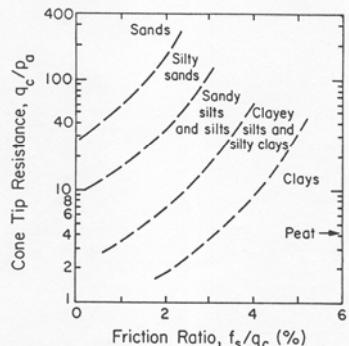
(Hegazy, 1998)



CPT Soil Behavioral Classification



CPT Soil Behavioral Classification

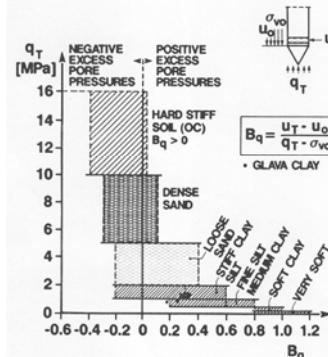


CPT Soil Behavioral Classification

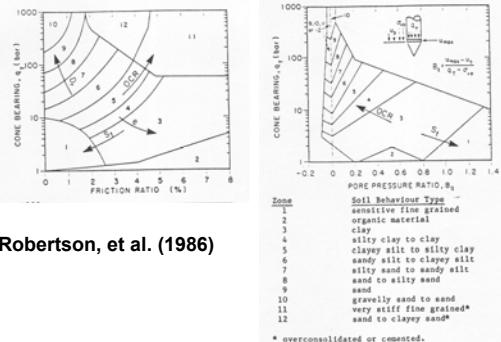
Norwegian Institute of Technology (NTH)

Norwegian University of Science & Tech (NTNU)

Senneset, et al. (1988, ISOPT-Orlando; 1989 Transportation Res. Record)



CPT Soil Behavioral Classification



Robertson, et al. (1986)

Normalized Piezocone Parameters

□ Normalized Tip Resistance:

$$Q = (q_T - \sigma_{vo}) / \sigma_{vo}$$

□ Normalized Sleeve Friction:

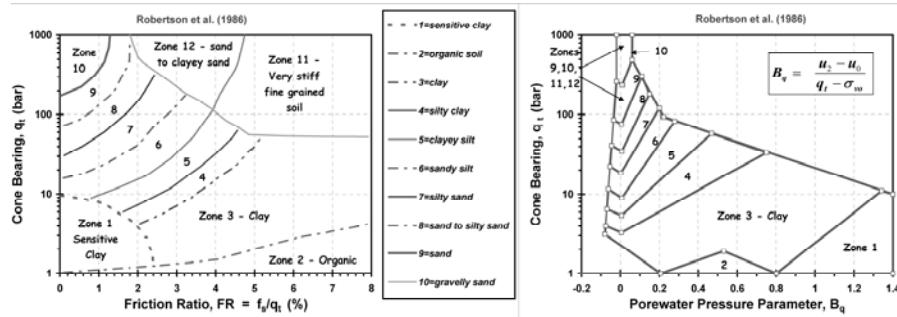
$$F = f_s / (q_T - \sigma_{vo})$$

□ Normalized Excess Porewater Pressure Reading (shoulder):

$$B_q = (u_2 - u_0) / (q_T - \sigma_{vo})$$

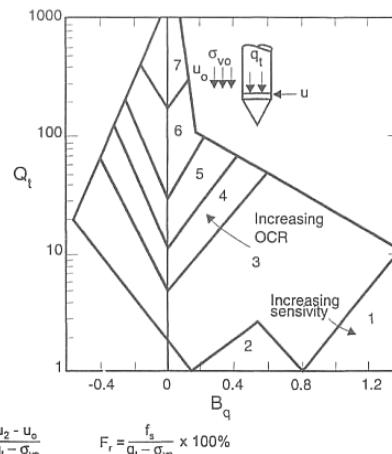
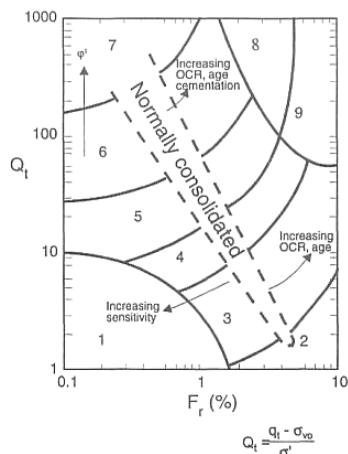
CPT Soil Behavioral Classification

Robertson, et al. (1986) Use of In-Situ Tests in Geot. Engrg, ASCE GSP 6



CPT Soil Behavioral Classification

Robertson (CGJ 1990 and 1991 closure)



Zone	Soil behaviour type
1.	Sensitive, fine grained;
2.	Organic soils-peats;
3.	Clays-clay to silty clay;

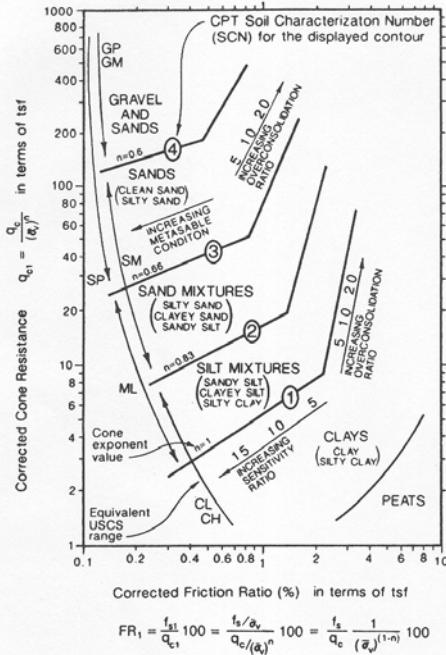
Zone	Soil behaviour type
4.	Silt mixtures clayey silt to silty clay
5.	Sand mixtures; silty sand to sand silty
6.	Sands; clean sands to silty sands

Zone	Soil behaviour type
7.	Gravelly sand to sand;
8.	Very stiff sand to clayey sand
9.	Very stiff fine grained

CPT Soil Behavioral Classification

Olsen & Malone (Penetration Testing 1988, Orlando)

Olsen & Mitchell (1995, Intl. Symposium on Cone Penetration Testing, Swedish Geotechnical Society)



Soil Behavioral Type from Piezocene

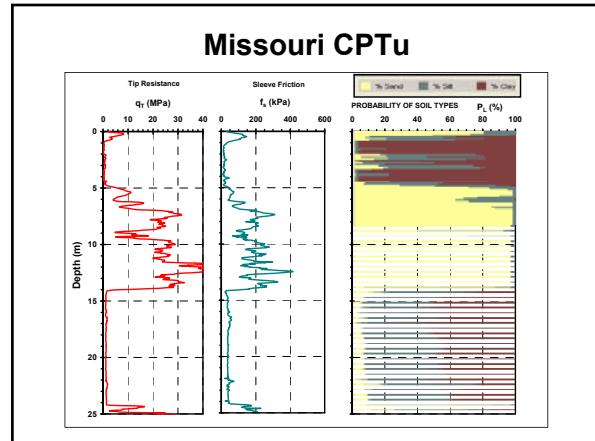
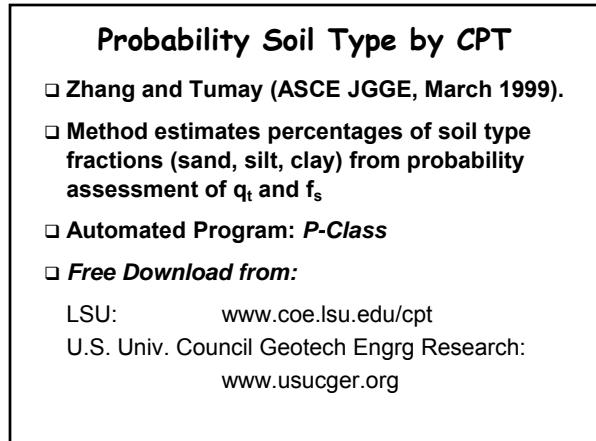
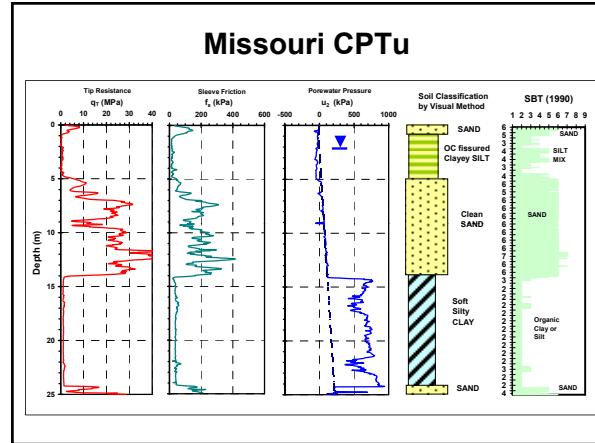
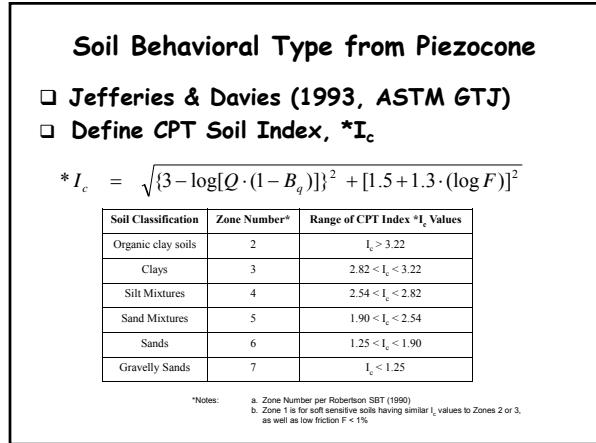
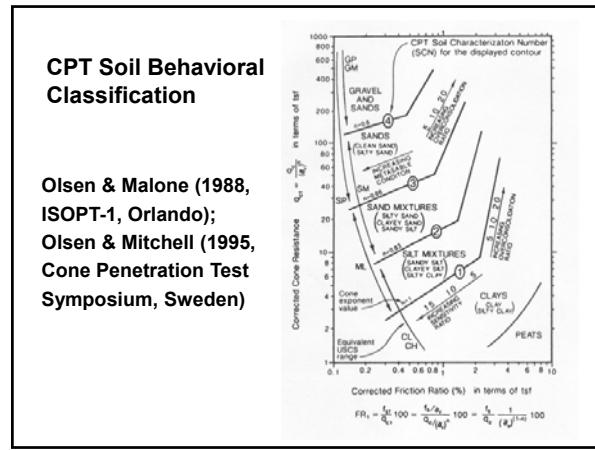
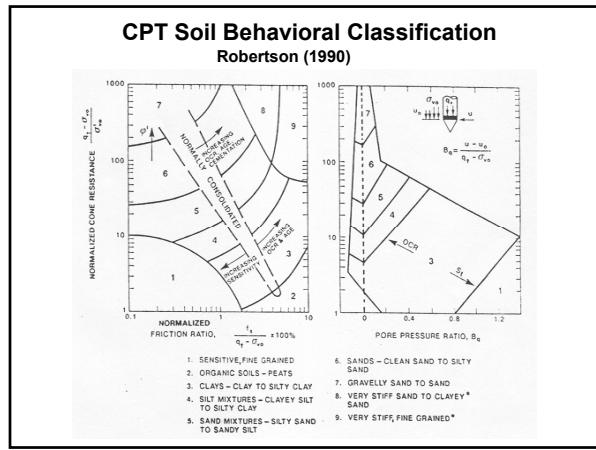
- Jefferies & Davies (1993, ASTM GTJ)
- Define CPT Soil Index, * I_c

$$*I_c = \sqrt{\{3 - \log[Q \cdot (1 - B_q)]\}^2 + [1.5 + 1.3 \cdot (\log F)]^2}$$

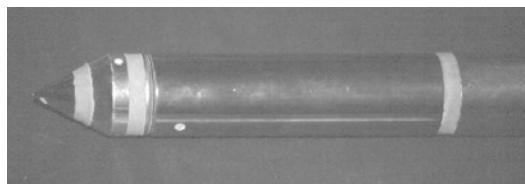
Soil Classification	Zone Number*	Range of CPT Index * I_c Values
Organic clay soils	2	$I_c > 3.22$
Clays	3	$2.82 < I_c < 3.22$
Silt Mixtures	4	$2.54 < I_c < 2.82$
Sand Mixtures	5	$1.90 < I_c < 2.54$
Sands	6	$1.25 < I_c < 1.90$
Gravelly Sands	7	$I_c < 1.25$

*Notes:

- a. Zone Number per Robertson SBT (1990)
- b. Zone 1 is for soft sensitive soils having similar I_c values to Zones 2 or 3, as well as low friction $F < 1\%$

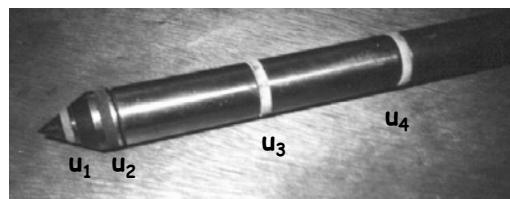


Piezocene Penetrometers



Fugro Triple-Element Piezocene
Used in North Sea Investigations
(Norwegian Institute of Technology)

Multi-Element Piezocene Penetrometers



Quad-Element Piezocene (Oxford University)

Multi-Friction Sleeved Penetrometer (Frost & DeJong, 2001)



Memocone (cableless system) - Envi AB



Memory chip in penetrometer. Synchronize with depth wheel & data logger at surface

Audio- Cone (cableless system)- Geotech AB



Uses audio signal to send CPTu data up
the inside of the rods. Decoder at surface

Slot "Filter" for Porewater Pressures

- Elmgren, K. (1995). Slot-type pore pressure CPT-u filters. *Proceedings, International Symposium on Cone Penetration Testing (CPT'95)*, Vol. 2, Linköping, Sweden, 9-12.
- Larsson, R. (1995). Use of thin slot as filter in piezocene tests. *Proceedings, International Symposium on Cone Penetration Testing (CPT'95)*, Vol. 2, Linköping, Sweden, 35-40.

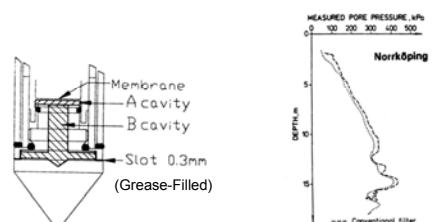
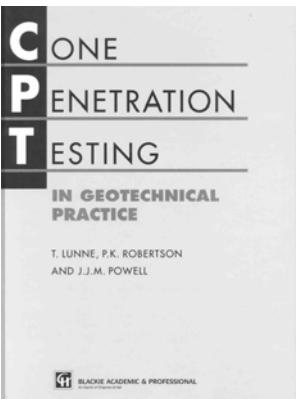


Fig. C-1. Slot filter for Memocone.
(Elmgren, 1995)

Fig. C-2. Comparative filter results.
(Larsson, 1995)



CPT Related Websites

- ❑ ISSMGE TC 16 (In-Situ Testing):
www.geoforum.com/tc16
- ❑ US Univ. Council on Geot. Engrg Research:
<http://www.usucqer.org/>
- ❑ GT In-Situ Research Group:
<http://www.ce.gatech.edu/~geosys>
- ❑ The book: *CPT in Geotechnical Practice*
(Lunne, Robertson, & Powell, 1997):
<http://www.routledge.com/>

Piezocene Penetrometers



Various Penetrometers Used at Georgia Tech

GEOPHYSICAL SITE CHARACTERIZATION

- Mechanical Wave Measurements
- Electromagnetic Wave Techniques

Geophysical Methods

□ Mechanical Wave Measurements

- Crosshole Tests (CHT)
- Downhole Tests (DHT)
- Spectral Analysis of Surface Waves
- Seismic Refraction
- Suspension Logging

□ Electromagnetic Wave Techniques

- Ground Penetrating Radar (GPR)
- Electromagnetic Conductivity (EM)
- Surface Resistivity (SR)
- Magnetometer Surveys (MT)

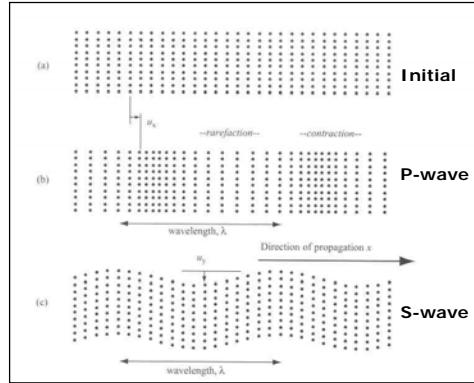
Mechanical Wave Geophysics

- Nondestructive measurements ($\gamma_s < 10^{-4}\%$)
- Both borehole geophysics and non-invasive types (conducted across surface).
- Measurements of wave dispersion: velocity, frequency, amplitude, attenuation.
- Determine layering, elastic properties, stiffness, damping, and inclusions
- Four basic wave types: Compression (P), Shear (S), Rayleigh (R), and Love (L).

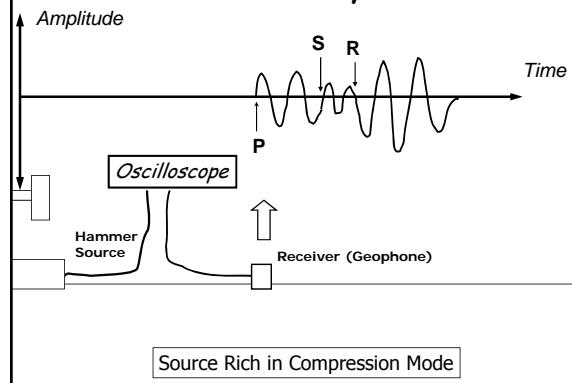
Mechanical Wave Geophysics

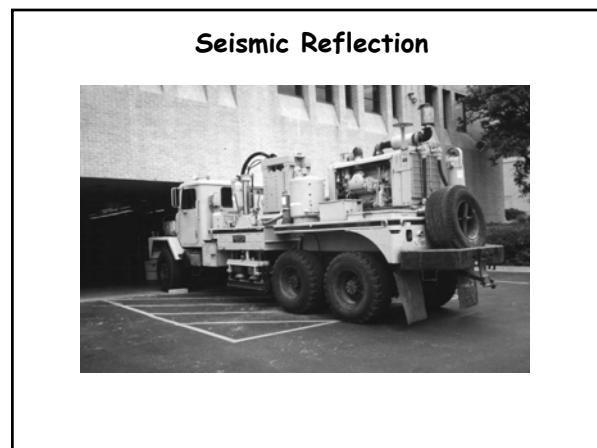
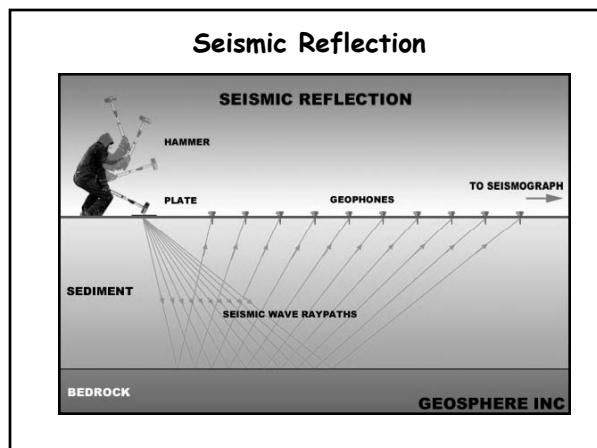
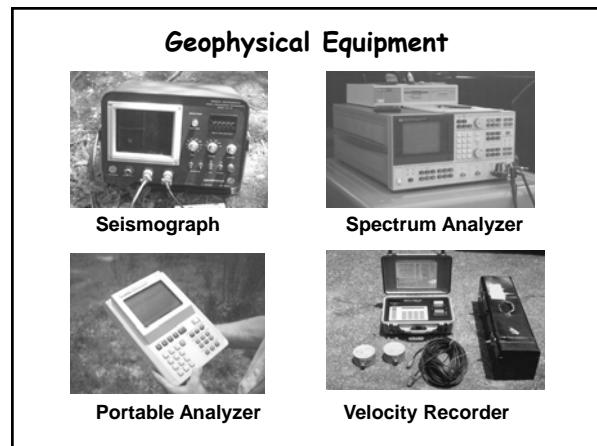
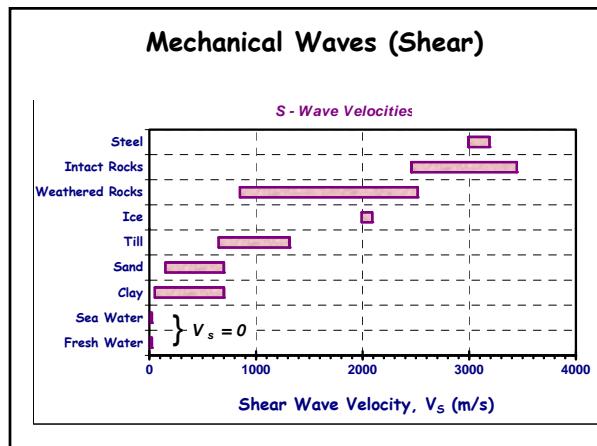
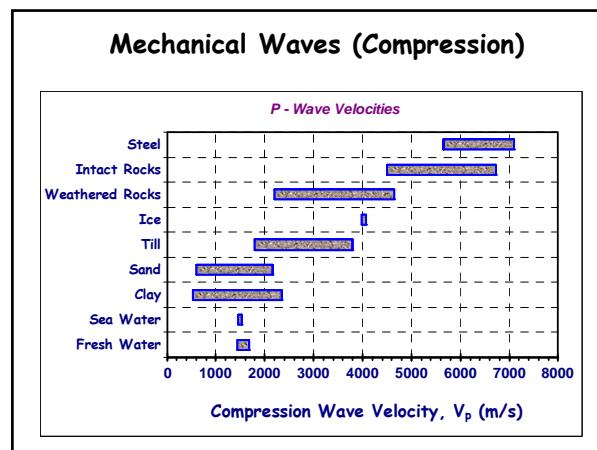
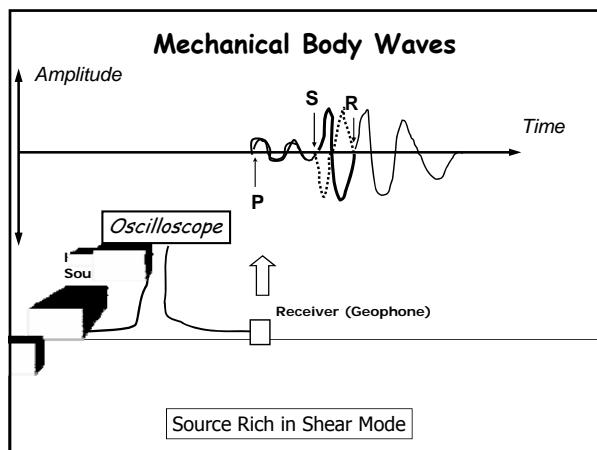
- Compression (P-) wave is fastest wave; easy to generate.
- Shear (S-) wave is second fastest wave. Is directional and polarized. Most fundamental wave to geotechnique.
- Rayleigh (R-) or surface wave is very close to S-wave velocity (90 to 94%). Hybrid P-S wave at ground surface boundary.
- Love (L-) wave: interface boundary effect

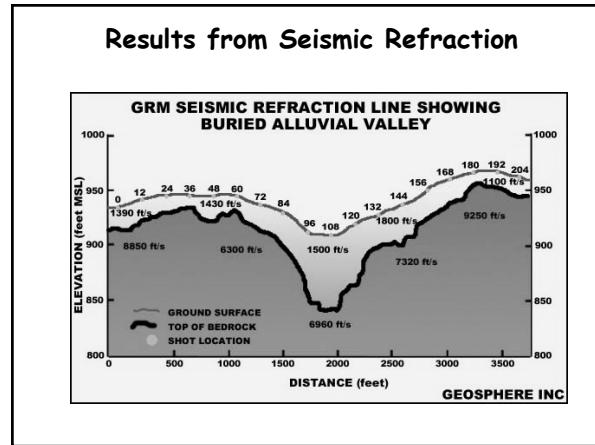
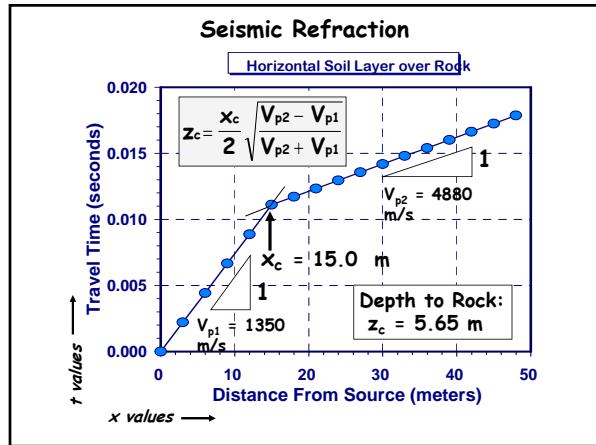
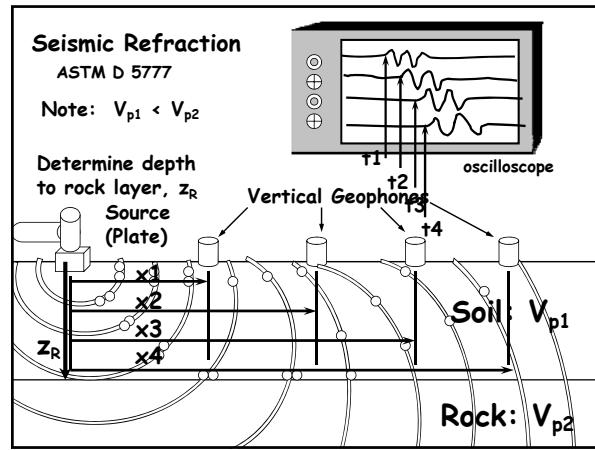
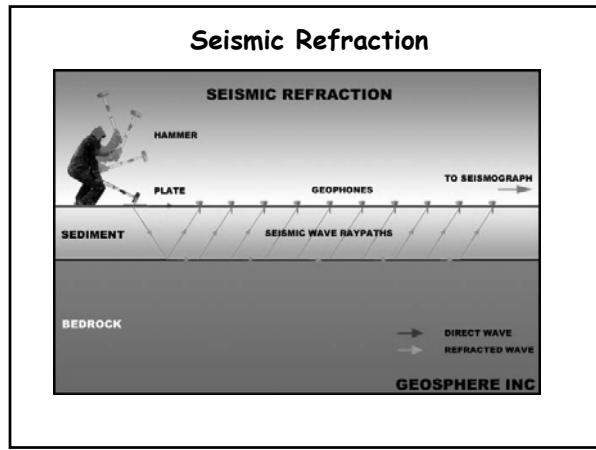
Mechanical Body Waves



Mechanical Body Waves

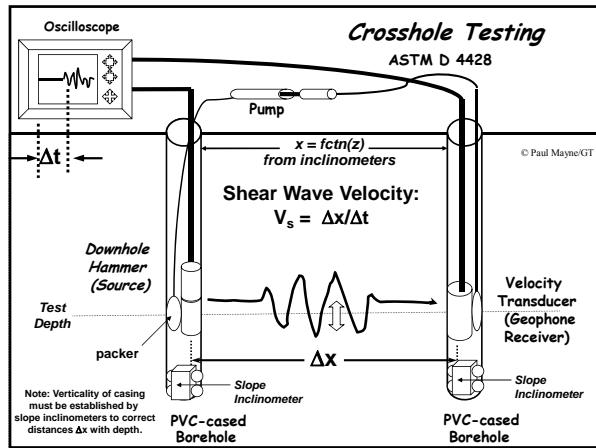






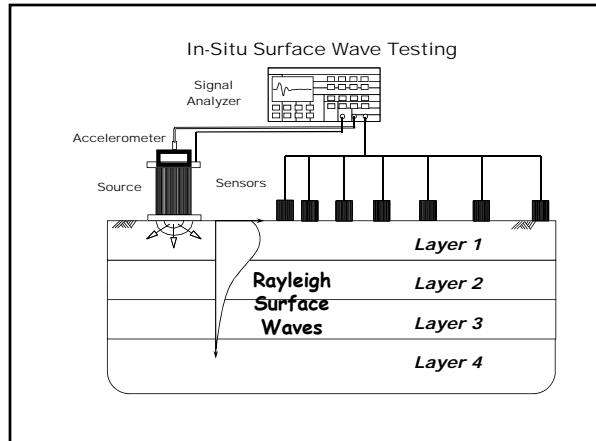
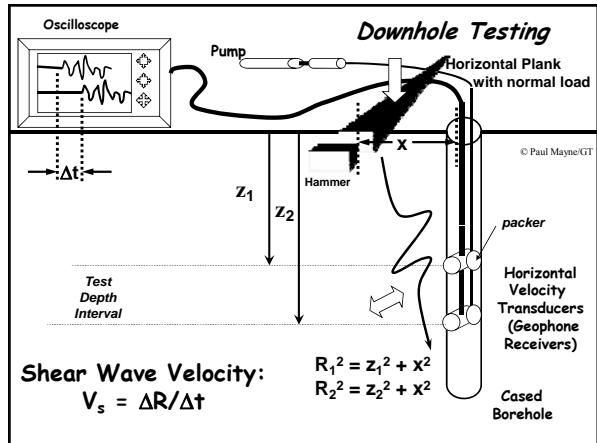
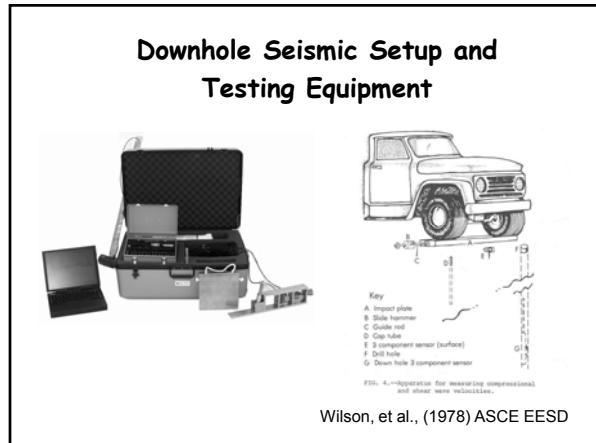
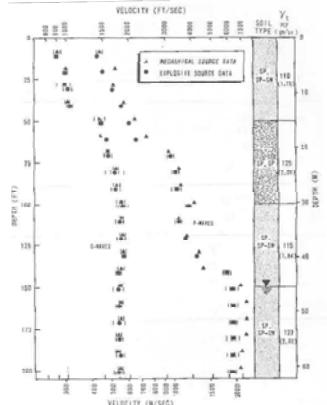
- Shear Wave Velocity, V_s**
- Fundamental measurement in all solids (steel, concrete, wood, soils, rocks)
 - Initial small-strain stiffness represented by shear modulus: $G_0 = \rho_T V_s^2$ (alias $G_{dyn} = G_{max} = G_0$)
 - Applies to all static & dynamic problems at small strains ($\gamma_s < 10^{-6}$)
 - Applicable to both undrained & drained loading cases in geotechnical engineering.





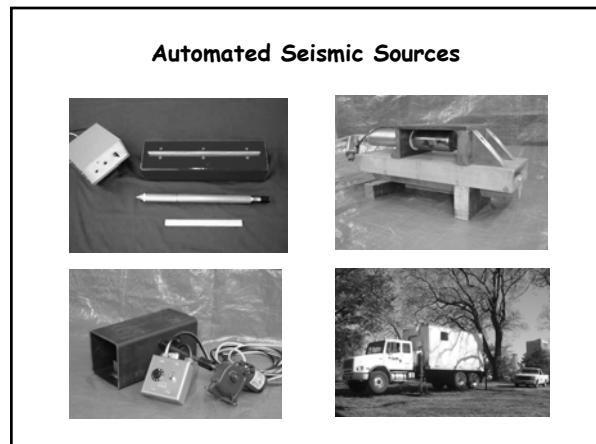
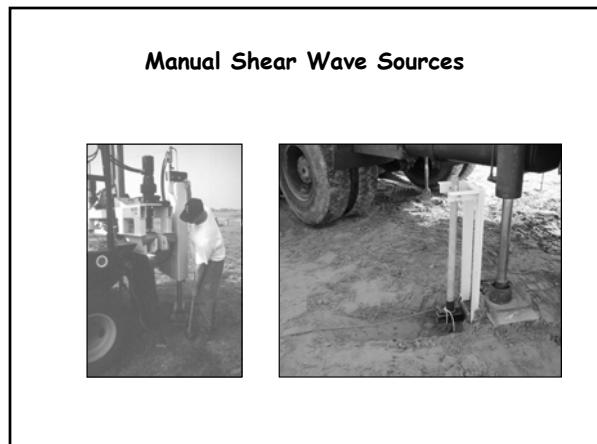
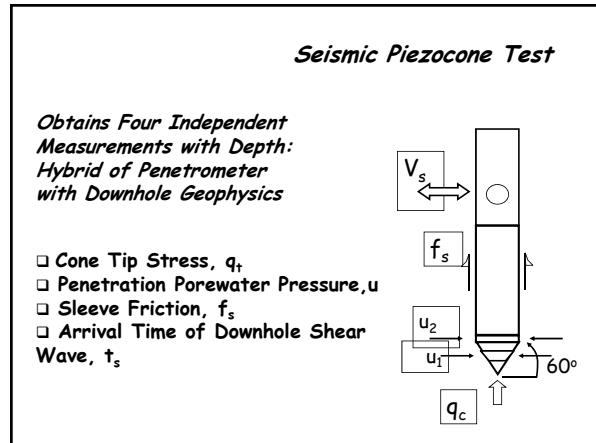
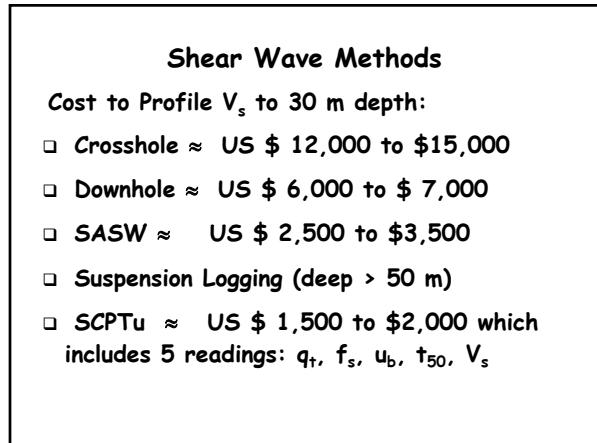
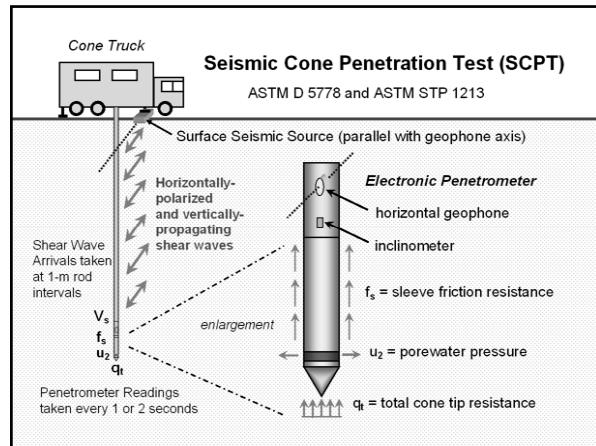
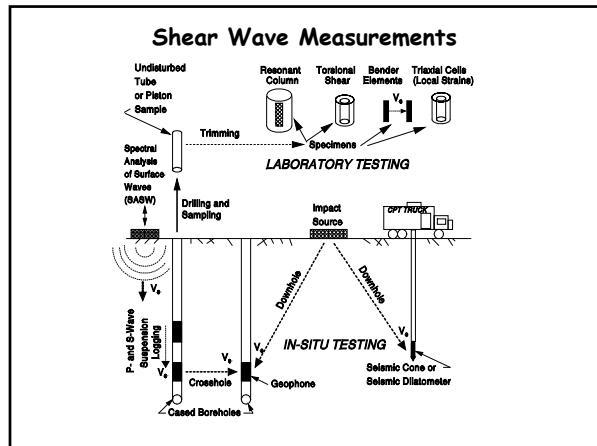
Results from Crosshole Seismic Tests

Reference: McLamore, Anderson, & Espana (1978), ASTM STP 654

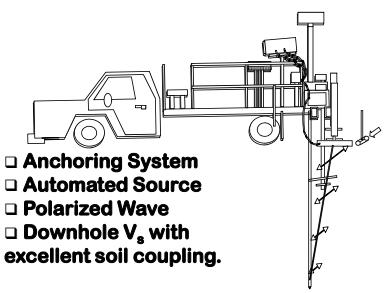


Surface Wave Measurements

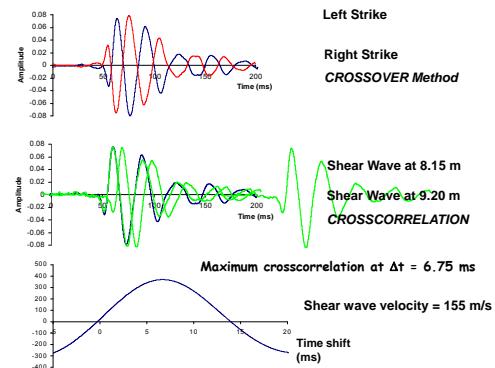
- Spectral Analysis of Surface Waves SASW (transient)
- Continuous Surface Waves (CSW): variable excitation using surface vibrator
- Modal Analysis of Surface Waves (MASW)
- Passive Analysis of Surface Waves (low frequency content)



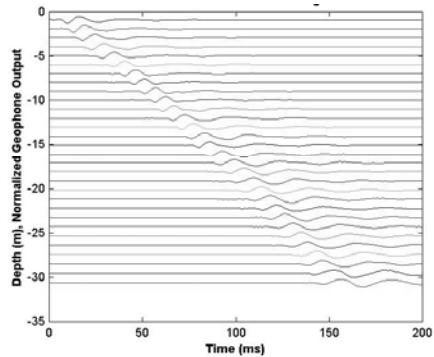
Downhole Shear Wave Velocity



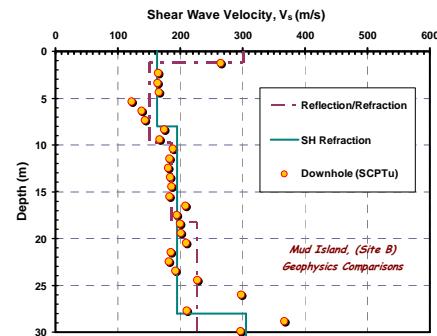
Downhole Shear Waves



Complete Set of Shear Wave Trains Mud Island Site A, Memphis TN



Comparison of Shear Wave Methods



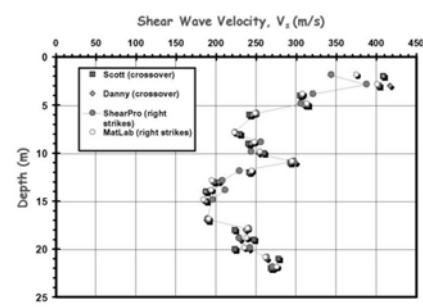
AutoSeis

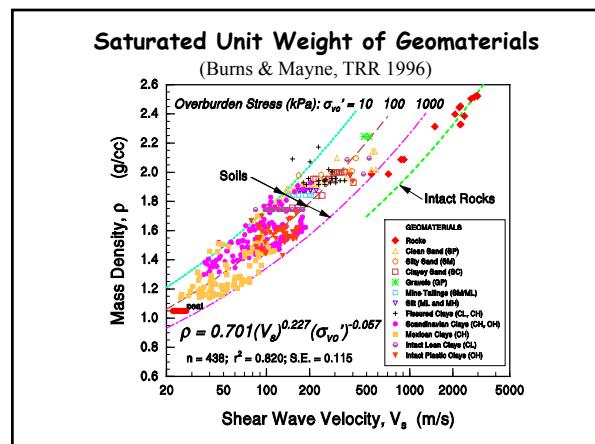
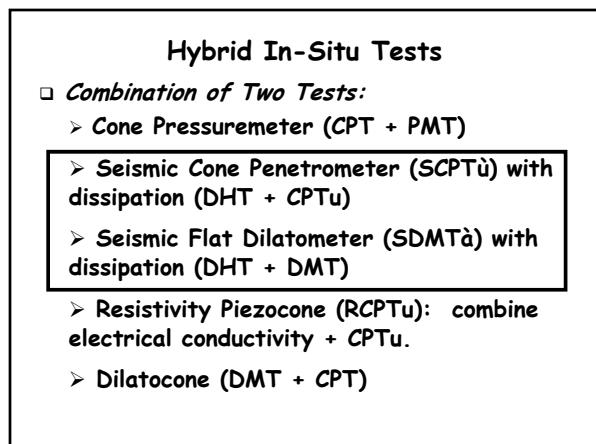
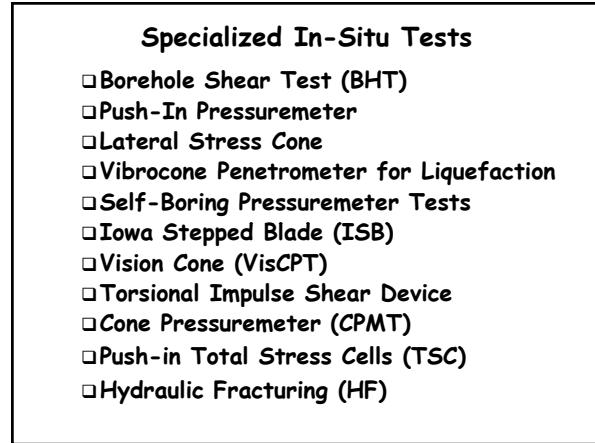
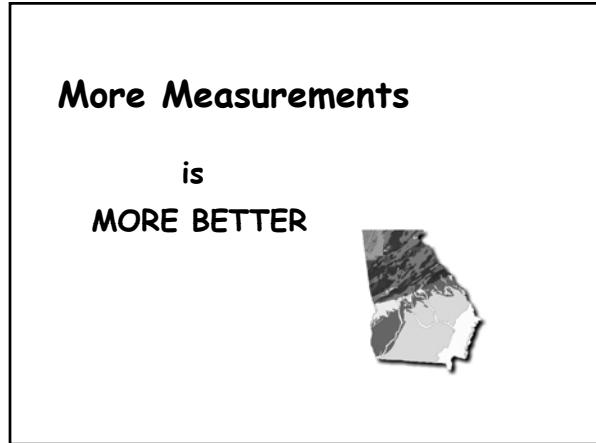
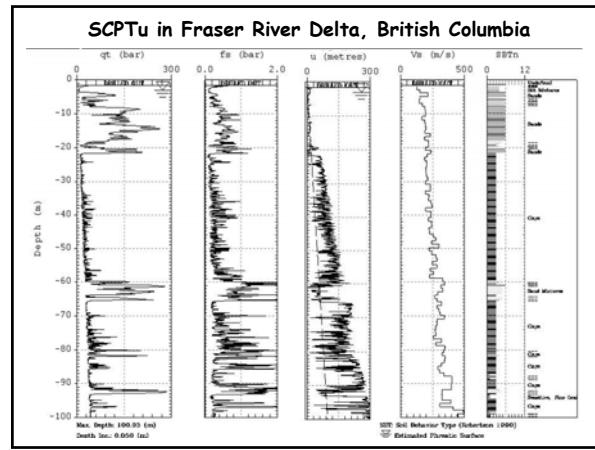
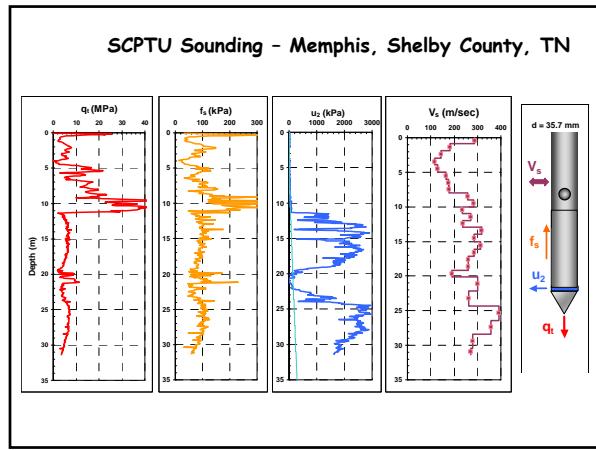
- Electro-Mechanical off 12-volt
- Repeatable, Portable, 30-m depths
- Finite Precision: www.finiteprecision.com



Crosscorrelation of Wavelets

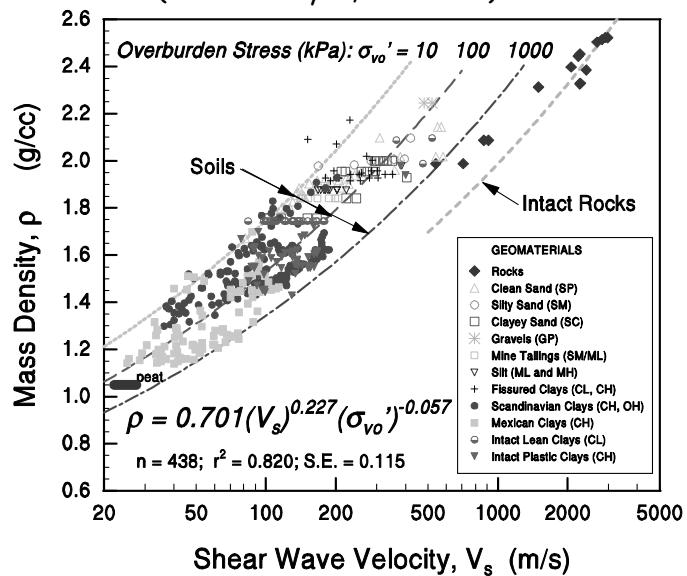
- Freeware: ShearPro 1.4
- Download: www.ce.gatech.edu/~geosys





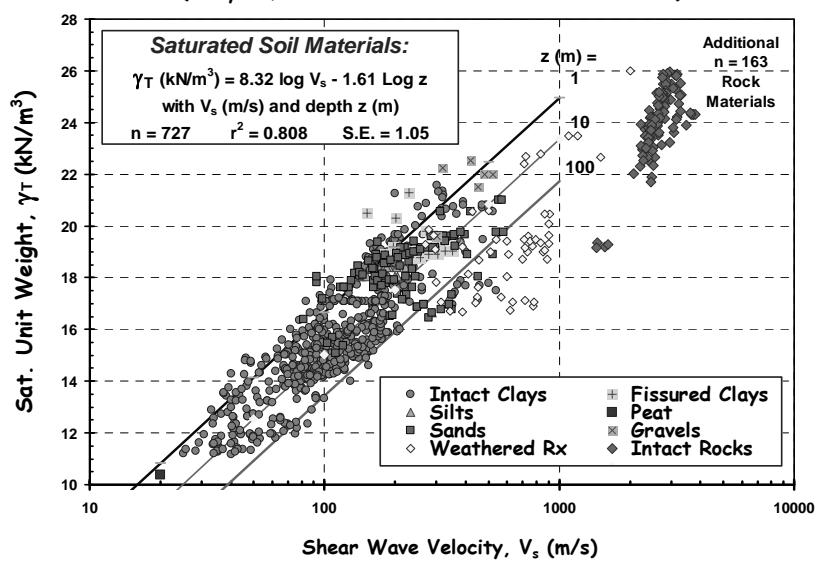
Saturated Unit Weight of Geomaterials

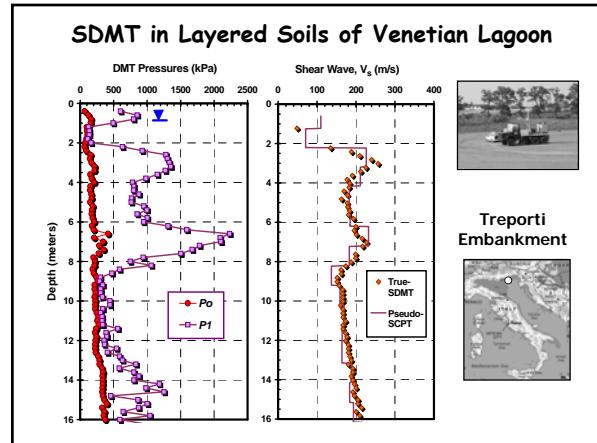
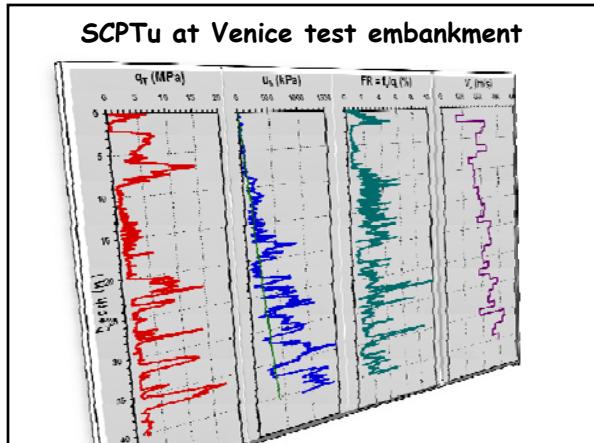
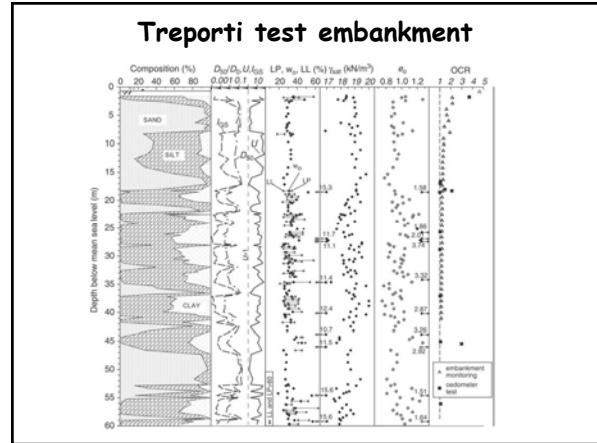
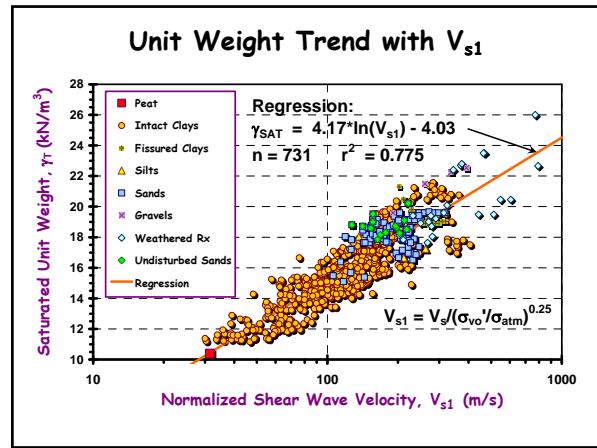
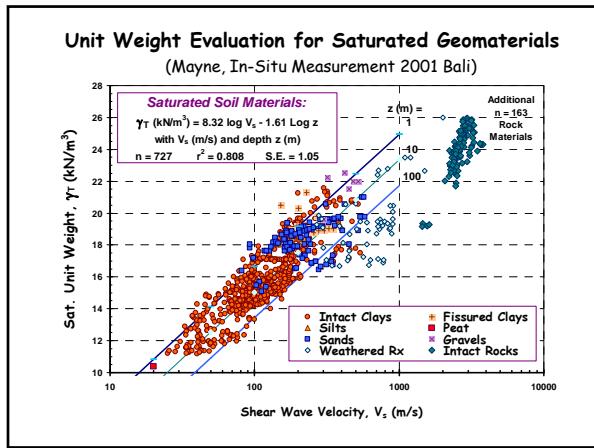
(Burns & Mayne, TRR 1996)

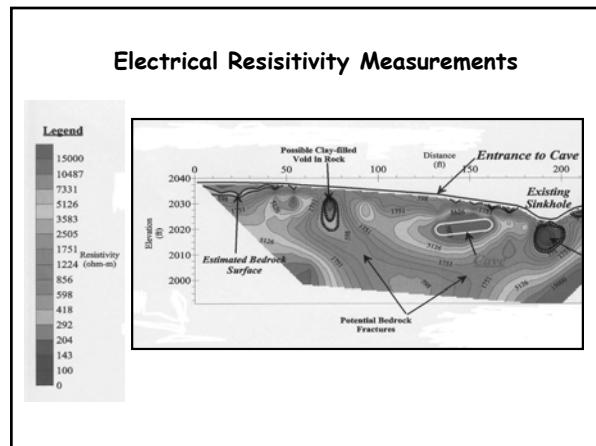
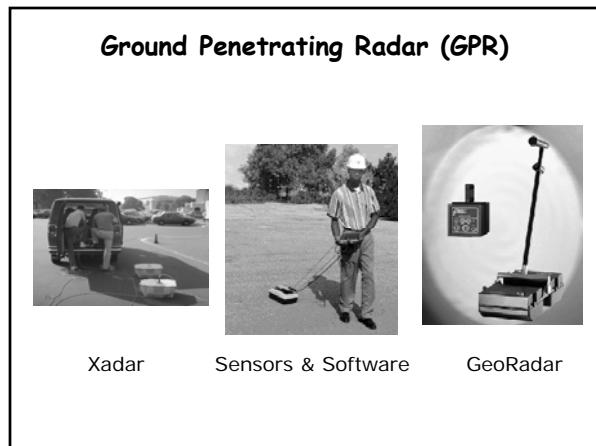
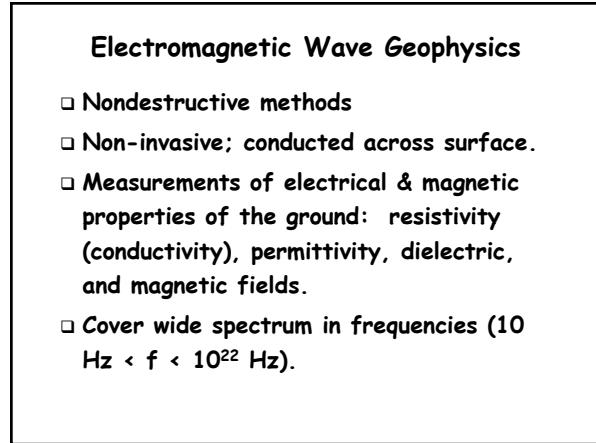
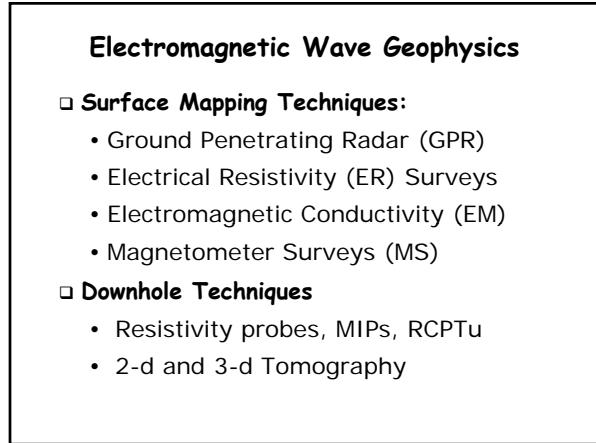
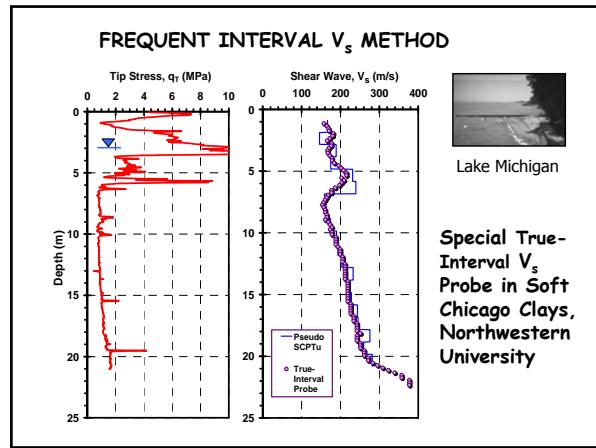
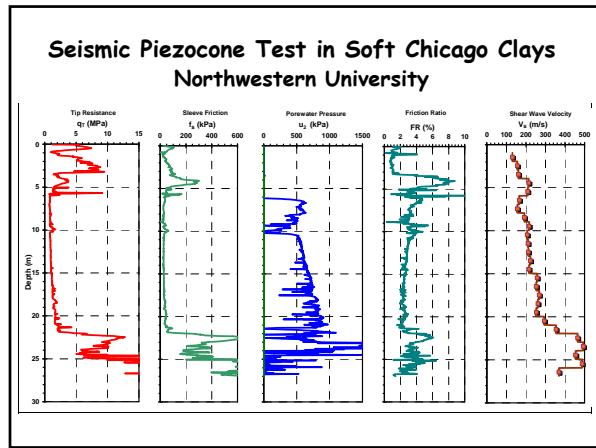


Unit Weight Evaluation for Saturated Geomaterials

(Mayne, In-Situ Measurement 2001 Bali)







Electrical Resistivity Measurements

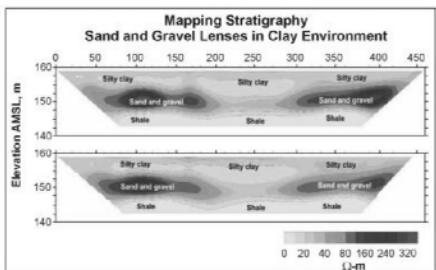


Figure 109. Resistivity data plotted to form a depth vs resistivity section.
(Advanced Geosciences, Inc.)

Electromagnetic Conductivity (EM)



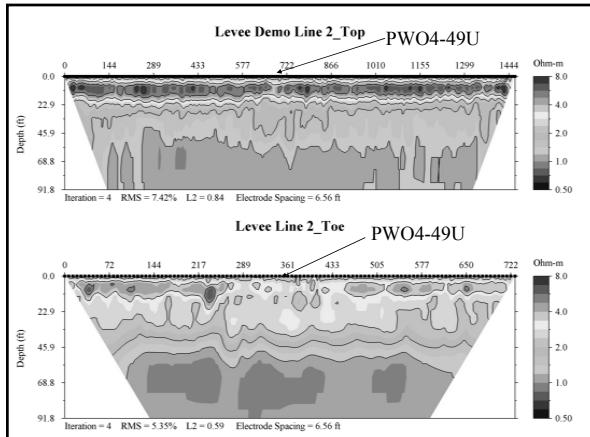
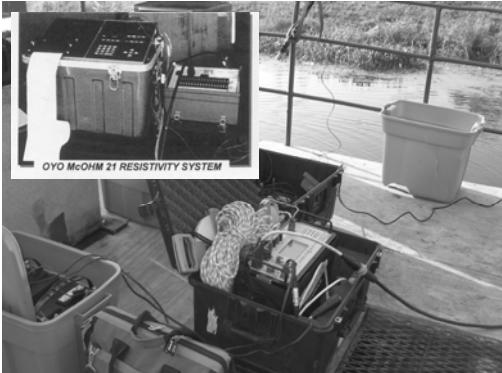
Figure 106. Dipping clay-shale layer.



Figure 107. Clay in alluvium.

New Orleans Levees

- Embankment levees constructed on soft clays
- Presence of very soft peats and organic clays
- Stability problems
- Settlement and long-term creep



References on Geophysics

- Application of Geophysical Methods to Highway Related Problems (FHWA Manual DTFH68-02-P-00083; 2003)
- Soils and Waves by Santamarina, Klein, and Fam (2001, Wiley & Sons)
- ISSMGE TC 10 - Geophysics in Geotechnical Engineering:
 - www.geoforum.com/tc10

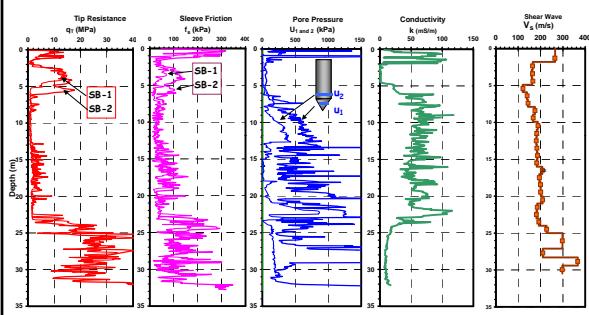


RCPTu

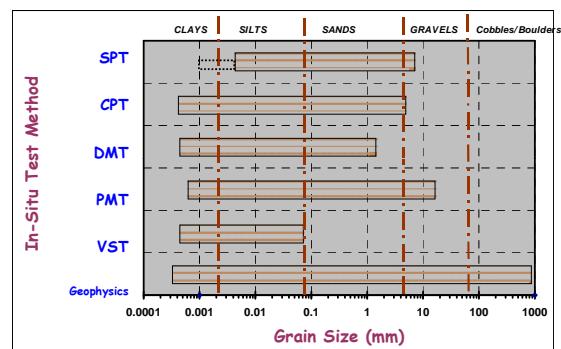
- Resistivity (or Conductivity) Penetrometers
- Dielectric (or Permittivity) Penetrometers

Seismic Resistivity Soundings (SRCPTu)

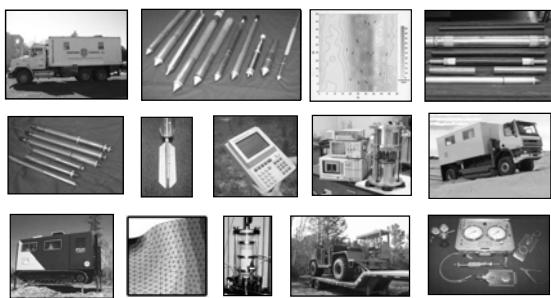
Combined RCPTu1 and SCPTu2 at Mud Island, Memphis



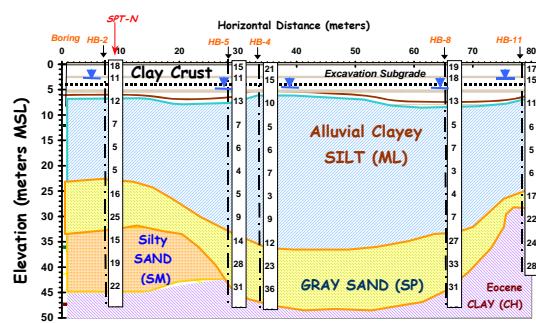
Applicability of In-Situ Tests

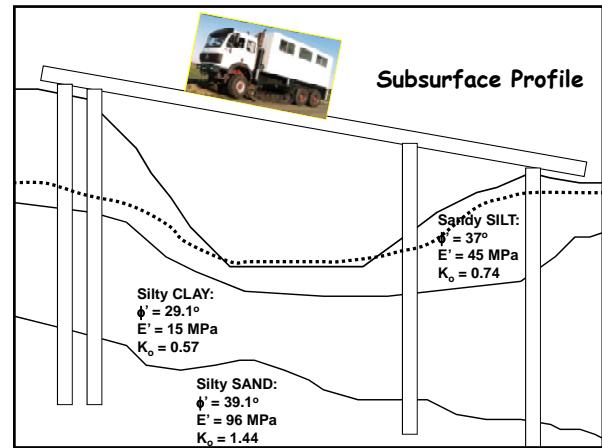
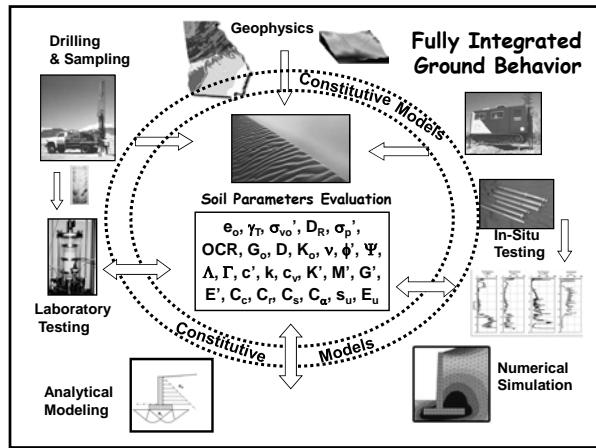


Geotechnical Site Characterization



Subsurface Profile Developed from Geotechnical Data





Relative Density of Clean Sands

- Unit Weight
- Mass Density
- Void Ratio
- Relative Density



Pila Dune Sand
Archechon, France

Paul W. Mayne, PhD, P.E.

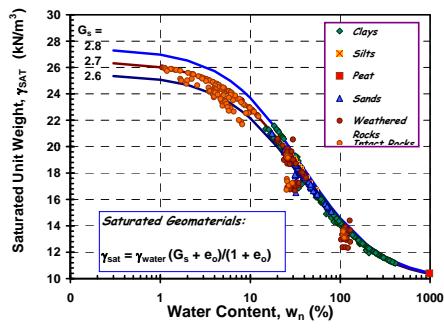
Georgia Tech

Mass Density and Unit Weight

- Total Unit Weight: $\gamma_T = \rho_T g_a$
- Soil Identities:
 - $G_s w_n = S e$
 - $\gamma_T = G_s \gamma_w (1 + w_n)/(1 + e)$
- where G_s = specific gravity, w_n = water content, S = degree of saturation, e = void ratio, γ_w = unit weight of water; g_a = grav. Constant (9.8 m/s^2)

Georgia Tech

Unit Weight of Geomaterials



Georgia Tech

Relative Density of Sands

- Relative Density (D_R) is measure of packing of clean sands with < 16% fines (SP, SW, SP-SM, SP-SC); and gravels

$$D_R = \frac{e_{\max} - e_0}{e_{\max} - e_{\min}}$$

Where e_{\max} (loosest state), e_{\min} (densest state), and e_0 (current state).

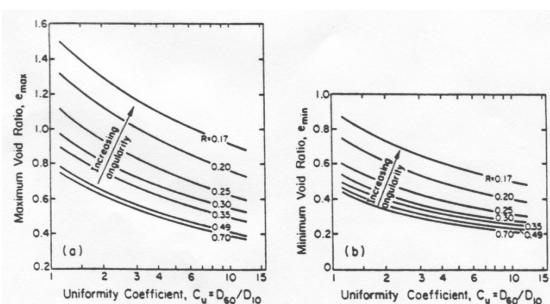
Georgia Tech

Relative Density Criteria for Sands (after Holtz, W.G., 1973, ASTM STP 523)

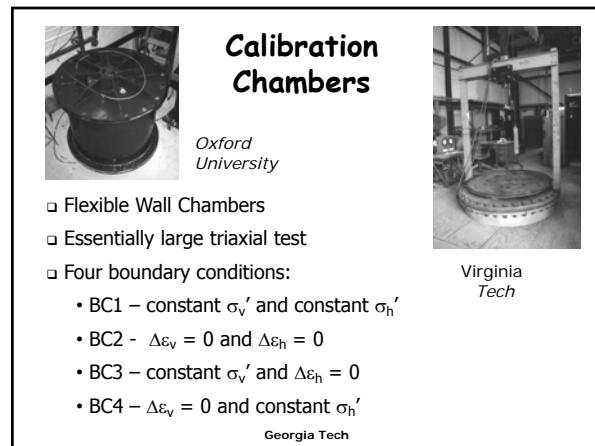
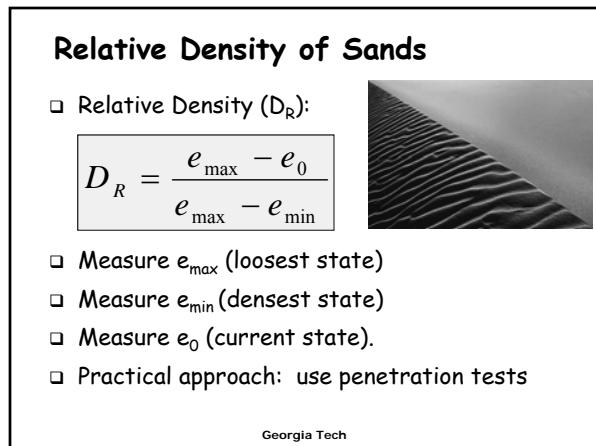
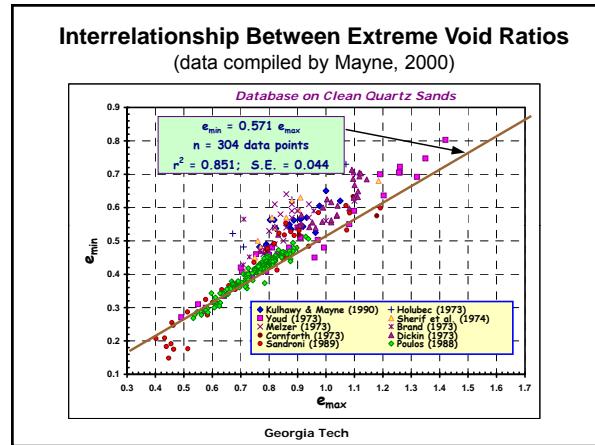
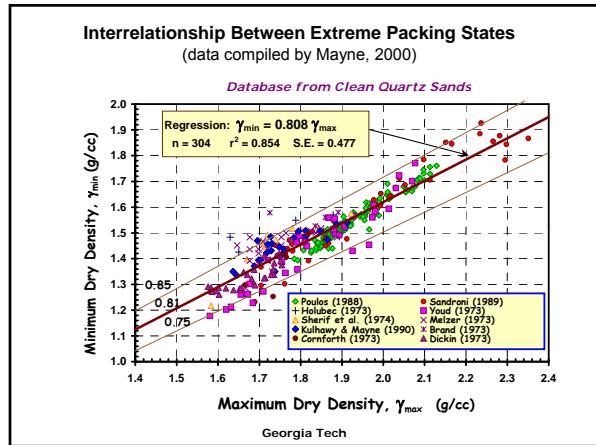
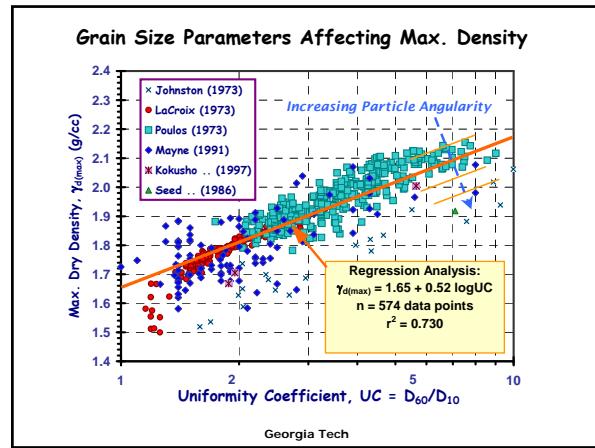
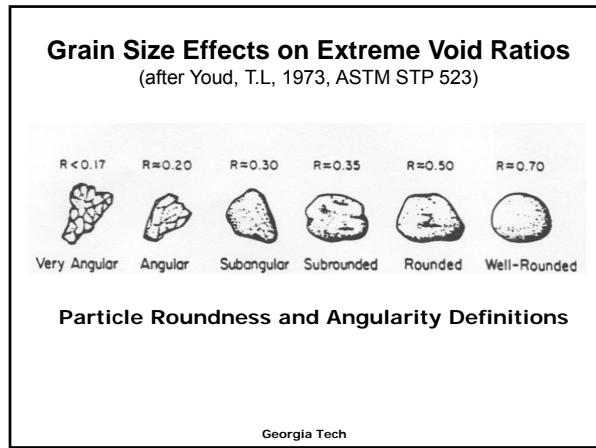
TABLE 1—Various terms used to describe the state of denseness.											
Relative Density, percent	0	10	20	30	40	50	60	70	80	90	100
USBR [6]	very loose	loose	medium		dense		very dense				
Lambe and Whitman [7]											
Burmister [8]	loose		medium		compact		very compact				
Meyerhoff [8]	very loose	loose	compact	dense	very dense						
Hough [9]	loose		firm		compact		very compact				
Tachebotaroff [10]	loose		medium		dense						
Plummer and Dore [11]											

Georgia Tech

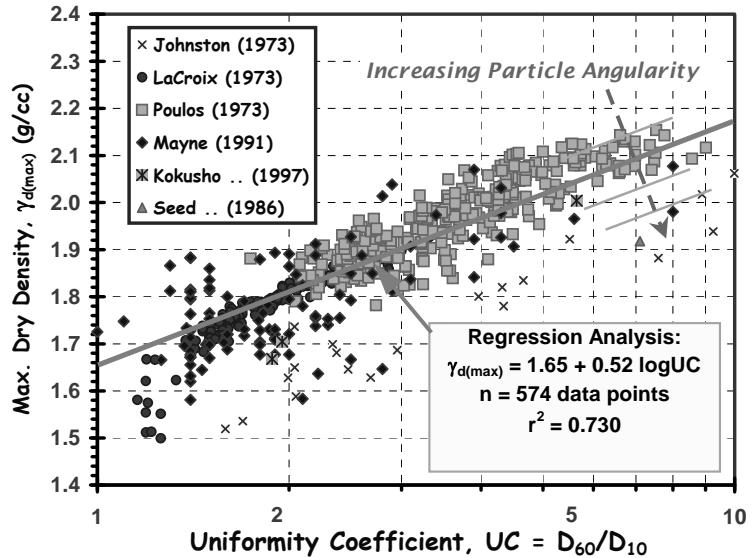
Grain Size Effects on Extreme Void Ratios (after Youd, T.L., 1973, ASTM STP 523)



Georgia Tech

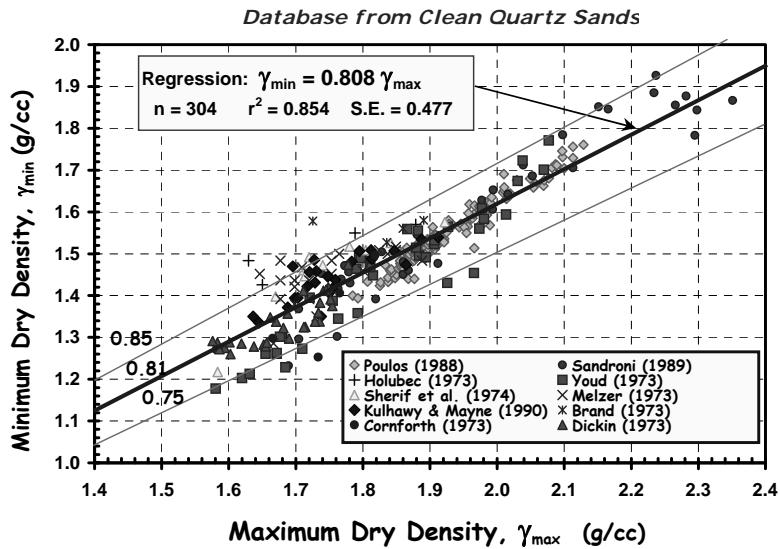


Grain Size Parameters Affecting Max. Density



Georgia Tech

Interrelationship Between Extreme Packing States (data compiled by Mayne, 2000)

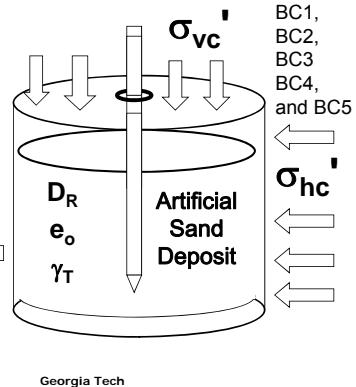


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Calibration Chamber Testing

- Mineralogy
- Grain Distribution
- Angularity
- Index Parameters
- Age
- Origin
- Dry or Saturated
- Stress History

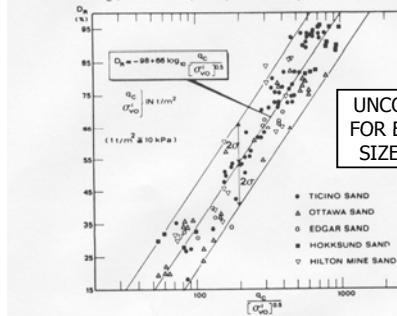
q_c and f_s
 p_0 and p_1
 N_{60}
 P_L



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Relative Density from CPTs in Chambers

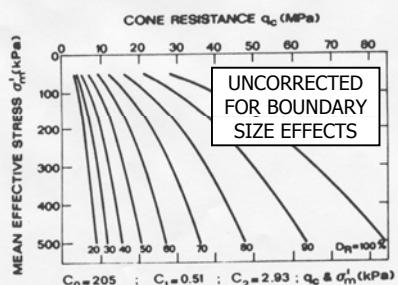
Jamiolkowski, Ladd, Germaine & Lancellotta (1985). New developments in field & lab testing of soils. Proceedings, 11th ICSMFE, Vol. 1, San Francisco, 57-153.



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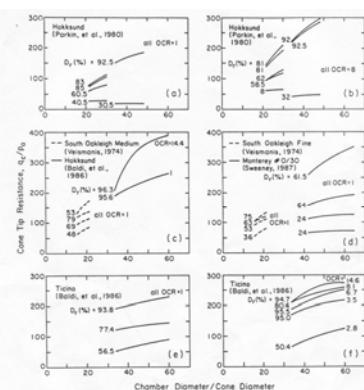
Relative Density from CPTs in Chambers

Baldi, Bellotti, et al. (1986). Interpretation of CPTs and CPTUs: Drained penetration of sands. Proceedings, Fourth International Geotechnical Seminar, Singapore, 143-156.



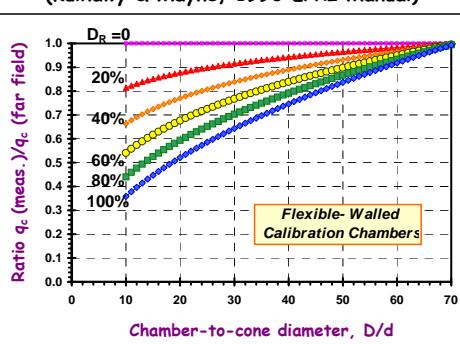
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Correction Factors for Boundary Size Effects



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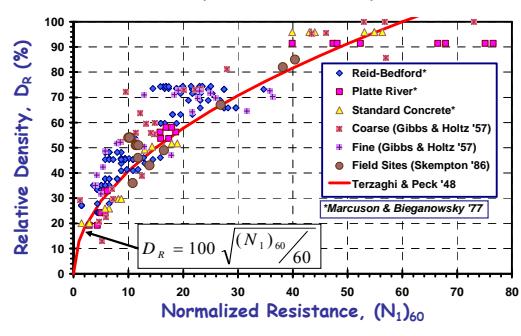
Calibration Chamber Correction Factors (Kulhawy & Mayne, 1990 EPRI Manual)



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Relative Density from SPT-N Value

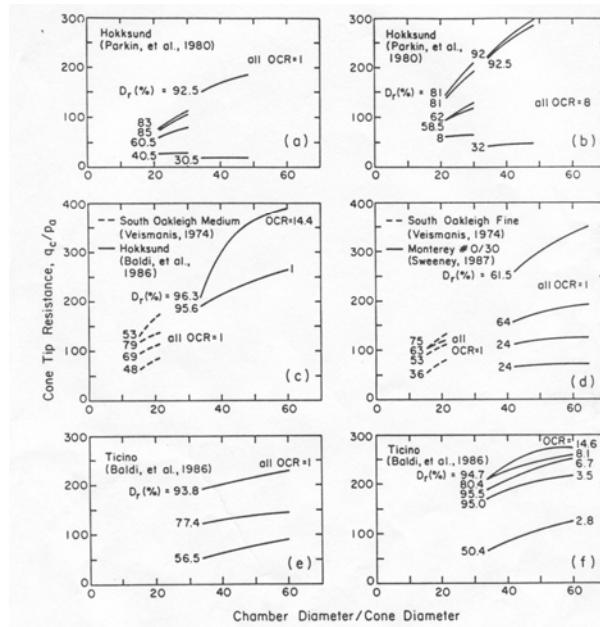
After A.W. Skempton (Geotechnique, 1986)



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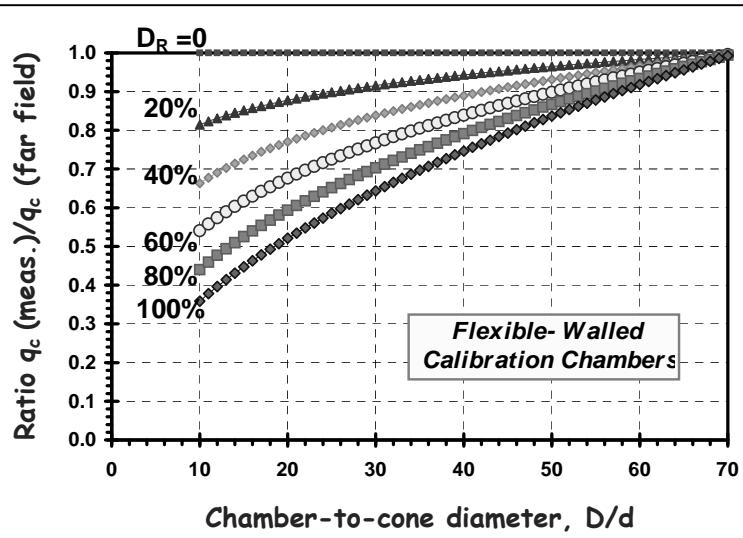
Calibration Chamber Testing

Correction Factors for Boundary Size Effects



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Calibration Chamber Correction Factors (Kulhawy & Mayne, 1990 EPRI Manual)



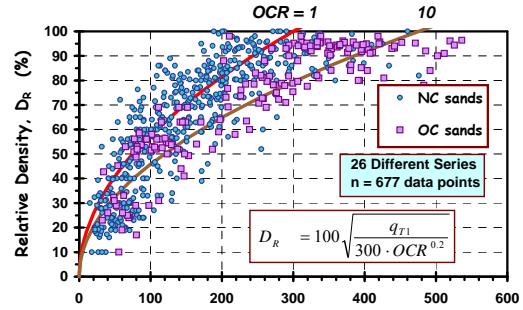
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Relative Density of Quartz Sands

- ❑ Use stress-normalized resistances:
- ❑ SPT: $(N_1)_{60} = N_{60}/(\sigma_{vo}'/\sigma_{atm})^{0.5}$
where σ_{vo}' = effective overburden stress
 $\sigma_{atm} = 1 \text{ atm} \approx 1 \text{ tsf} \approx 1 \text{ kg/cm}^2 = 100 \text{ kPa}$
- ❑ CPT: $q_{t1} = (q_t/\sigma_{atm})/(\sigma_{vo}'/\sigma_{atm})^{0.5}$
- ❑ If all stress terms are in atmospheres then:
SPT: $(N_1)_{60} = N_{60}/(\sigma_{vo}')^{0.5}$
CPT: $q_{t1} = q_t/(\sigma_{vo}')^{0.5}$

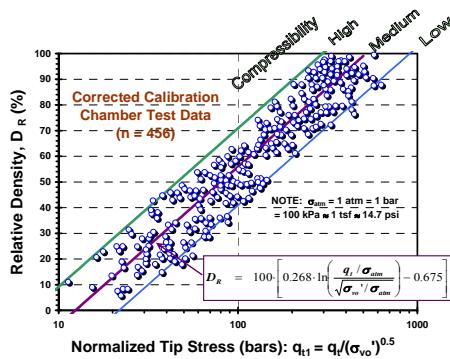
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Relative Density from CPT Resistances (after Kulhawy & Mayne, 1990 EPRI Manual)

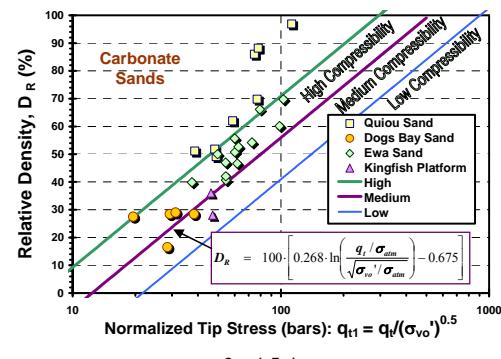


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Corrected Calibration Chamber Test Data (Jamiolkowski et al. 2001, ASCE GSP 119)



Relative Density: Carbonate Sands Singapore Workshop (2006): Char. & Engng Properties Natural Soils



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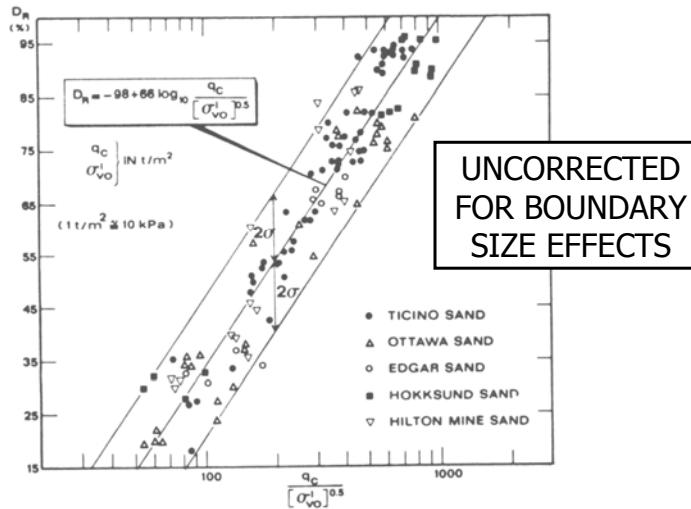
Other approaches to assess state (e_o , D_R , γ_t)

- ❑ Nuclear Cone using radio-isotopes (e.g. Mimura, et al., CPT'95)
- ❑ Shear Wave Velocity (Mayne, 2001)
- ❑ Resistivity Cone (Piccolo, et al. 1995)
- ❑ Dielectric Probe (Santamarina, et al. 2001)
- ❑ Time Domain Reflectometry (TDR) to measure water content (Dowding, 2001; Drnevich 2006).

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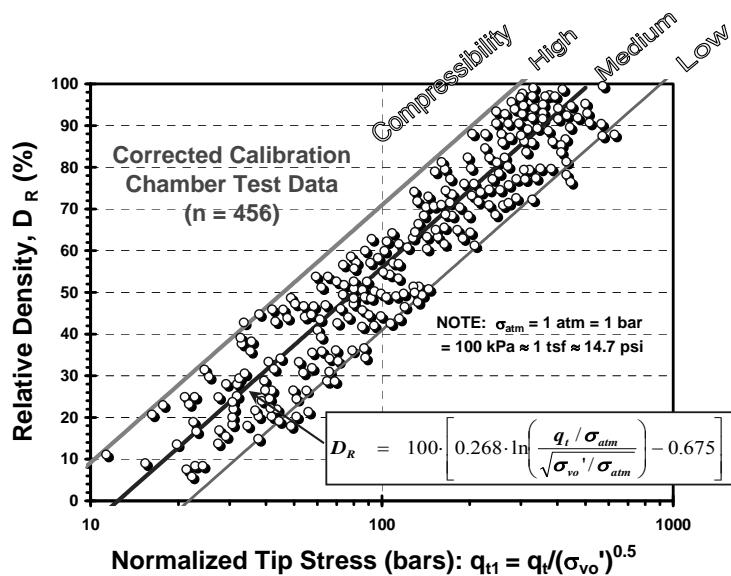
Relative Density from CPTs in Chambers

Jamiolkowski, Ladd, Germaine & Lancellotta (1985). New developments in field & lab testing of soils. Proceedings, 11th ICSMFE, Vol. 1, San Francisco, 57-153.



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Corrected Calibration Chamber Test Data (Jamiolkowski et al. 2001, ASCE GSP 119)

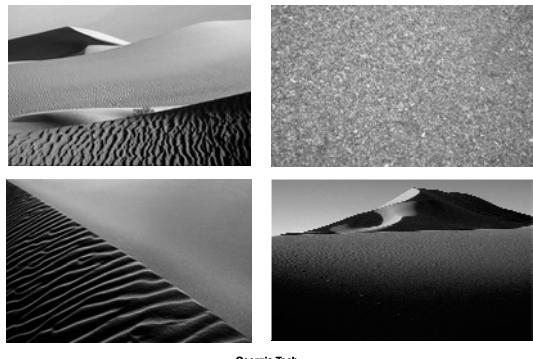


June 2008

Effective Stress Friction Angle from CPT

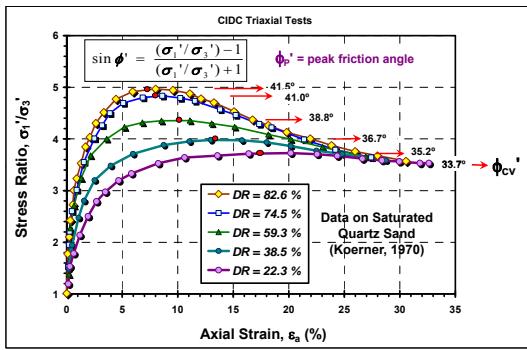
Paul W. Mayne, PhD, P.E.
Georgia Institute of Technology

Natural Clean Siliceous Sands



Georgia Tech

Friction Angle of Sands



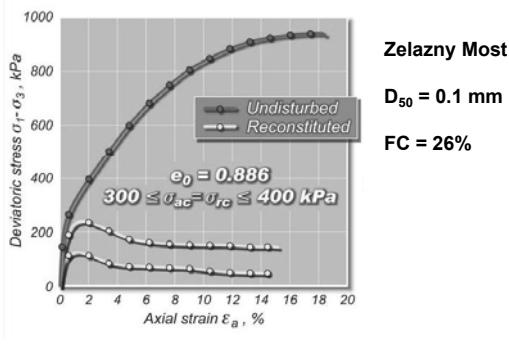
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Undisturbed vs. Reconstituted Sands

- Reconstituted: Artificial
 - Air Pluviation
 - Moist Tamped
 - Water Sedimented
 - Slurried
 - Compacted
- Undisturbed: Natural
 - 1-d Freezing (tubes or coring)
 - Agar Injection

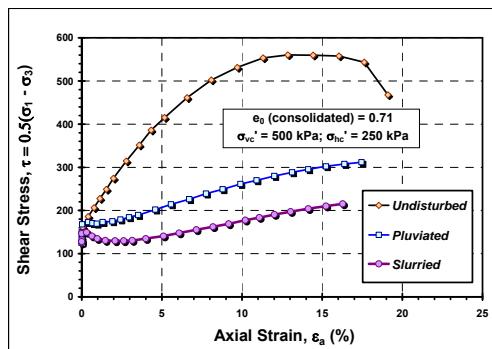
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Polish Tailings Sand (Jamiolkowski 2006)

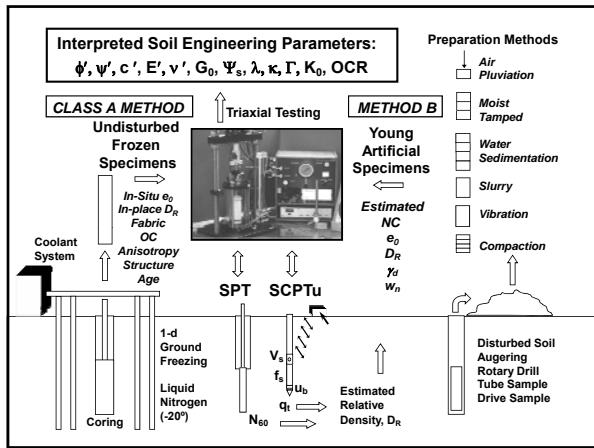


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Silty Sand (Høeg, Dyvik, & Sandækken, 2000)



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Undisturbed Sand Database

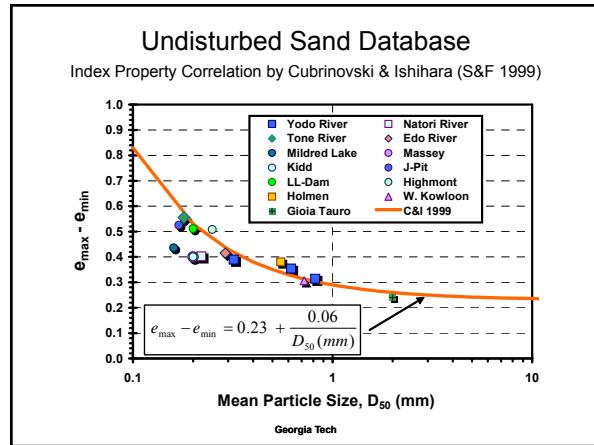
Sand Name	Country	Reference Sources	Type Sand	Method of Sampling
W. Kowloon	China	Lee, Shen, Leung, Mitchell (1999)	Hydraulic Fill	Mazier Tube
Yodo	Japan	Mimura (2003)	Natural Alluvial	Frozen
Yodo	Japan	Mimura (2003)	Natural Alluvial	Frozen
Yodo	Japan	Mimura (2003)	Natural Alluvial	Frozen
Natori	Japan	Matsu & Tsutsumi (1998)	Natural Alluvial	Frozen
Tone	Japan	Matsu & Tsutsumi (1998)	Natural Alluvial	Frozen
Edo	Japan	Yamashita et al. (2003)	Natural Alluvial	Frozen
Mildred L.	Canada	Robertson, Wride, et al. (2000)	Hydraulic Fill	Frozen
Massey	Canada	Robertson, Wride, et al. (2000)	Natural Alluvial	Frozen
Kidd	Canada	Robertson, Wride, et al. (2000)	Natural Alluvial	Frozen
J-pit	Canada	Robertson, Wride, et al. (2000)	Hydraulic Fill	Frozen
LL Dam	Canada	Robertson, Wride, et al. (2000)	Tailings	Frozen
Hightmont	Canada	Robertson, Wride, et al. (2000)	Tailings	Frozen
Holmen	Norway	Lunne, Long, & Forsberg (2003)	Natural Alluvial	Tube/Frozen
Gioia Tauro	Italy	Ghianna & Porcino (2006)	Natural Coarse Sands	Frozen

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Undisturbed Sand Database

Sand Name	Depth (m)	GWL (m)	Mean z (m)	Solids G_s	Fines (%)	UC = D_{50}/D_{10}	D ₅₀ (mm)
W. Kowloon	11.8	9	11.8	2.63	1	6.4	0.72
Yodo	8.1	2.1	8.1	2.64	1.9	3.1	0.32
Yodo	10.9	2.1	10.9	2.63	0.27	3.3	0.82
Yodo	12.7	2.1	12.7	2.63	2.1	3	0.62
Natori	8.25	2.1	8.25	2.65	0.23	2	0.22
Tone	7.3	1.4	7.3	2.68	3.78	2	0.18
Edo	3.85	2.1	3.85	2.68	0.42	2.2	0.29
Mildred L.	27 - 37	21	32	2.66	10	2.22	0.16
Massey	8 - 13	1.5	10.5	2.68	5	1.57	0.2
Kidd	12 - 17	1.5	14.5	2.72	5	1.78	0.2
J-pit	3 - 7	0.5	5	2.62	15	2.5	0.17
LL Dam	6 - 10	2.1	8	2.66	8	2.78	0.2
Hightmont	8 - 12	4	10	2.66	10	4	0.25
Holmen	5 to 15	1	10	2.71	2	3	0.55
Gioia Tauro	1.3 - 4.2	1.5	3	2.69	0.66	2.1	2

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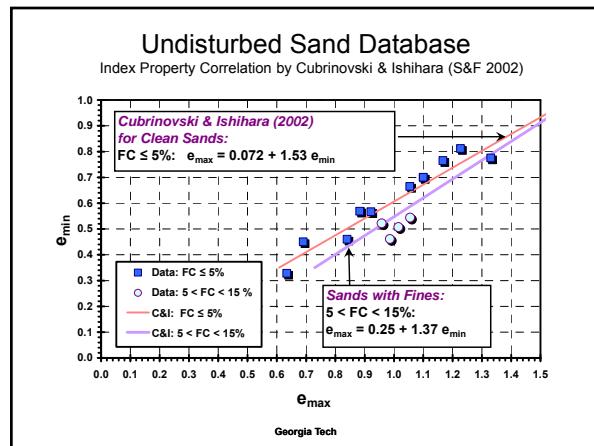


Undisturbed Sand Database

(Ref: 2006 JK Mitchell Lecture)

Sand Name	Country	Location	Void Ratio Parameters e_a , e_{max}	D_R (%)	σ_{vo}^* (atm)	In-Situ Test Methods	Laboratory Triaxial Tests		
Kowloon	China	0.492	0.634	46.5	1.80	SPT	CPTu	CIUC	
Yodo	Japan	0.820	1.054	0.665	60.2	1.02	SPT	SCPTu	CIUC
Yodo	Japan	0.720	0.883	0.569	51.9	1.23	SPT	SCPTu	CIUC
Yodo	Japan	0.790	0.921	0.567	37.0	1.43	SPT	SCPTu	CIUC
Natori	Japan	0.857	1.167	0.765	77.2	0.87	SPT	SCPTu	CIUC
Tone	Japan	0.947	1.330	0.775	69.1	0.84	SPT	SCPTu	CIUC
Edo	Japan	1.043	1.227	0.812	44.3	0.51	SPT	SCPTu	CIUC
Mildred L.	Canada	0.768	0.958	0.522	43.6	5.16	SPT	SCPTu	CIUC, CK <u>UC</u>
Massey	Canada	0.970	1.100	0.700	32.5	1.20	SPT	SCPTu	CIUC, CK <u>UC</u>
Kidd	Canada	0.981	1.100	0.700	29.8	1.60	SPT	SCPTu	CIUC, CK <u>UC</u>
J-pit	Canada	0.762	0.986	0.461	42.7	0.55	SPT	SCPTu	CIUC, CK <u>UC</u>
LL Dam	Canada	0.849	1.055	0.544	40.3	1.00	SPT	SCPTu	CIUC, CK <u>UC</u>
Hightmont	Canada	0.825	1.015	0.507	37.4	1.38	SPT	SCPTu	CK <u>UC</u> , CK <u>DC</u>
Holmen	Norway	0.724	0.840	0.460	30.5	1.10	SPT	SCPTu	CK <u>UC</u> , CK <u>DC</u>
Gioia Tauro	Italy	0.589	0.690	0.450	42.0	0.42	SPT	SCPTu	CIUC, CIUC, CTX

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Undisturbed Sand Database

(Ref: 2006 James K. Mitchell Lecture)

Sand Name	Country Location	Void Ratio Parameters			D _R (%)	σ _{vo} ⁱ (atm)	In-Situ Test Methods	Laboratory Triaxial Tests
<i>Kowloon</i>	China	0.492	0.634	0.328	46.5	1.80	SPT	CPTu CIUC
<i>Yodo</i>	Japan	0.820	1.054	0.665	60.2	1.02	SPT	SCPTu CIDC
<i>Yodo</i>	Japan	0.720	0.883	0.569	51.9	1.23	SPT	SCPTu CIDC
<i>Yodo</i>	Japan	0.790	0.921	0.567	37.0	1.43	SPT	SCPTu CIDC
<i>Natori</i>	Japan	0.857	1.167	0.765	77.2	0.87	SPT	SCPTu CIDC
<i>Tone</i>	Japan	0.947	1.330	0.775	69.1	0.84	SPT	SCPTu CIDC
<i>Edo</i>	Japan	1.043	1.227	0.812	44.3	0.51	SPT	SCPTu CIDC
<i>Mildred L.</i>	Canada	0.768	0.958	0.522	43.6	5.16	SPT	SCPTu CIUC, CKoUC
<i>Massey</i>	Canada	0.970	1.100	0.700	32.5	1.20	SPT	SCPTu CIUC, CKoUC
<i>Kidd</i>	Canada	0.981	1.100	0.700	29.8	1.60	SPT	SCPTu CIUC, CKoUC
<i>J-pit</i>	Canada	0.762	0.986	0.461	42.7	0.55	SPT	SCPTu CIUC, CKoUC
<i>LL Dam</i>	Canada	0.849	1.055	0.544	40.3	1.00	SPT	SCPTu CIUC, CKoUC
<i>Highmont</i>	Canada	0.825	1.015	0.507	37.4	1.38	SPT	SCPTu CIUC, CKoUC
<i>Holmen</i>	Norway	0.724	0.840	0.460	30.5	1.10	SPT	SCPTu CKoUC, CKoDC
<i>Gioia Tauro</i>	Italy	0.589	0.690	0.450	42.0	0.42	SPT	SCPTu CIDC, CIUC, CTX

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Undisturbed Sand Database

Summary of SPT, CPT, Vs, and Triaxial Data

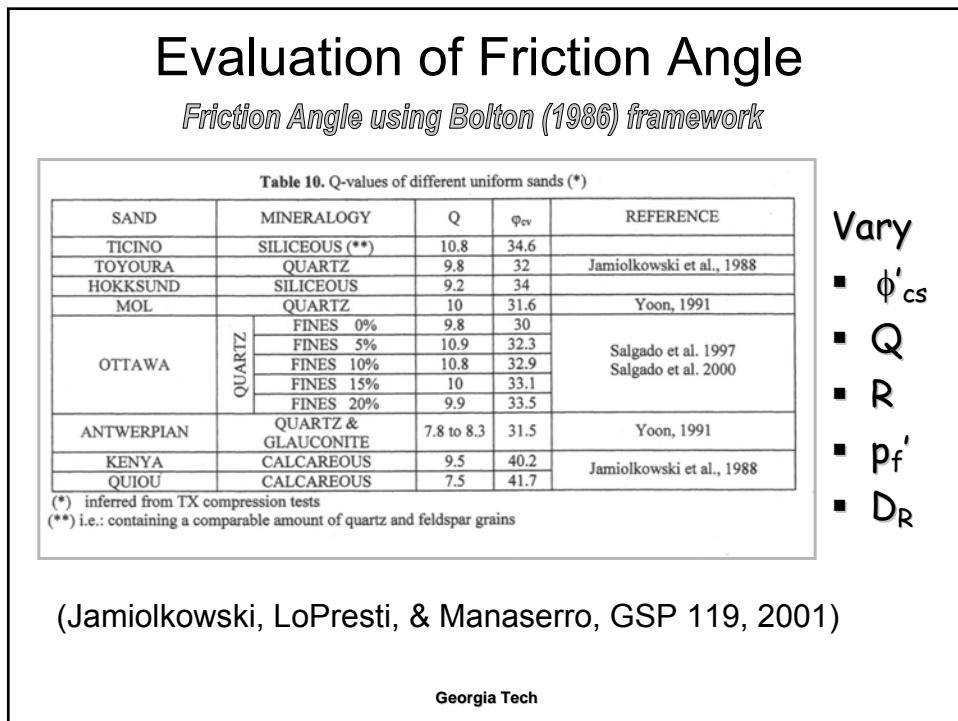
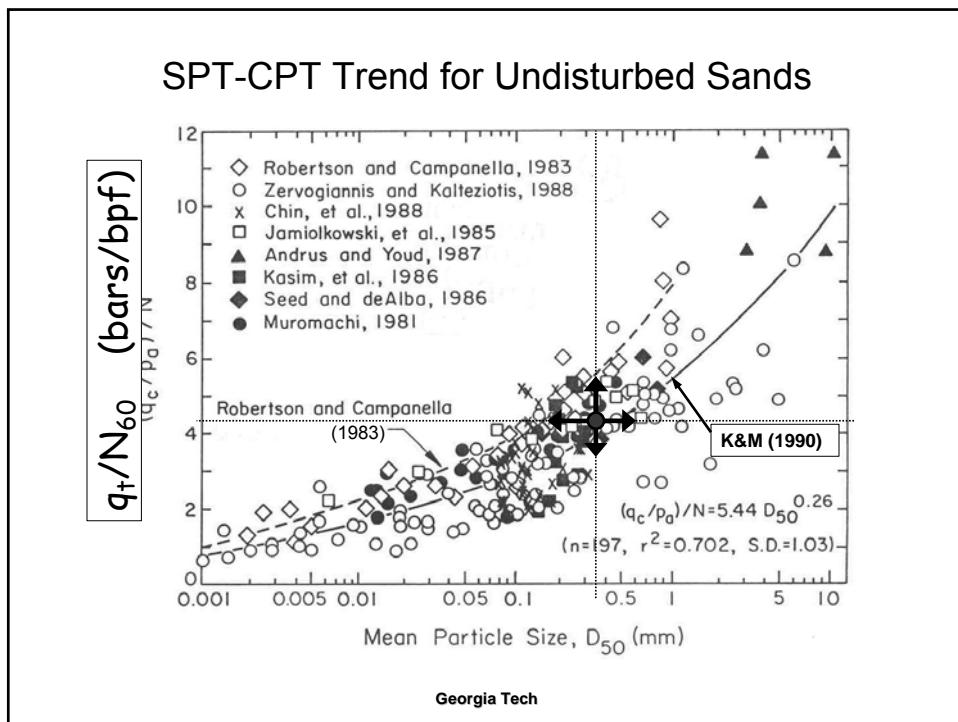
Sand Name	Laboratory	ϕ ⁱ (deg)	SPT (N _i) ₆₀	Normalized CPT Resistances and Readings				SBT**	V _{s1} (m/s)
Name	Triaxial Type			Q	F	f _{s/q_t} (%)	q _{t1}	Δu/σ _{vo} ⁱ	I _c
<i>Kowloon</i>	CIUC	38.1	21	55.4	1.84	1.80	74.5	0.067	2.23 NA
<i>Yodo</i>	CIDC	42.4	27	193.7	0.95	0.94	175.9	-0.051	1.63 197.2
<i>Yodo</i>	CIDC	38.4	35	102.6	1.03	1.01	101.6	-0.190	1.81 213.1
<i>Yodo</i>	CIDC	39.1	31	96.0	0.89	0.88	105.7	-0.186	1.76 195.2
<i>Natori</i>	CIDC	40.9	50	225.7	0.30	0.30	207.3	-0.482	1.04 218.1
<i>Tone</i>	CIDC	41.7	32	154.5	0.24	0.24	146.7	-0.458	1.07 203.2
<i>Edo</i>	CIDC	39.7	22	156.1	0.73	0.72	110.6	-0.082	1.55 163.8
<i>Mildred L.</i>	CIUC, CKoUC	39.6	18	32.0	0.73	0.70	73.8	0.025	1.99 156.4
<i>Massey</i>	CIUC, CKoUC	36.7	10	49.4	0.40	0.38	53.4	-0.089	1.63 168.2
<i>Kidd</i>	CIUC, CKoUC	37.3	13	52.4	0.37	0.36	68.3	-0.020	1.59 177.4
<i>J-pit</i>	CIUC, CKoUC	32.7	3	31.0	0.87	0.75	20.4	0.185	2.08 127.1
<i>LL Dam</i>	CIUC, CKoUC	39.1	5	38.6	0.41	0.39	39.4	0.011	1.73 153.1
<i>Highmont</i>	CIUC, CKoUC	41.5	5	35.2	0.38	0.38	43.9	0.112	1.74 141.3
<i>Holmen</i>	CKoUC, CKoDC	33.2	1	29.4	0.42	0.40	27.7	-0.325	1.83 156.6
<i>Gioia Tauro</i>	CIDC, CIUC, CTX	41.5*	30	279.2	0.26	0.26	174.3	-0.261	0.93 221.0

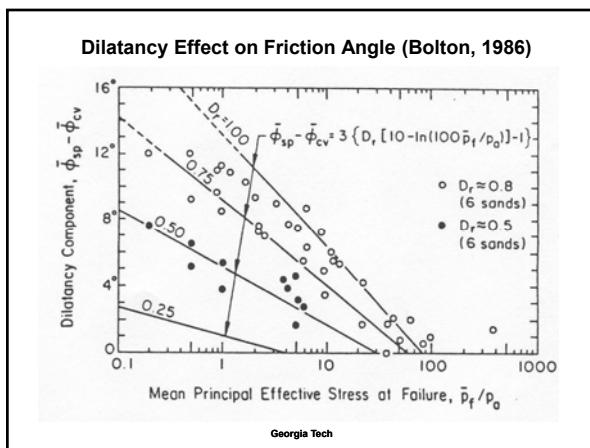
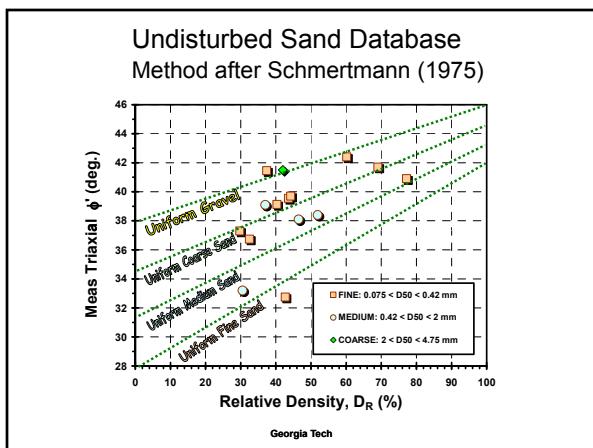
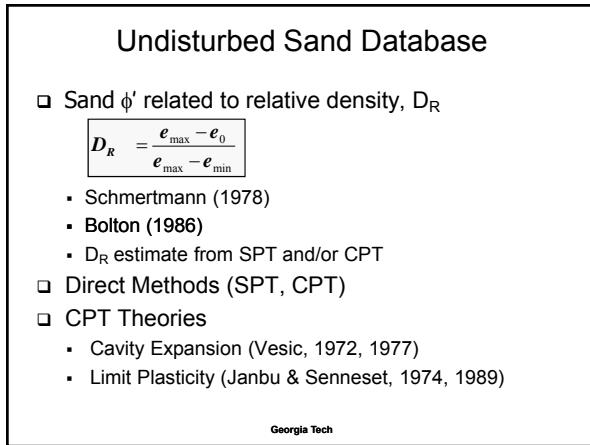
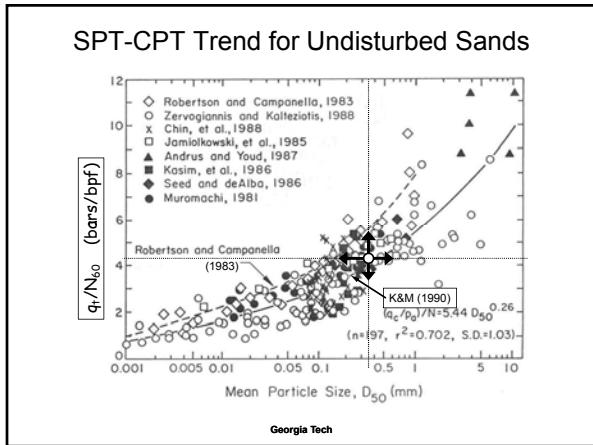
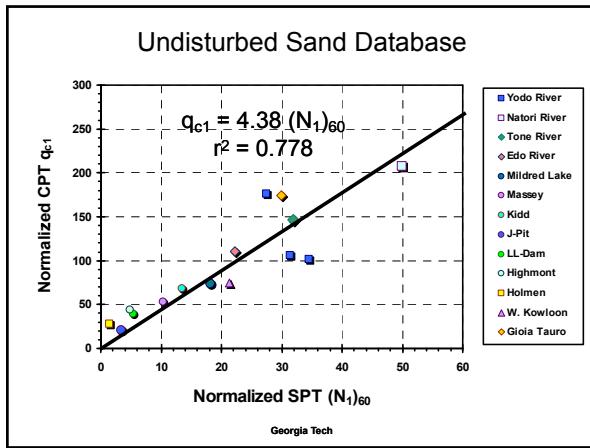
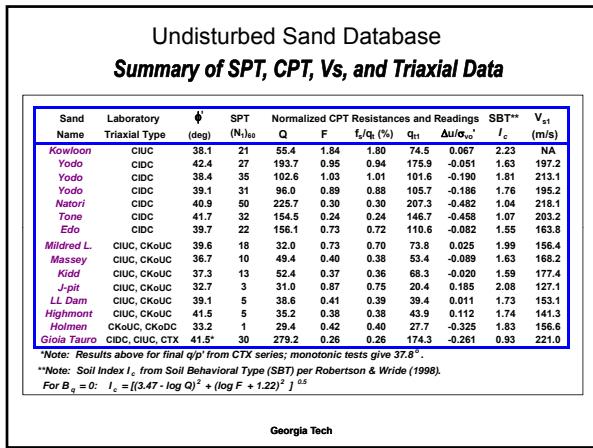
*Note: Results above for final q/p' from CTX series; monotonic tests give 37.8°.

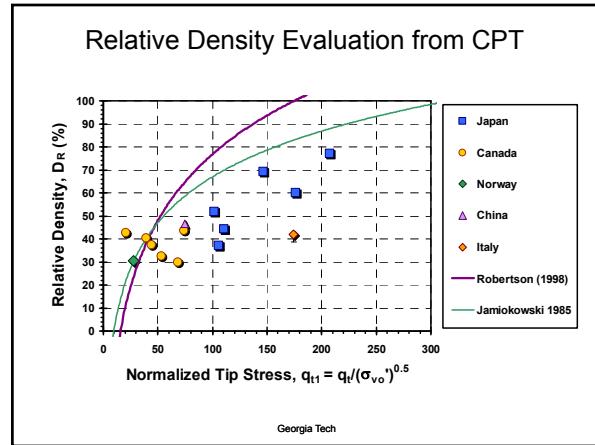
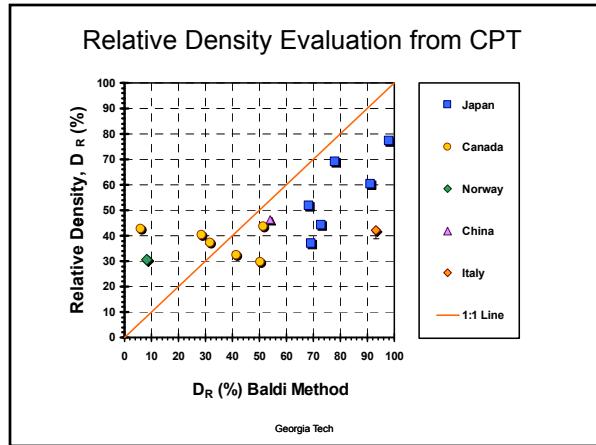
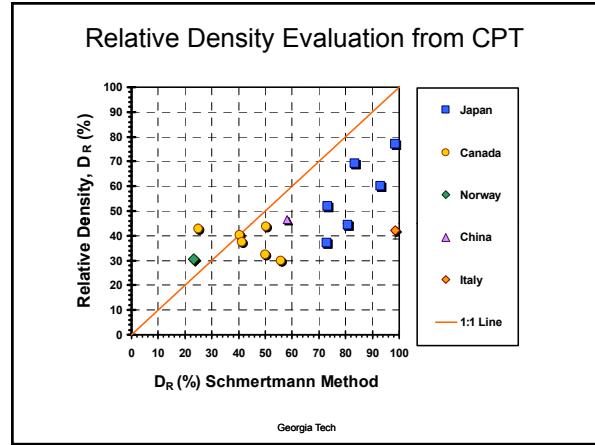
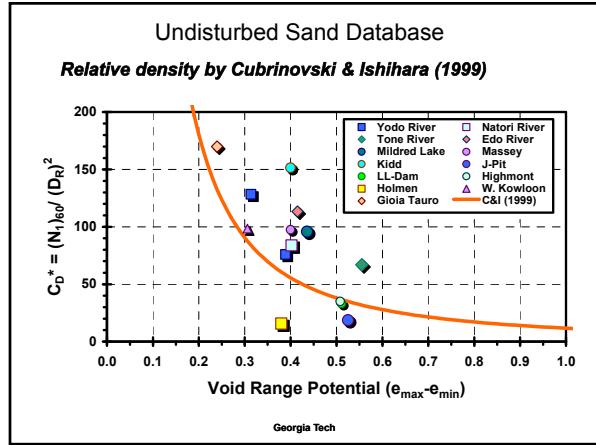
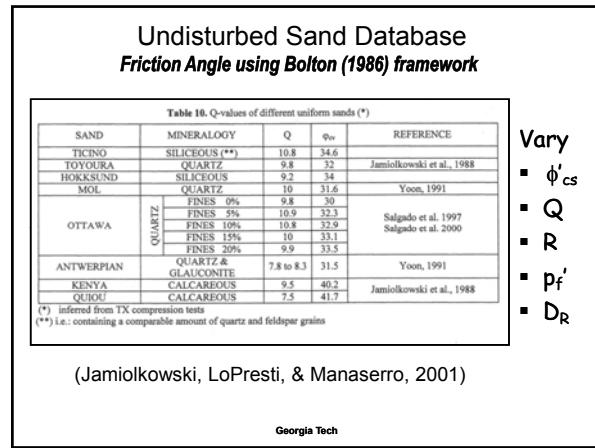
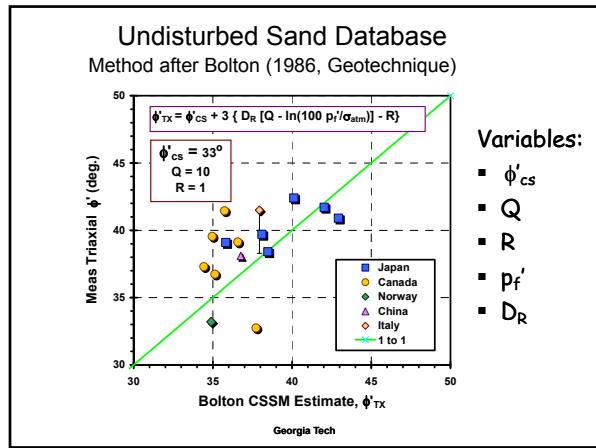
**Note: Soil Index I_c from Soil Behavioral Type (SBT) per Robertson & Wride (1998).

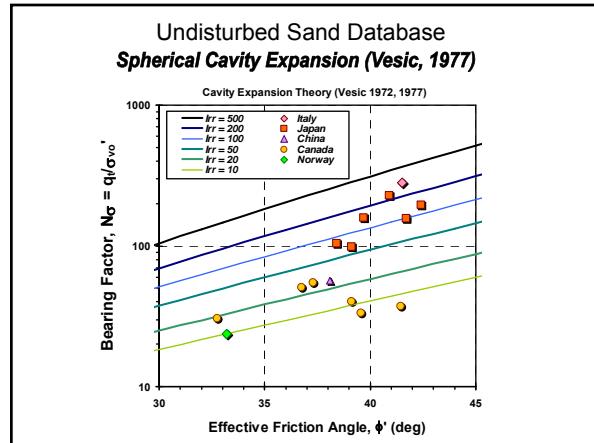
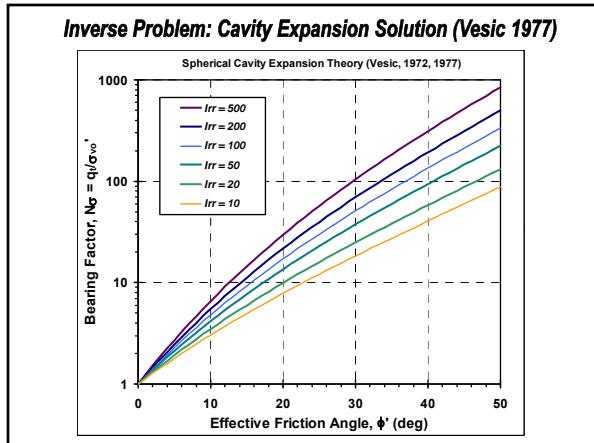
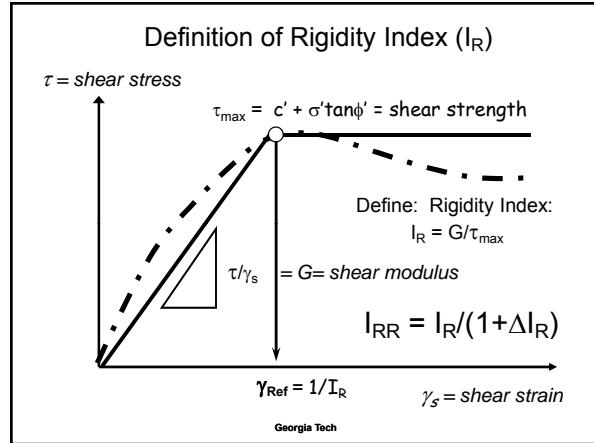
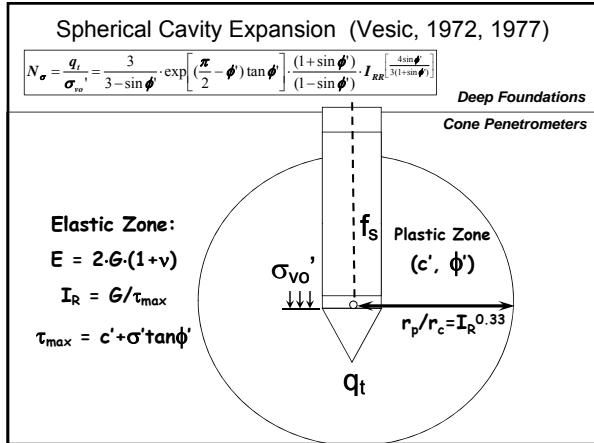
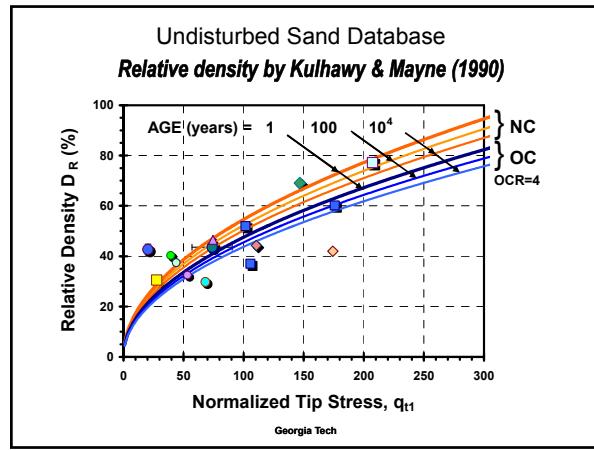
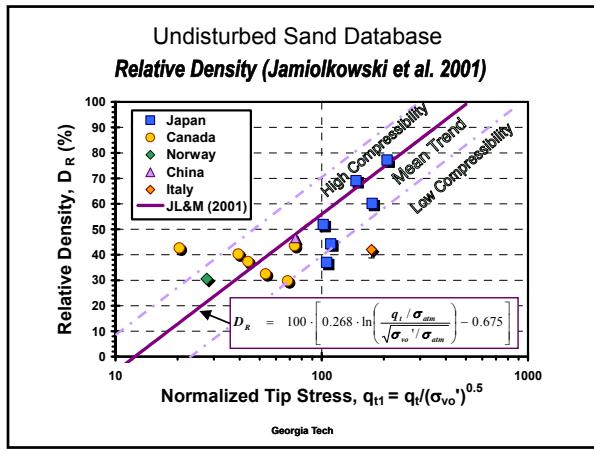
$$\text{For } B_q = 0: \quad I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$$

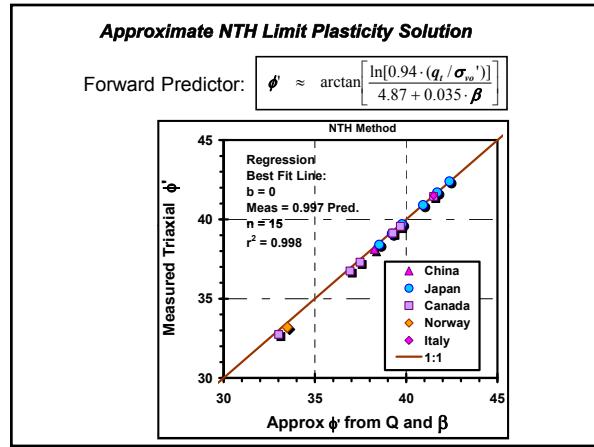
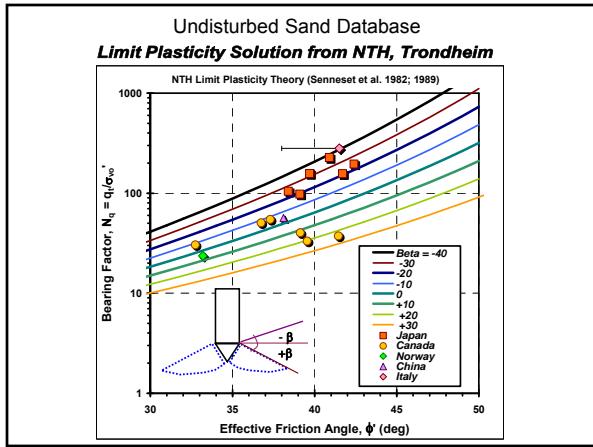
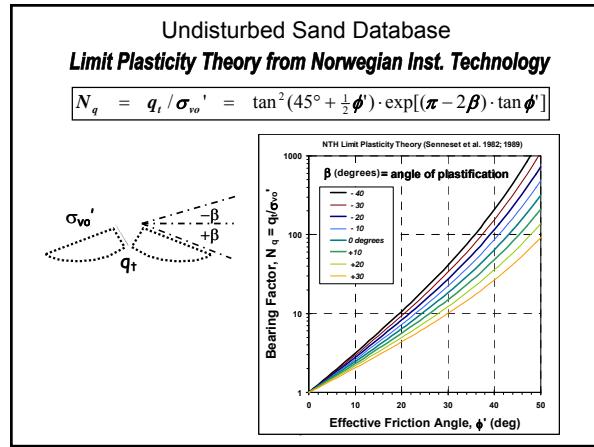
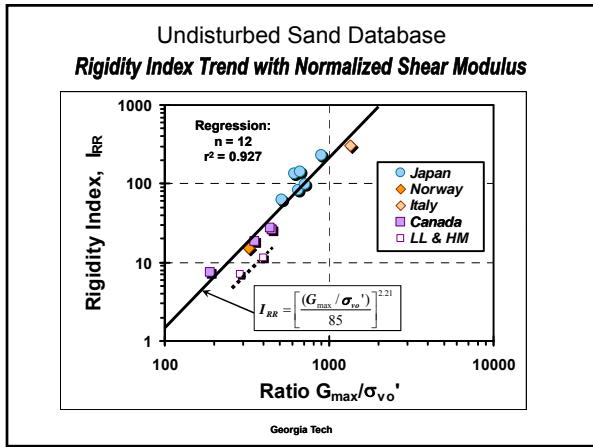
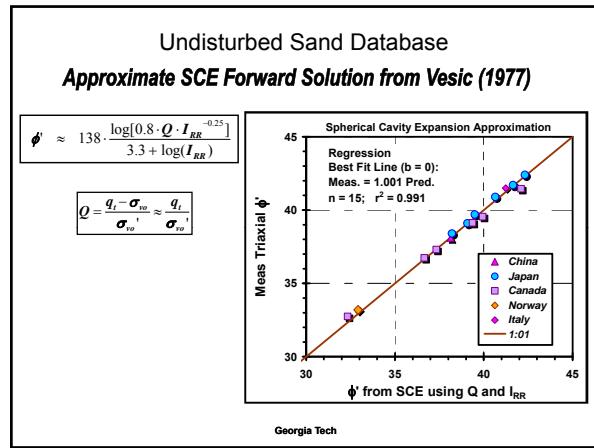
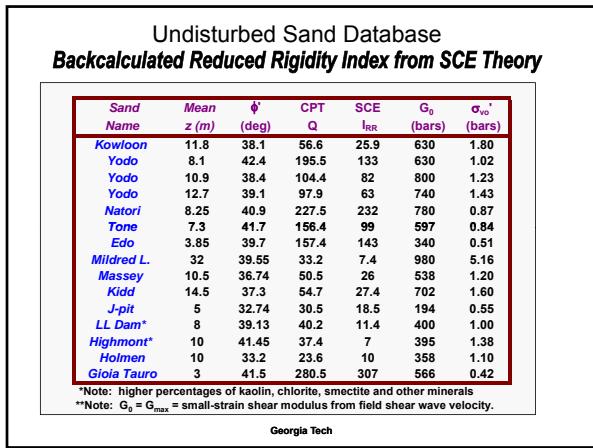
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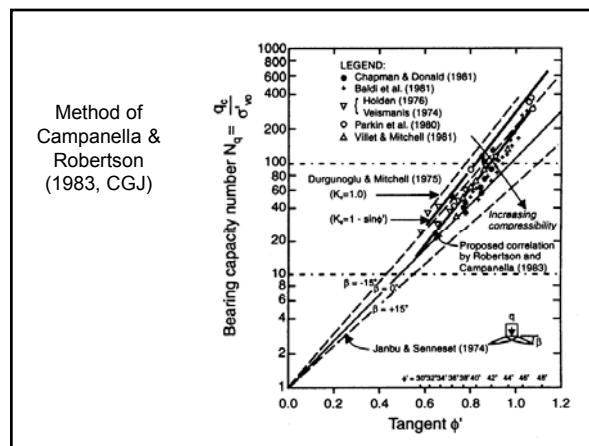
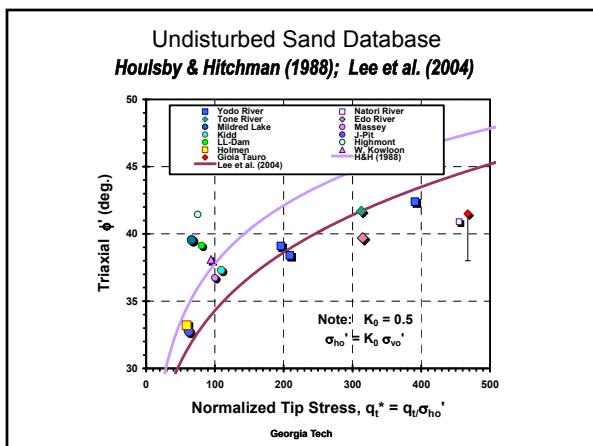
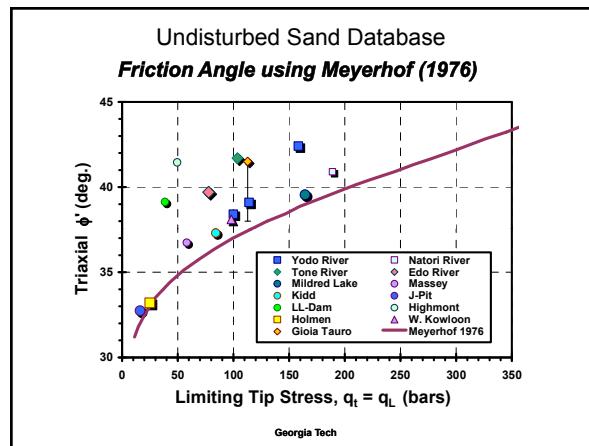
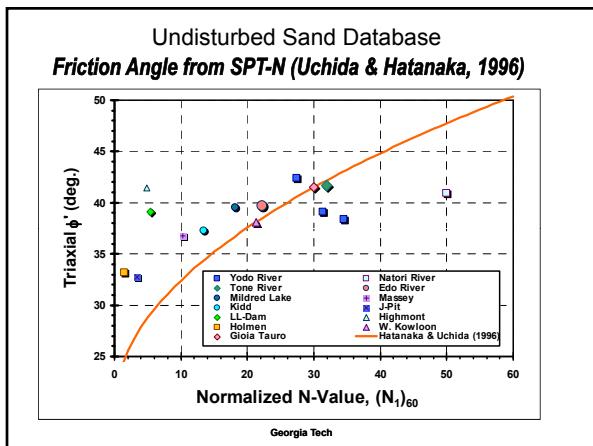
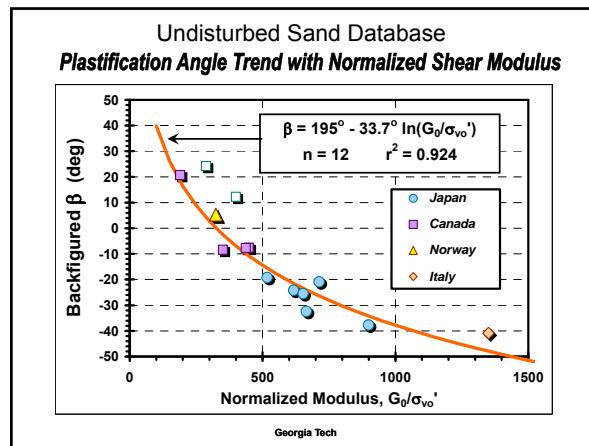
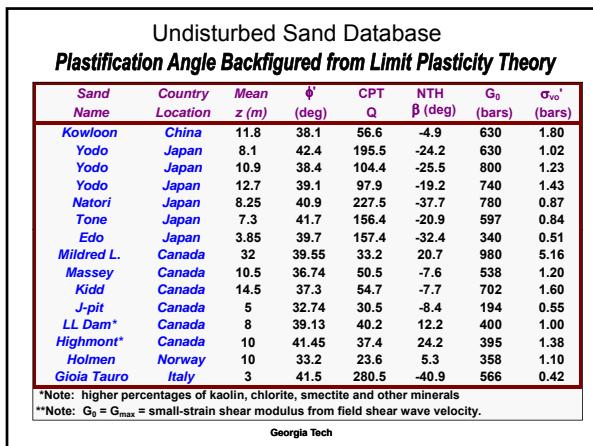


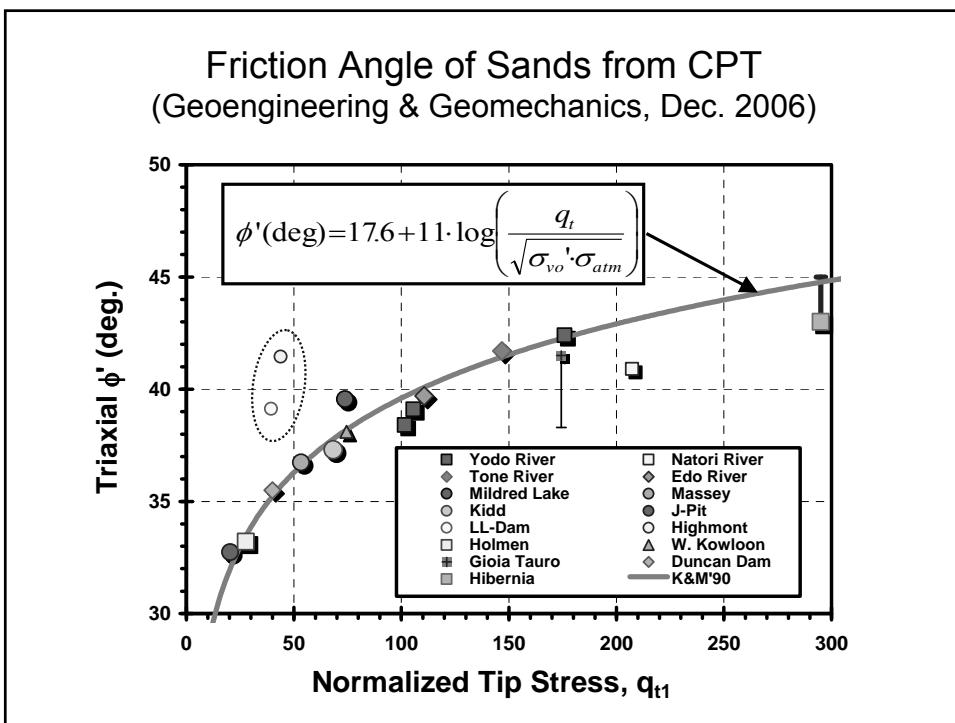
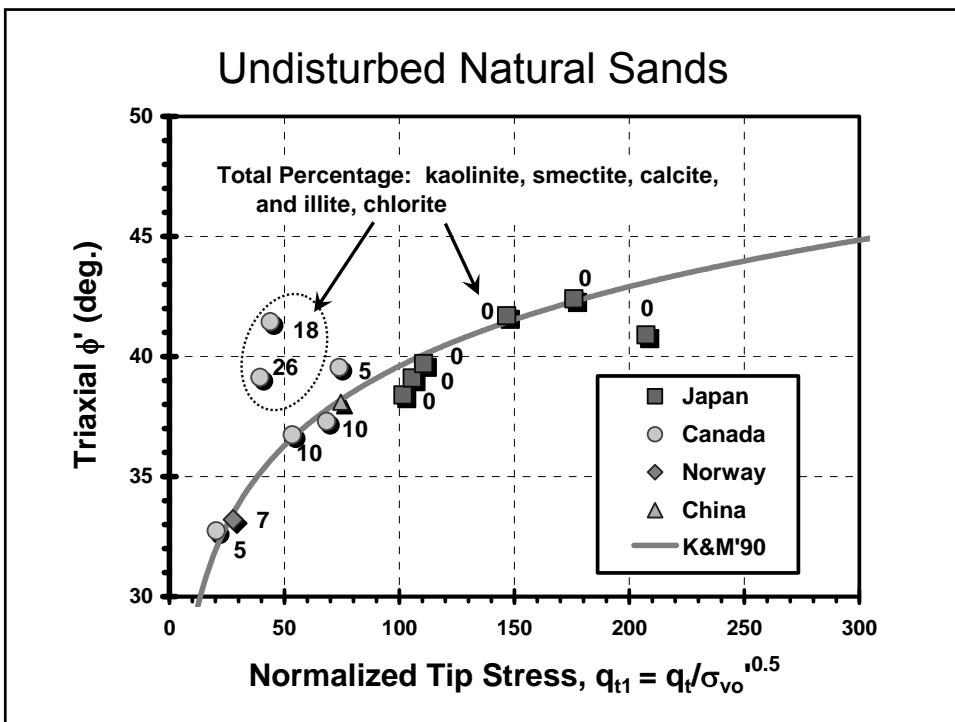


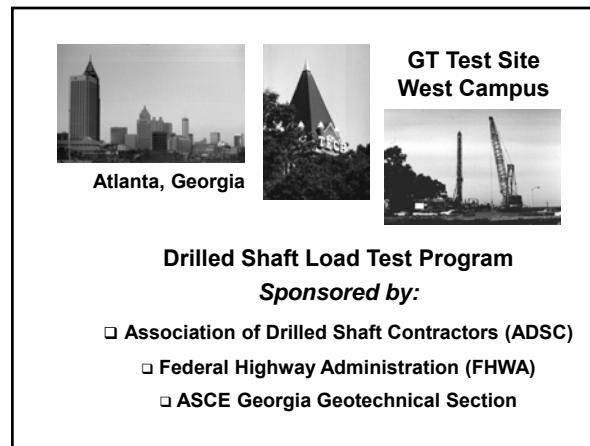
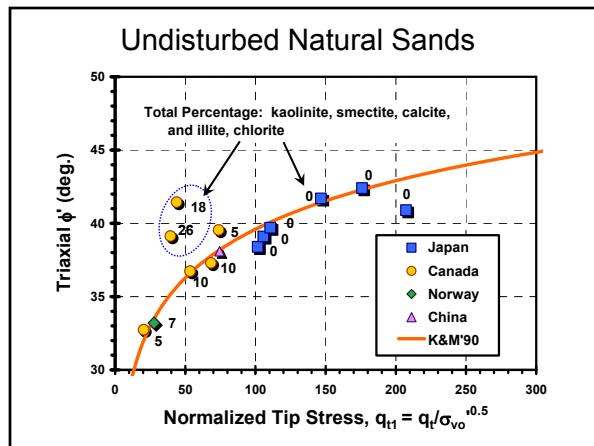
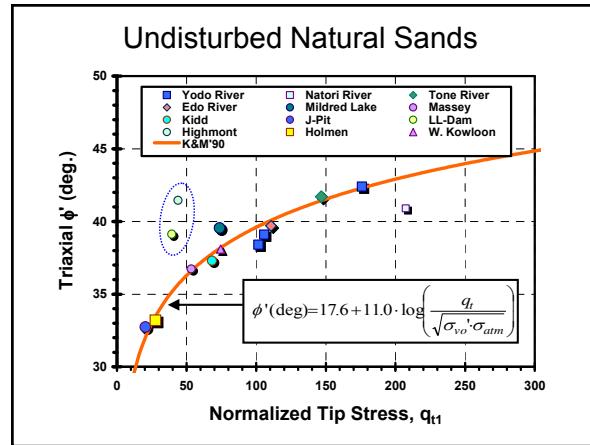
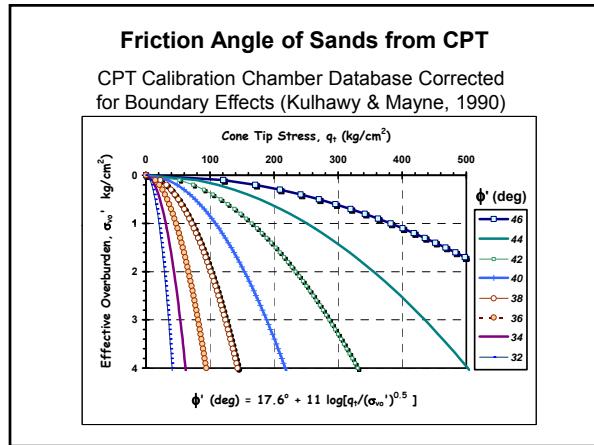
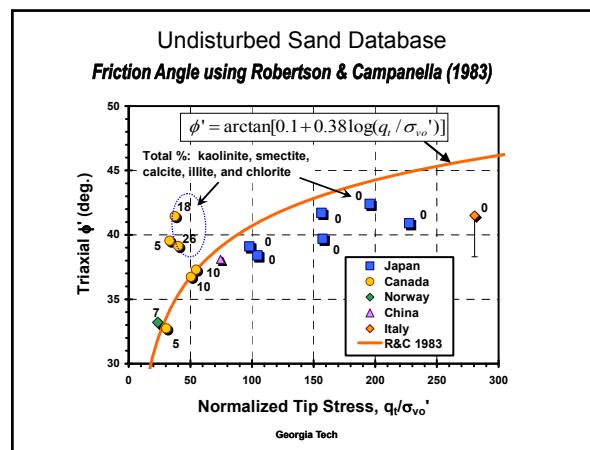
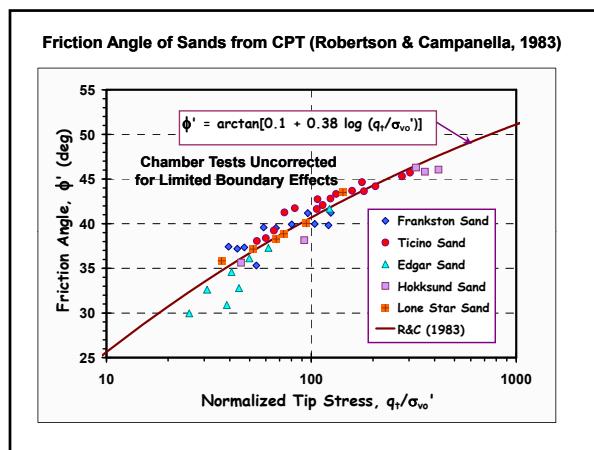


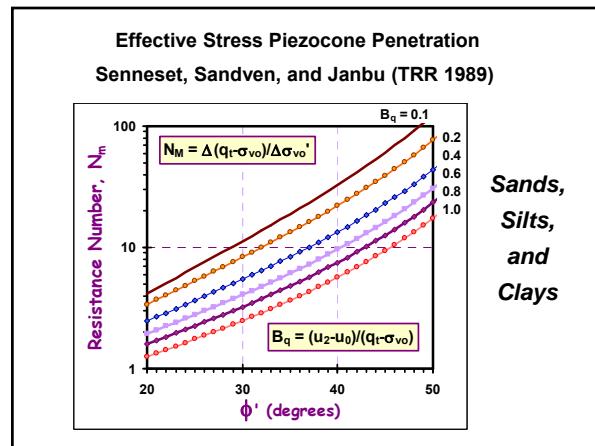
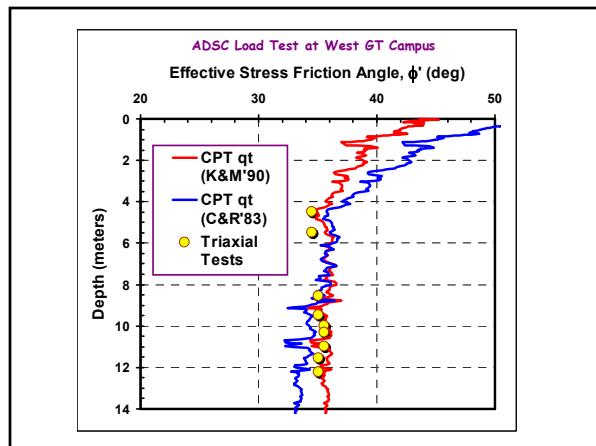
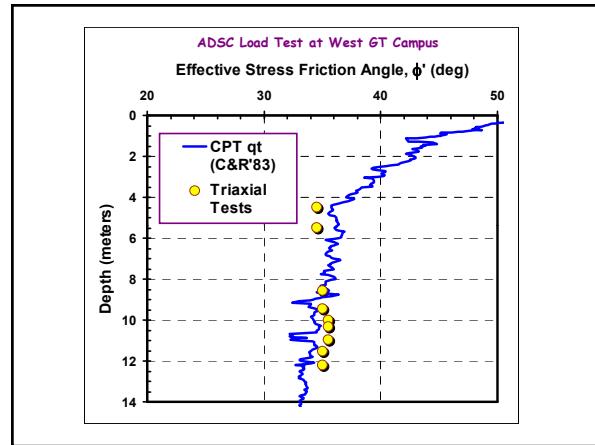
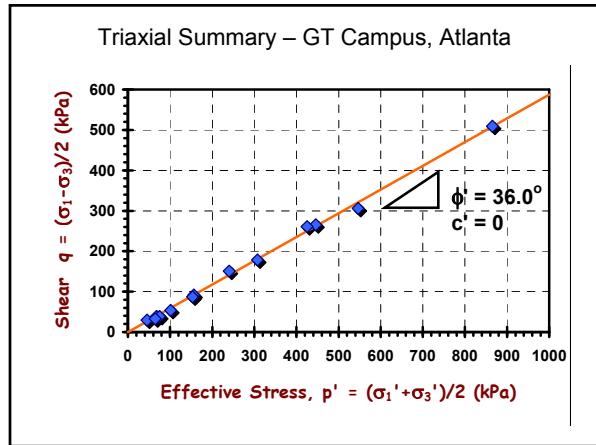
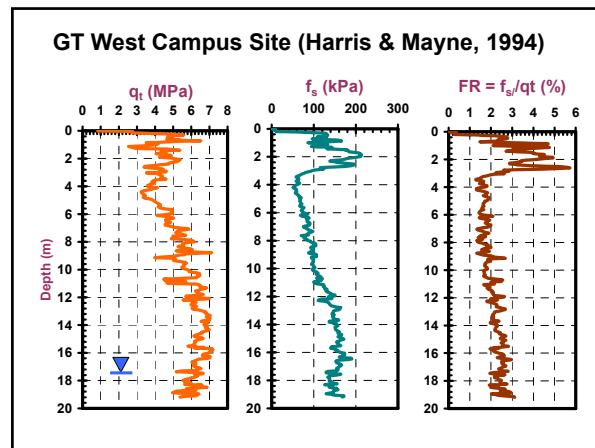
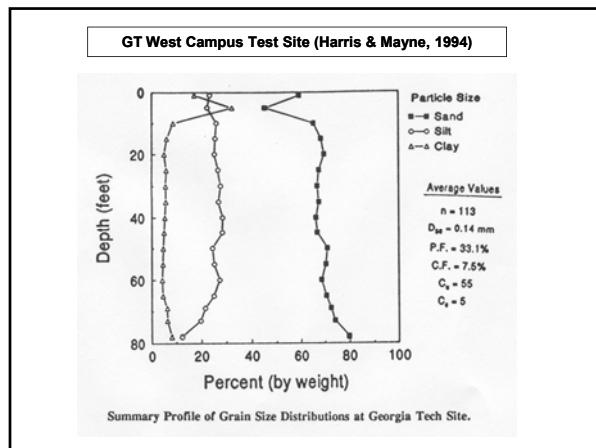




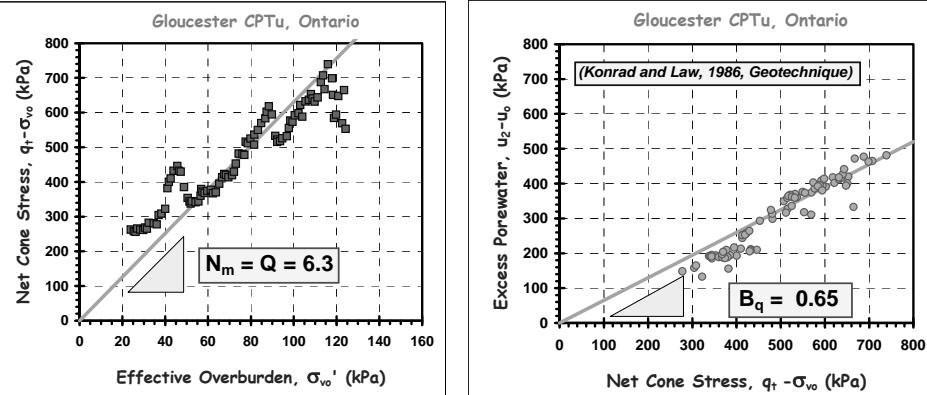








Canadian National Test Site, Gloucester, Ontario



Approximate NTH CPTU - ϕ' Method ($0.1 < B_q < 1.0$)

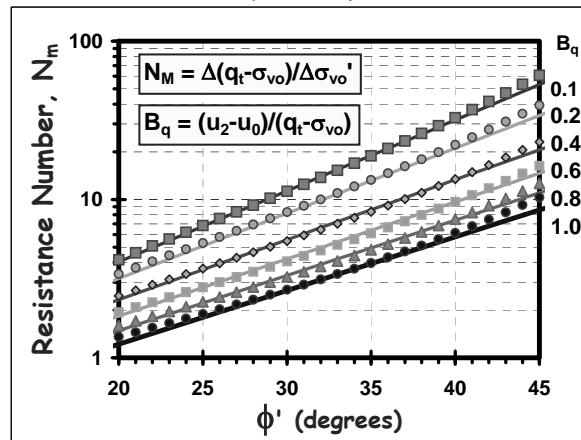
Senneset, Sandven, & Janbu (1989, Transportation Research Record 1235)

For $a' = 0$:

Then: $N_m = Q$

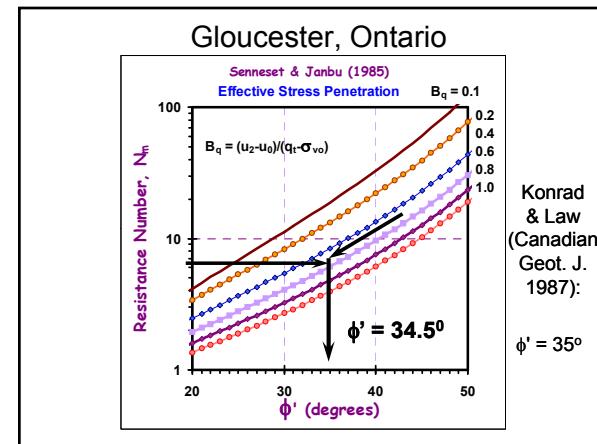
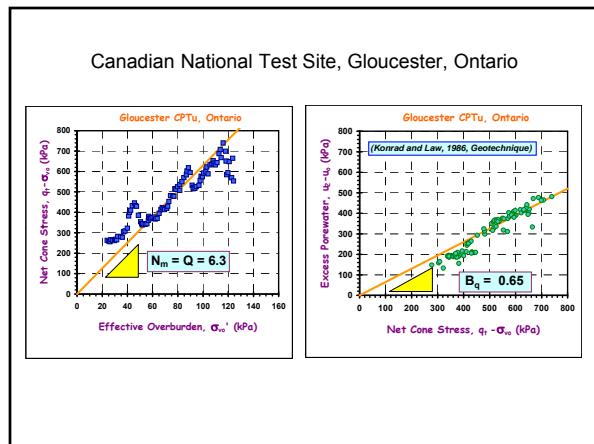
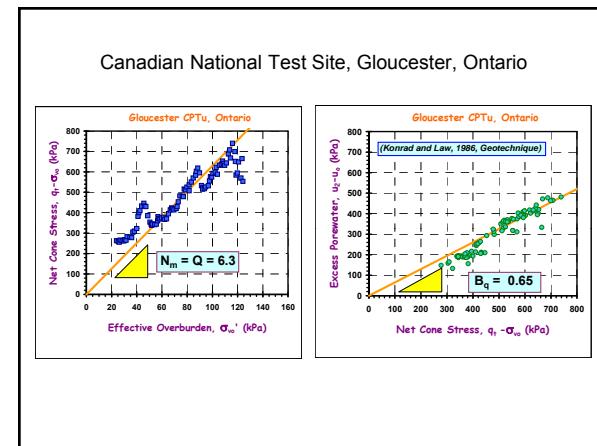
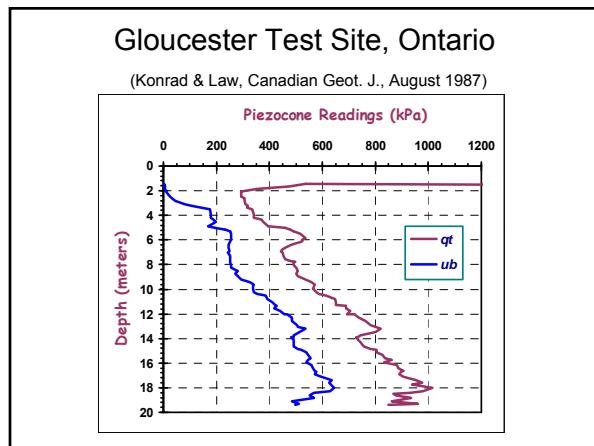
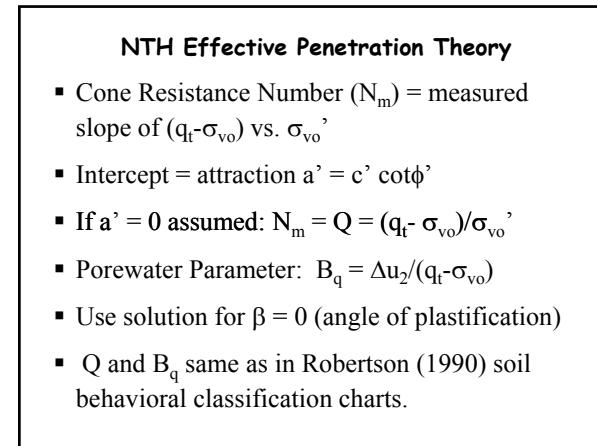
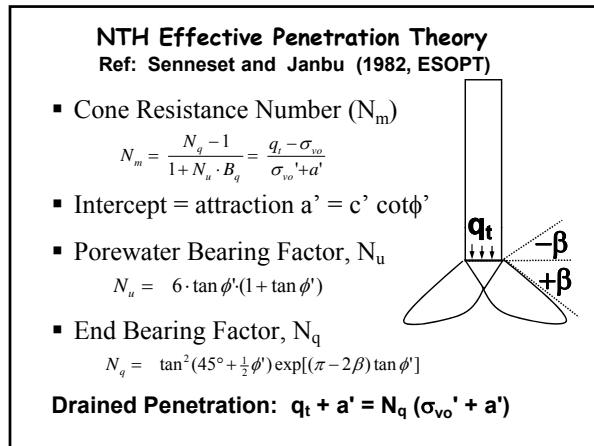
$$= (q_t - \sigma_{vo}) / \sigma_{vo}'$$

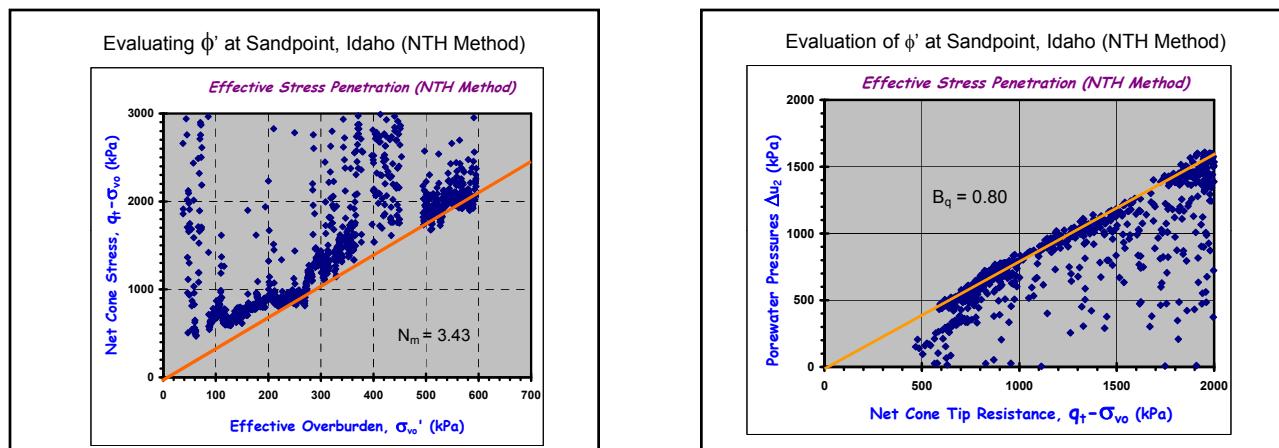
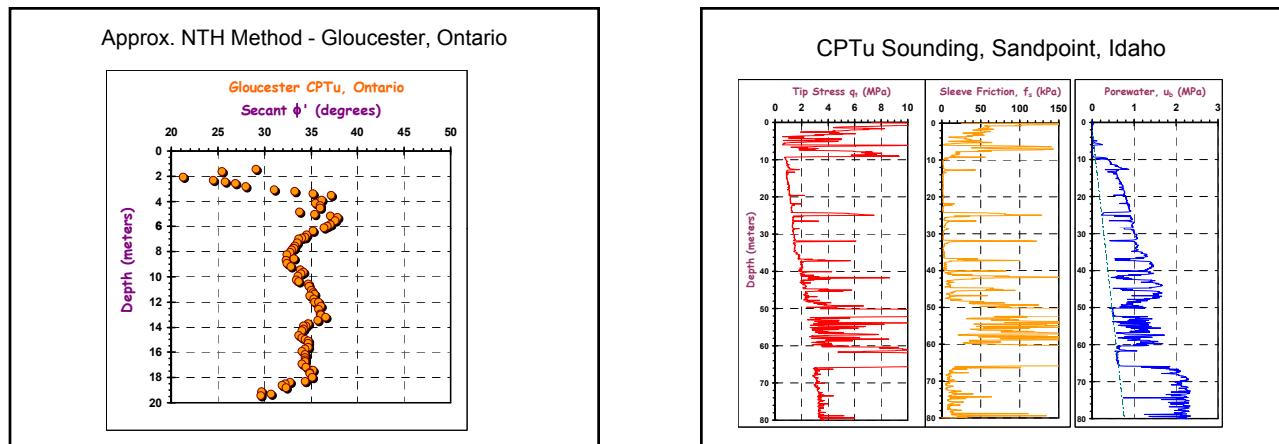
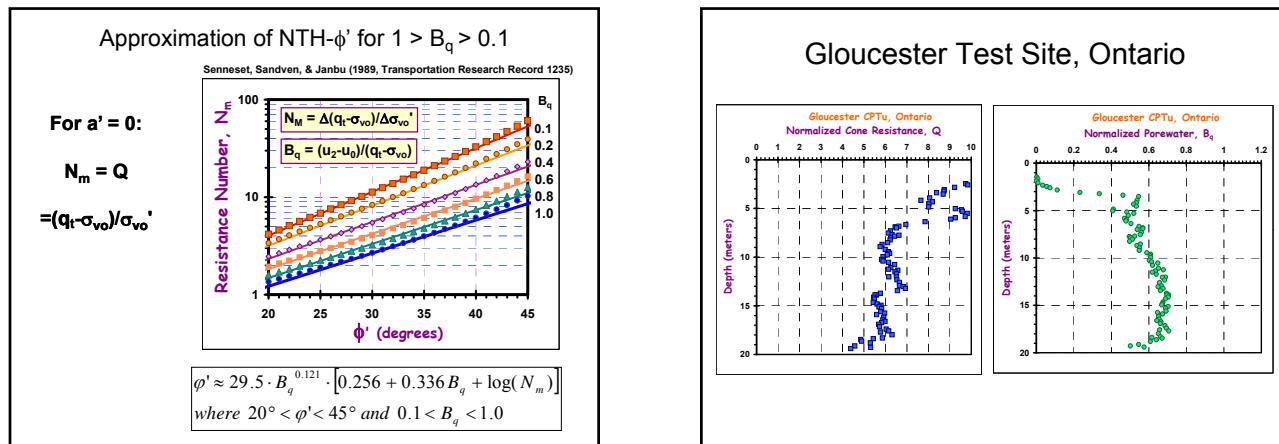
For all soil types



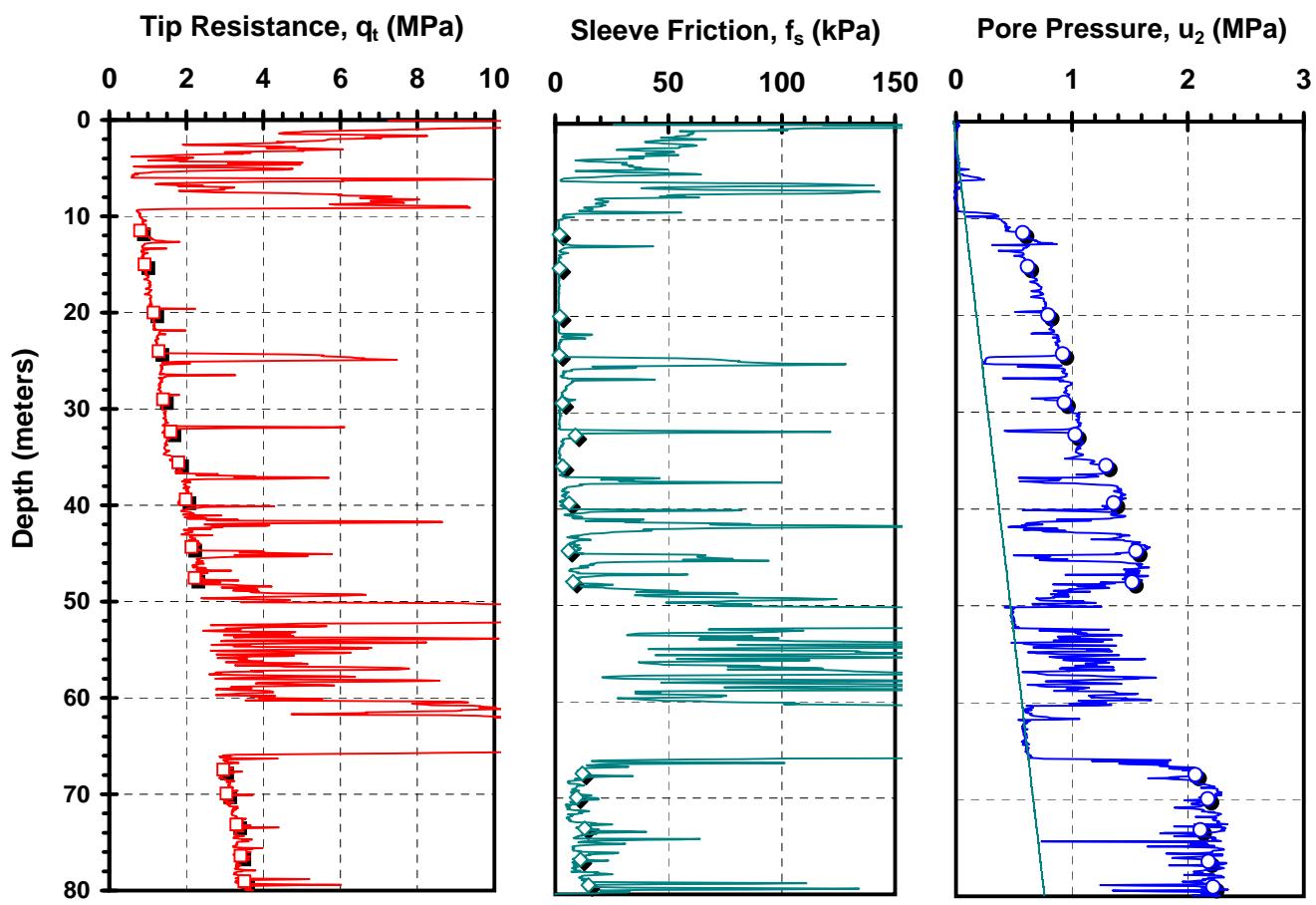
$$\phi' \approx 29.5 \cdot B_q^{0.121} \cdot [0.256 + 0.336 B_q + \log(N_m)]$$

where $20^\circ < \phi' < 45^\circ$ and $0.1 < B_q < 1.0$

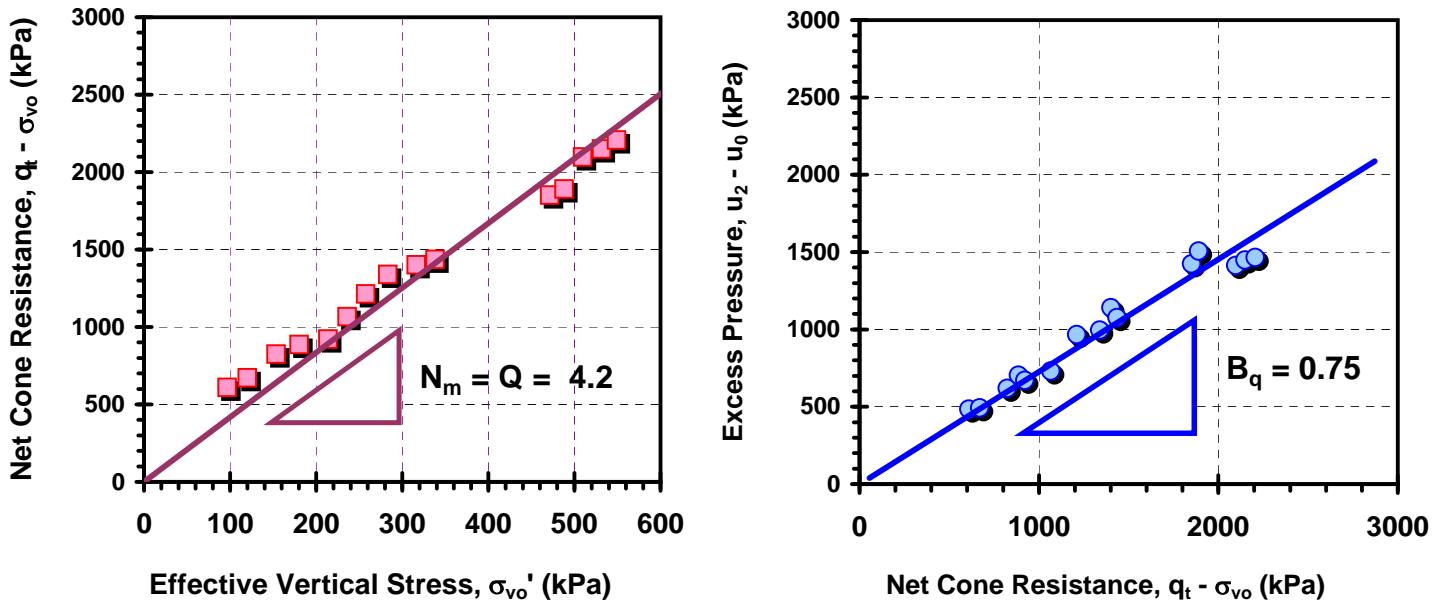


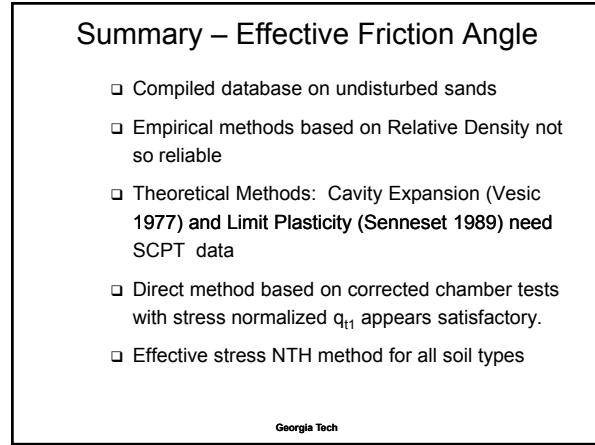
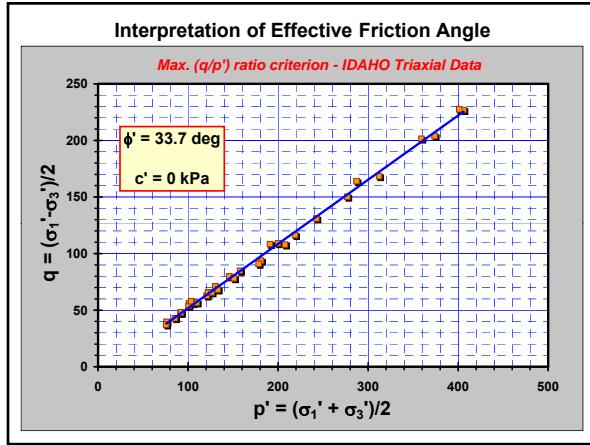
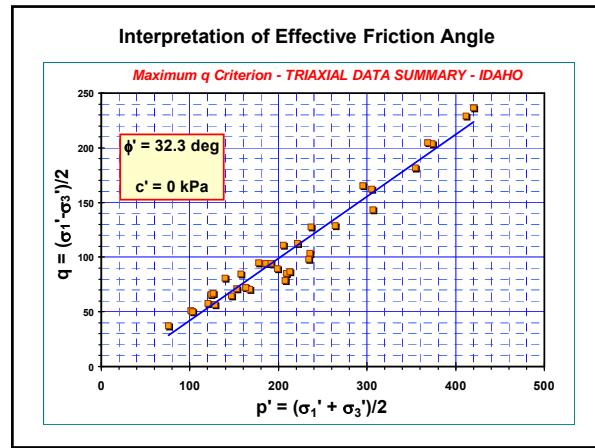
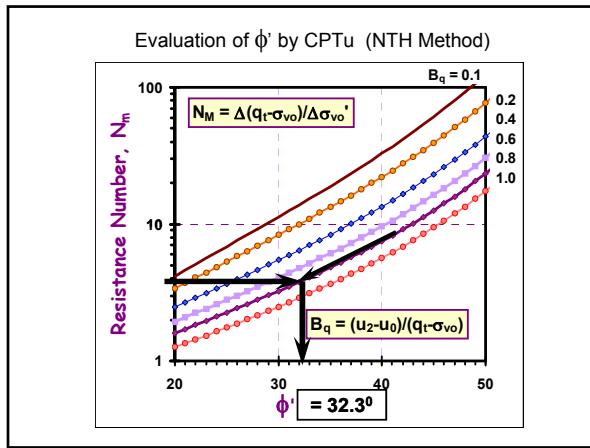


State Highway 95, Sandpoint Idaho - CPTu



NTH Processing for Effective Friction Angle (Seneset, et al. 1989, Transportation Research Record)





June 2008

Critical-State Soil Mechanics for Dummies

Paul W. Mayne
Georgia Institute of Technology

PROLOGUE

- Critical-state soil mechanics is an effective stress framework describing mechanical soil response
- In its simple form here, we consider only shear loading and compression-swelling.
- We merely tie together two well-known concepts: (1) one-dimensional consolidation behavior (via e -log σ' curves); and (2) shear stress-vs. normal stress (τ - σ'_v) plots from direct shear (alias Mohr's circles).

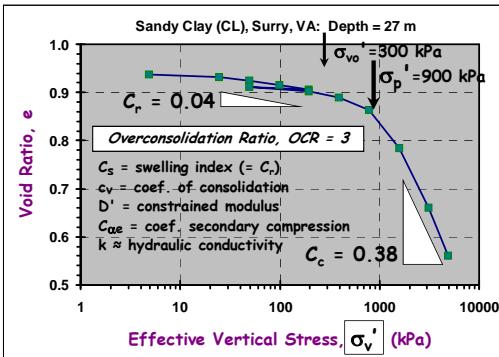
Critical State Soil Mechanics (CSSM)

- Experimental evidence
 - 1936 by Hvorslev (1960, ASCE)
 - Henkel (1960, ASCE Boulder)
 - Parry (1961)
 - Kulhawy & Mayne (1990): Summary of 200+ soils
- Mathematics presented elsewhere
 - Schofield & Wroth (1968)
 - Roscoe & Burland (1968)
 - Wood (1990)
 - Jefferies & Been (2006)
- Basic form: 3 material constants (ϕ' , C_c , C_s) plus initial state parameter (e_0 , σ'_{vo} , OCR)

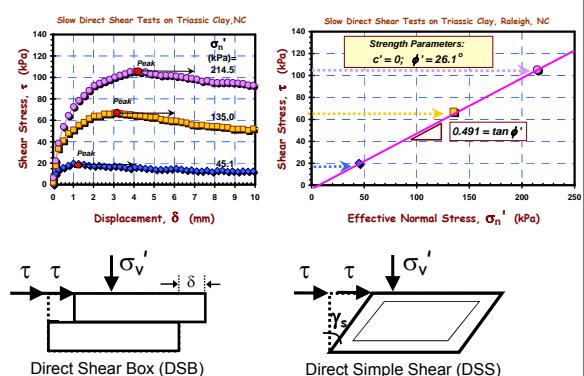
Critical State Soil Mechanics (CSSM)

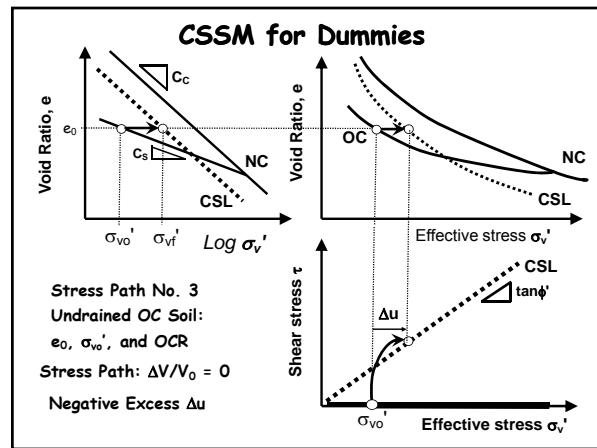
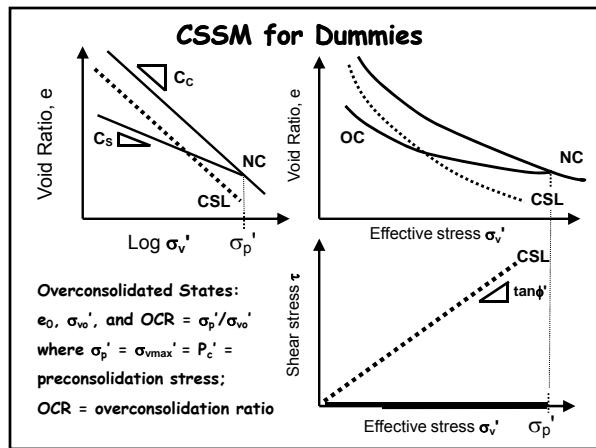
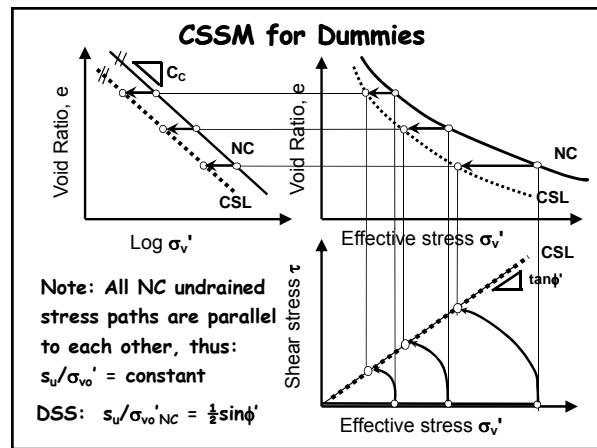
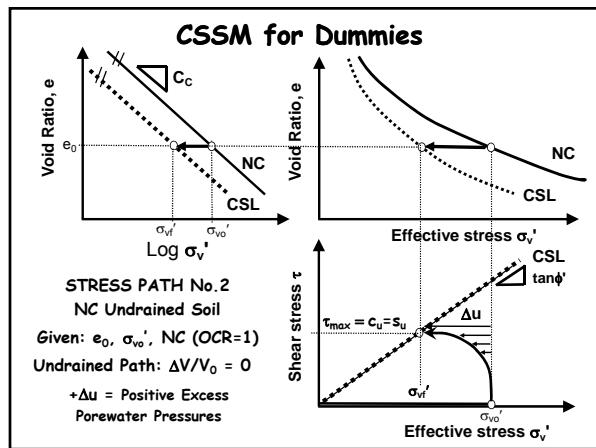
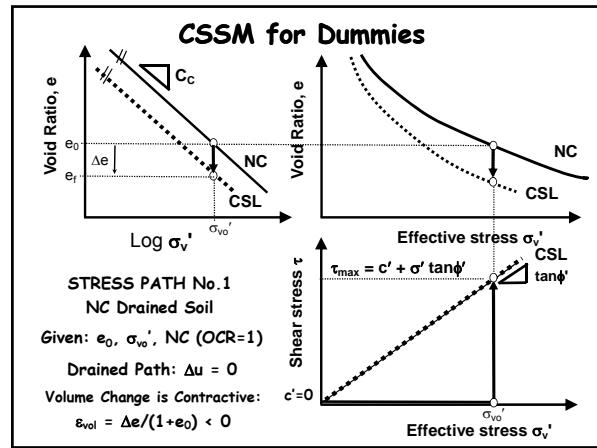
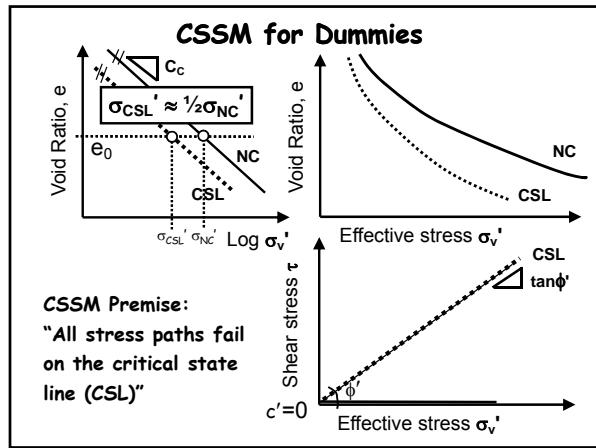
- Constitutive Models in FEM packages, including: Original Cam-Clay, Modified Cam Clay, NorSand, Bounding Surface, MIT-E3 (Whittle, 1993) & MIT-S1 (Pestana), Cap Model, "Ber-Klay", and others (Adachi, Oka, Ohta, Dafalias, Nova, Wood, Huerkel)
- "Undrained" is just one specific stress path
- Yet !!! CSSM is missing from most textbooks and undergrad & grad curricula.

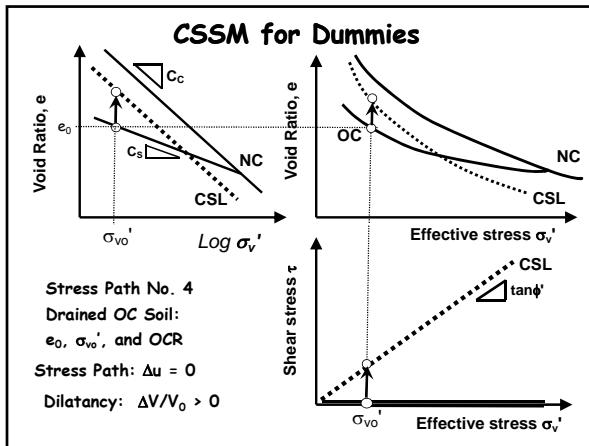
One-Dimensional Consolidation



Direct Shear Test Results



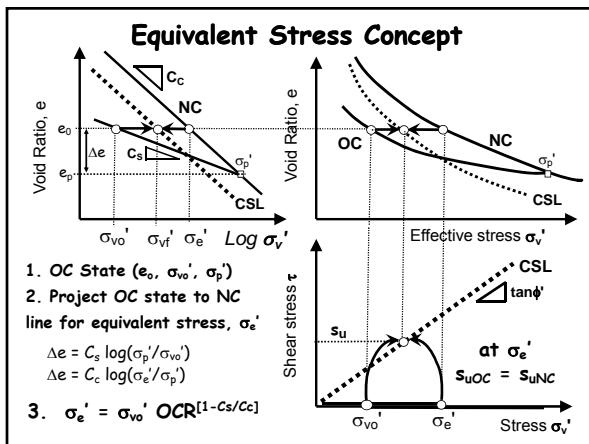




Critical state soil mechanics

- Initial state: e_0 , σ_{vo}' , and $OCR = \sigma_p'/\sigma_{vo}'$
- Soil constants: ϕ' , C_c , and C_s ($\Lambda = 1 - C_s/C_c$)
- For NC soil ($OCR = 1$):
 - Undrained ($\epsilon_{vol} = 0$): $+\Delta u$ and $\tau_{max} = s_u = c_u$
 - Drained ($\Delta u = 0$) and contractive (decrease ϵ_{vol})
- For OC soil:
 - Undrained ($\epsilon_{vol} = 0$): $-\Delta u$ and $\tau_{max} = s_u = c_u$
 - Drained ($\Delta u = 0$) and dilative (Increase ϵ_{vol})

There's more! Semi-drained, Partly undrained, Cyclic response.



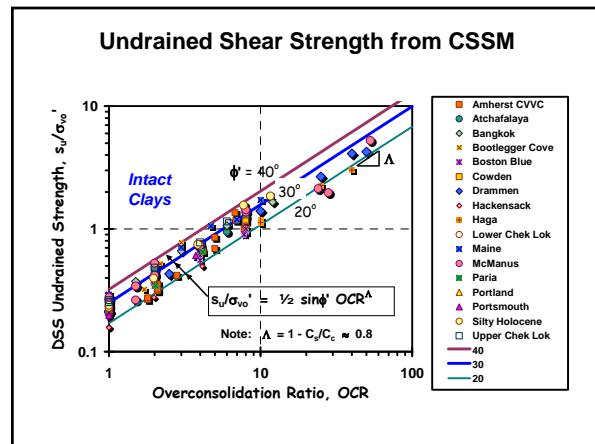
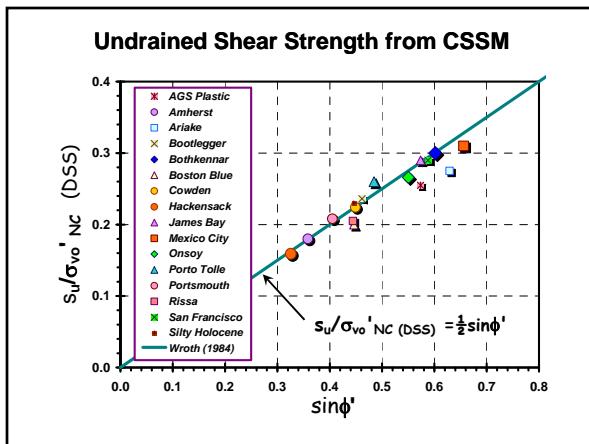
Critical state soil mechanics

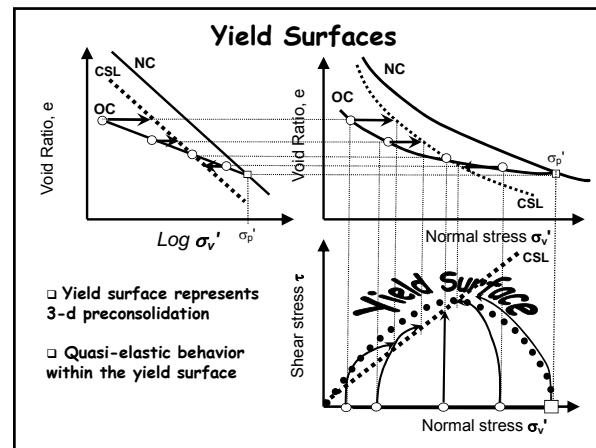
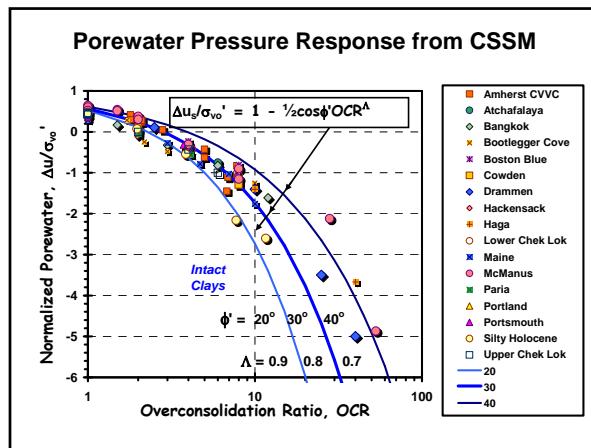
- Previously: $s_u/\sigma_{vo}' = \text{constant}$ for NC soil
- On the virgin compression line: $\sigma_{vo}' = \sigma_e'$
- Thus: $s_u/\sigma_e' = \text{constant}$ for all soil (NC & OC)
- For simple shear: $s_u/\sigma_e' = \frac{1}{2} \sin \phi'$
- Equivalent stress: $\sigma_e' = \sigma_{vo}' OCR^{[1-C_s/C_c]}$

Normalized Undrained Shear Strength:

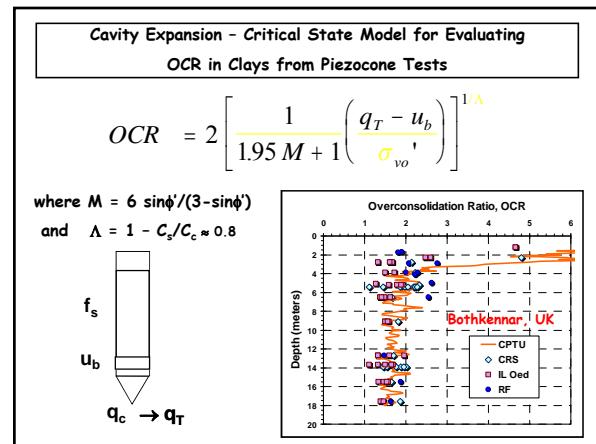
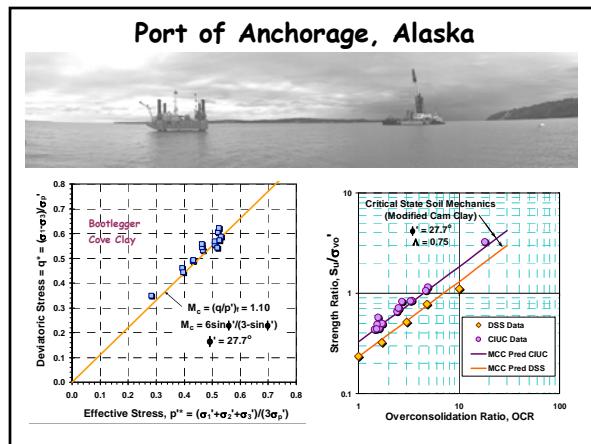
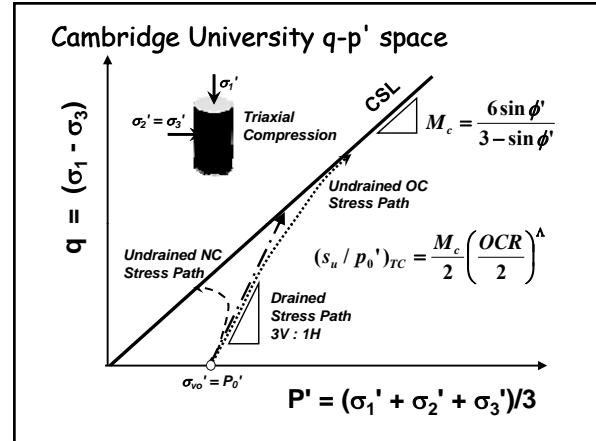
$$s_u/\sigma_{vo}' = \frac{1}{2} \sin \phi' OCR^\Lambda$$

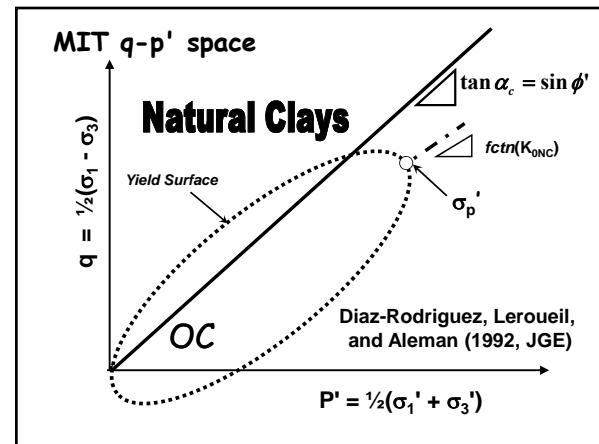
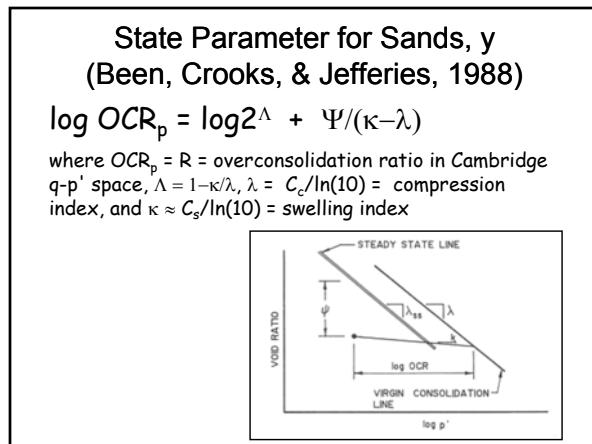
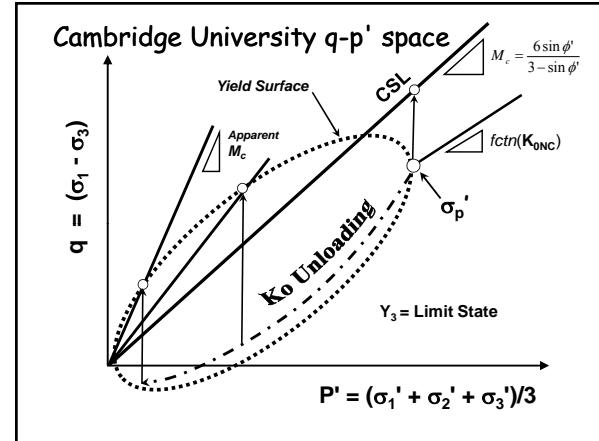
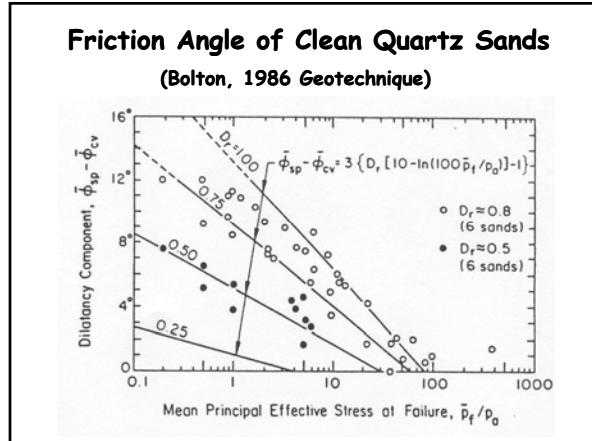
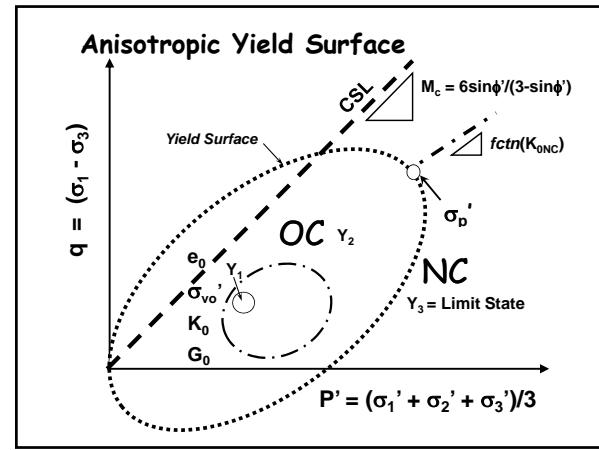
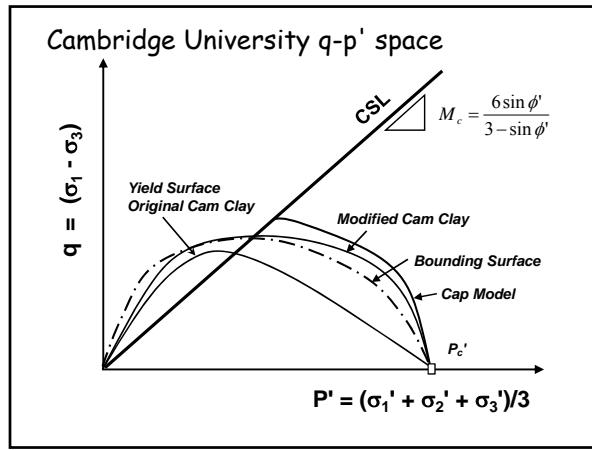
$$\text{where } \Lambda = (1 - C_s/C_c)$$





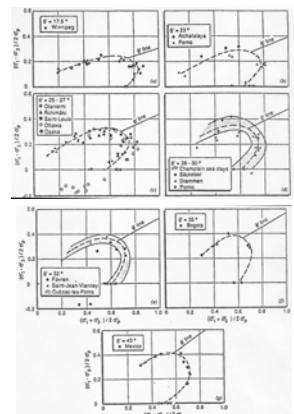
- Critical state soil mechanics**
- This powerpoint: www.ce.gatech.edu/~geosys
 - Classic book: *Critical-State Soil Mechanics* by Schofield & Wroth (1968): <http://www.geotechnique.info>
 - Schofield (2005) *Disturbed Soil Properties and Geotechnical Design* Thomas Telford
 - Wood (1990): *Soil Behaviour and CSSM*
 - Jefferies & Been (2006): *Soil liquefaction: a critical-state approach* www.informaworld.com





Yield Surfaces of Natural Clays

Diaz-Rodriguez,
Leroueil, & Aleman
(ASCE Journal
Geotechnical
Engineering July 1992)



Critical state soil mechanics

- Initial state: e_0 , σ_{vo}' , and $OCR = \sigma_p'/\sigma_{vo}'$
- Soil constants: ϕ' , C_c , and C_s
- Link between **Consolidation** and **Shear Tests**
- CSSM addresses:
 - NC and OC behavior
 - Undrained vs. Drained (and other paths)
 - Positive vs. negative porewater pressures
 - Volume changes (contractive vs. dilative)
 - $s_u/\sigma_{vo}' = \frac{1}{2} \sin\phi' OCR^\Lambda$ where $\Lambda = 1 - C_s/C_c$
- Yield surface represents 3-d preconsolidation

Profiling Overconsolidation Ratio in Clays by Piezocone

Paul W. Mayne

Georgia Tech

OCR Evaluation in Clay from Piezocone Tests

- Theoretical Basis: Cavity Expansion + Critical State Soil Mechanics
- Type 1 and 2 piezocone elements.
- Laboratory Chamber Tests
- Calibration with Field Piezocone Data and Test Sites with known preconsolidation stresses from oedometer tests on high-quality samples

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Oedometer Testing (one-dimensional consolidation)



Pneumatic Type Consolidometers

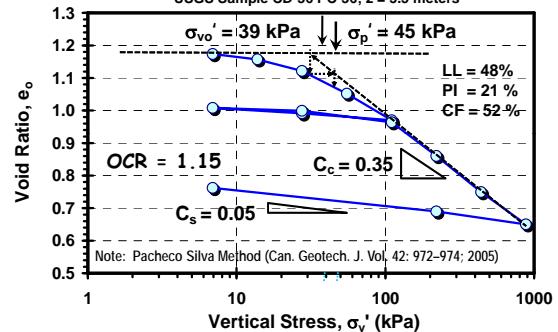
Moment Arm Device

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Lab Consolidation Test Results

Offshore Atlantic Continental Slope

USGS Sample CD-36 PC-36; z = 5.3 meters

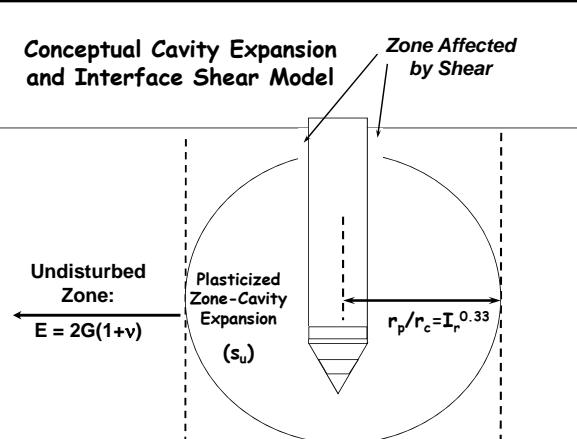


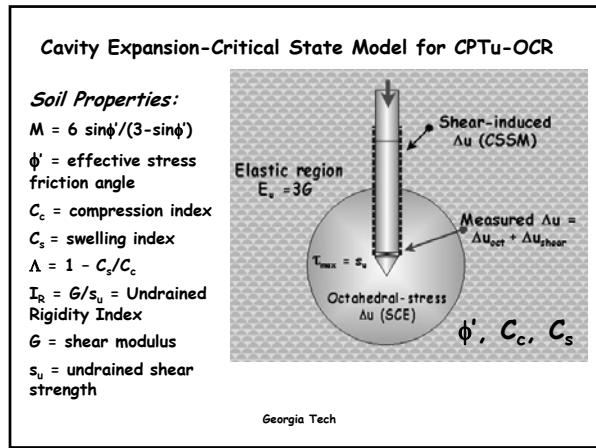
OCR Profiling in Clays

- Preconsolidation stress conventionally determined from 1-d consolidation or oedometer testing
- Limited amounts because of sampling issues + high costs
- Considerable times for standard incremental load testing (2 weeks/test)
- Can in-situ tests be used as supplement?

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Conceptual Cavity Expansion and Interface Shear Model





Cavity Expansion-Critical State Model

$$\text{Cone Tip Resistance: } q_t - \sigma_{vo} = N_{kt} s_u$$

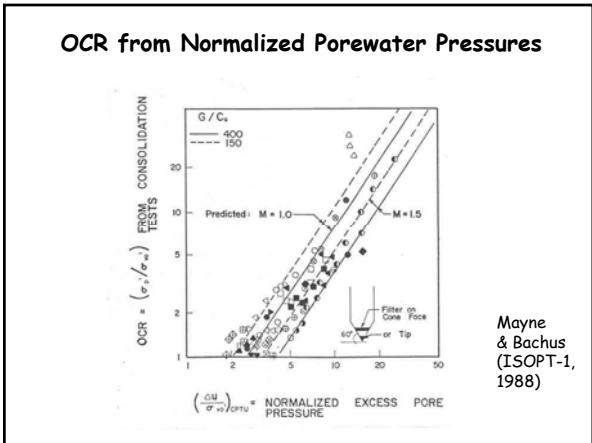
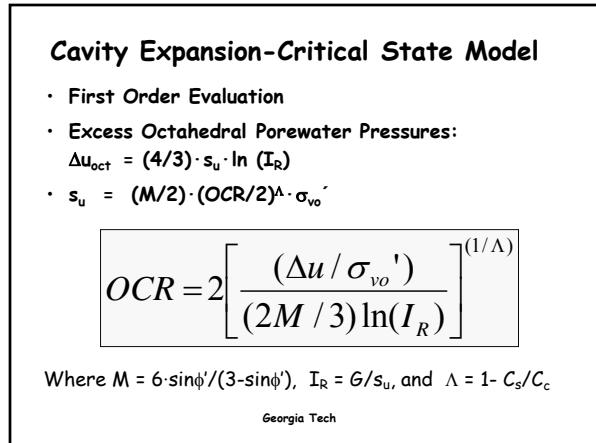
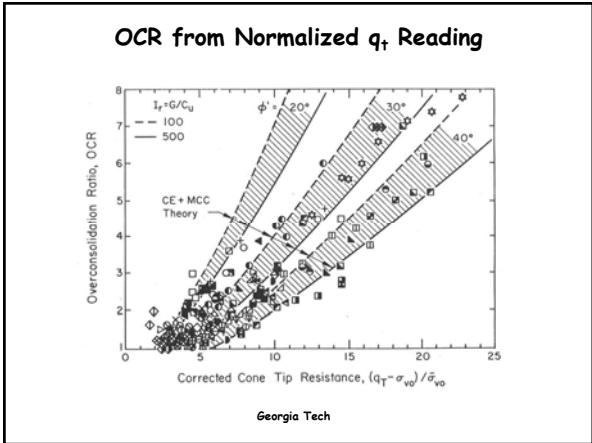
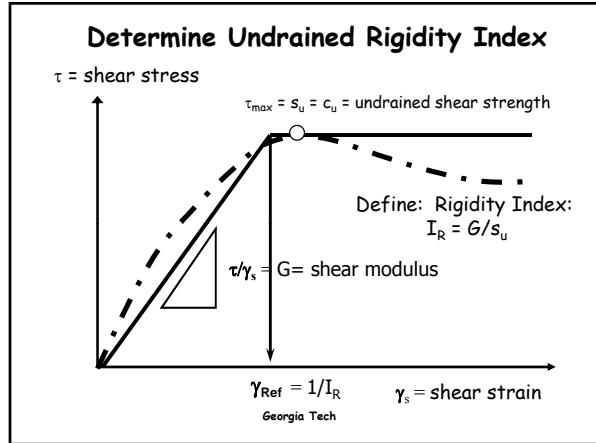
$$\cdot N_{kt} = (4/3) \cdot [\ln(I_R) + 1] + \pi/2 + 1$$

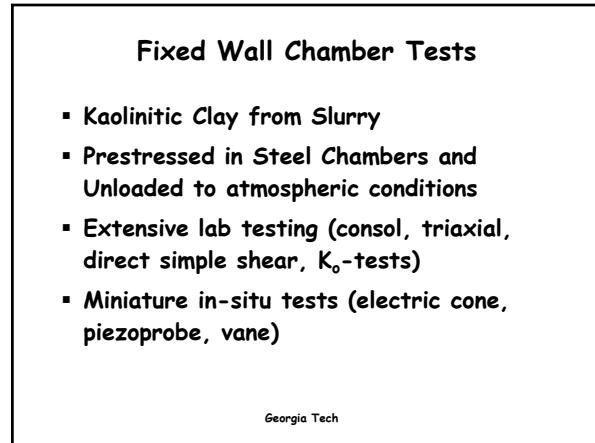
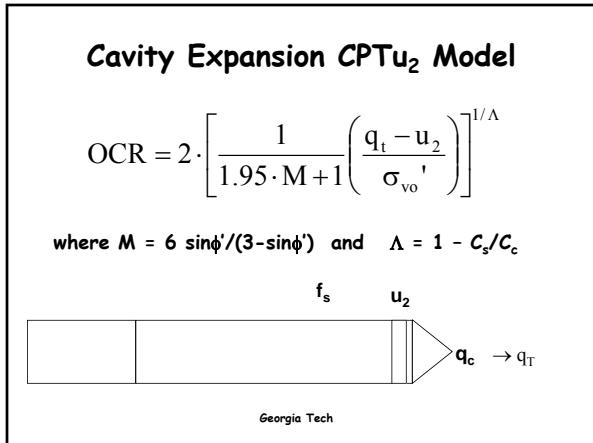
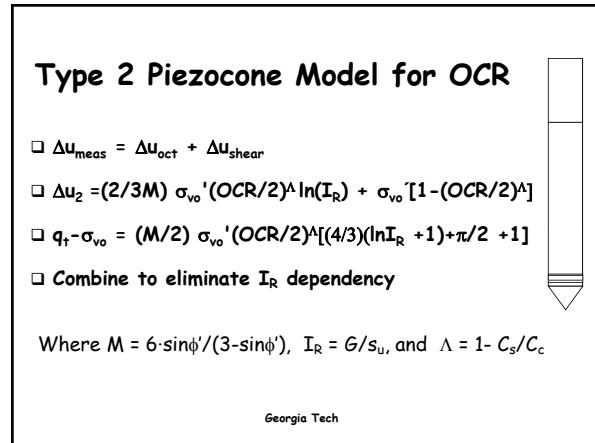
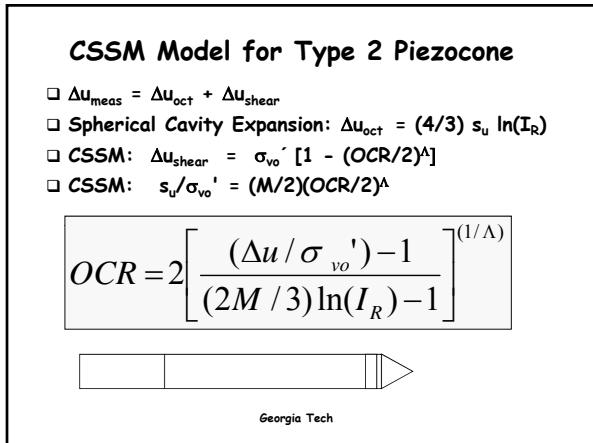
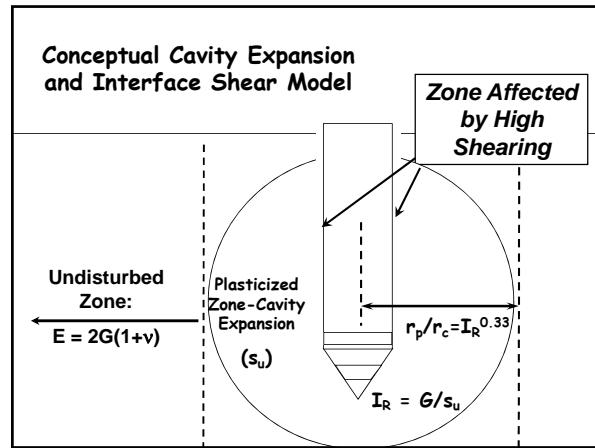
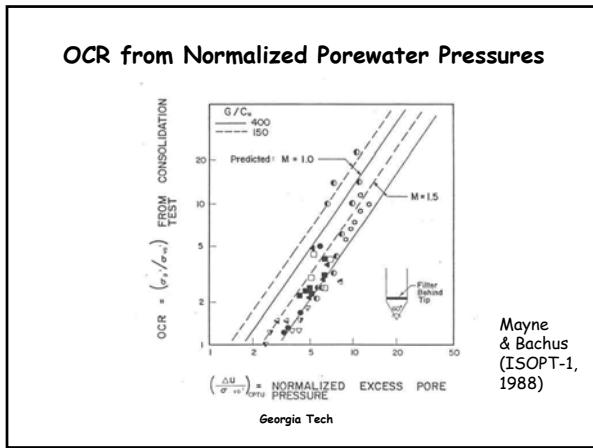
$$\cdot s_u = (M/2) \cdot (OCR/2)^\Lambda \cdot \sigma_{vo}'$$

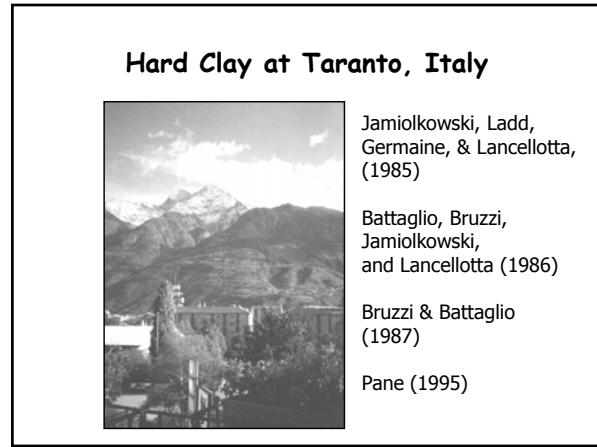
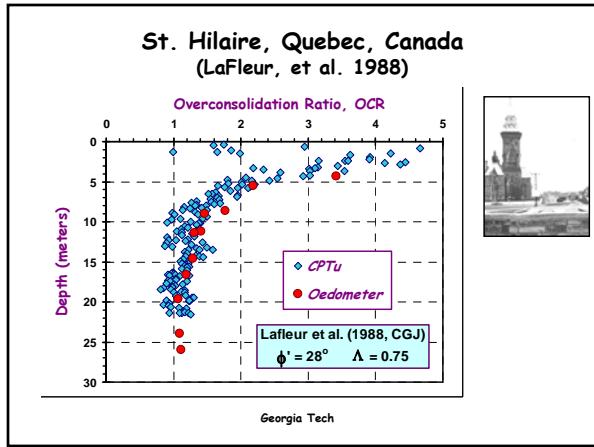
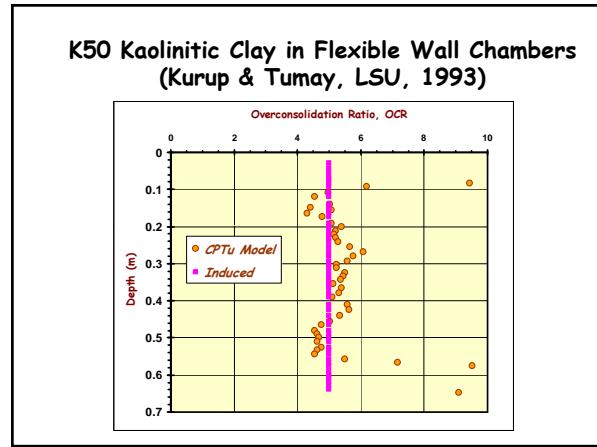
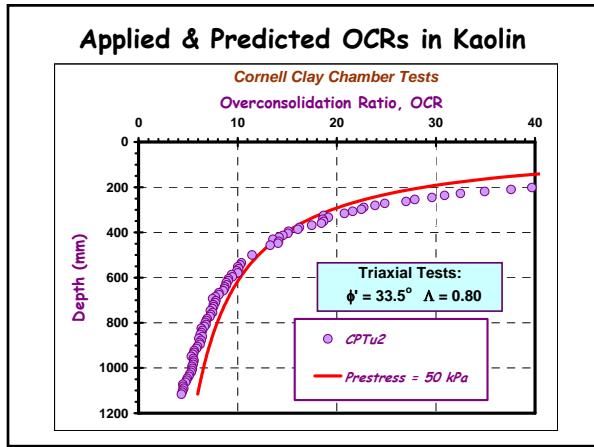
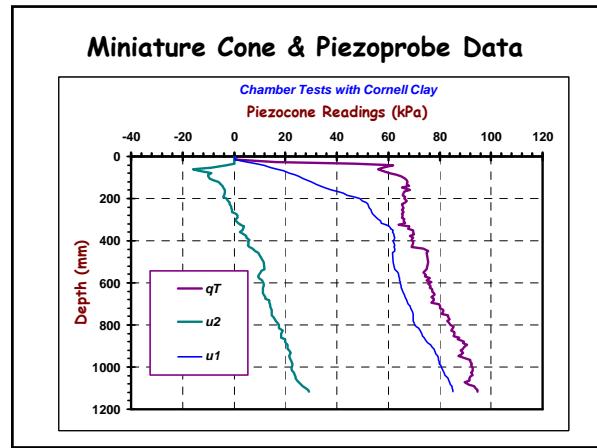
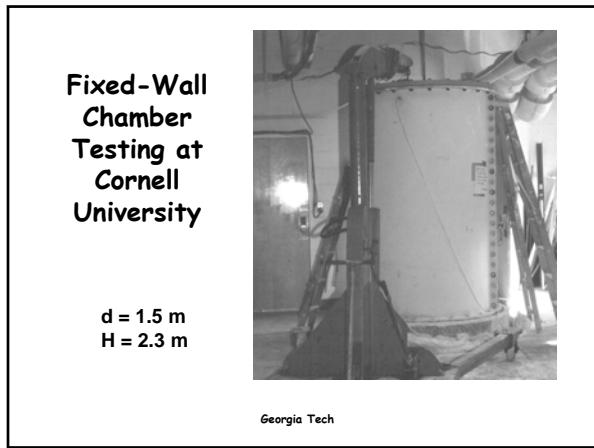
$$OCR = 2 \left[\frac{(2/M)(q_t - \sigma_{vo}')/\sigma_{vo}'}{(4/3)(\ln I_R + 1) + \pi/2 + 1} \right]^{(1/\Lambda)}$$

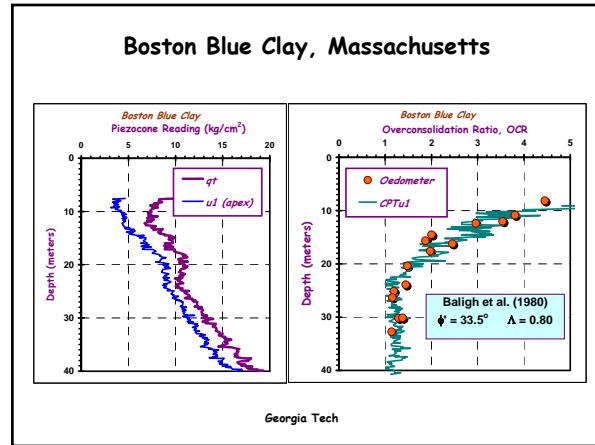
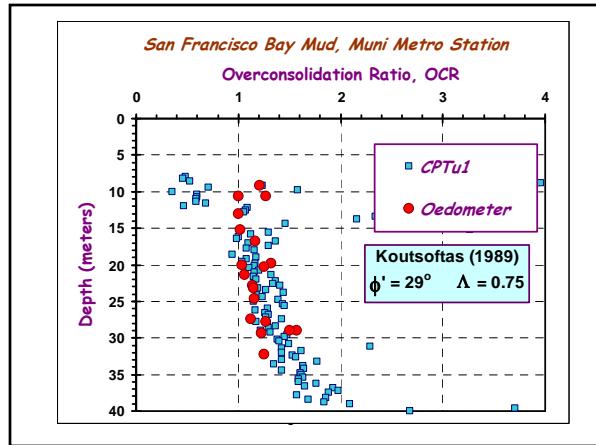
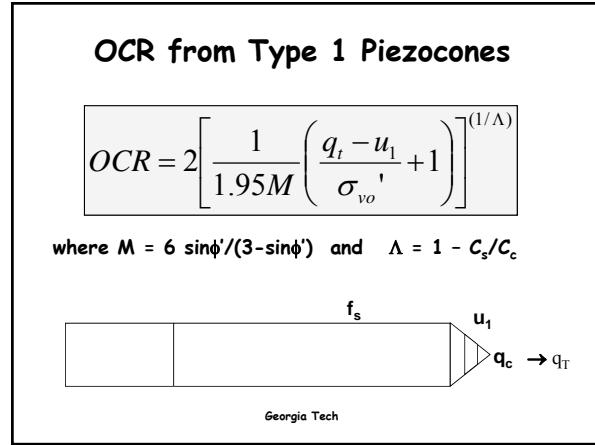
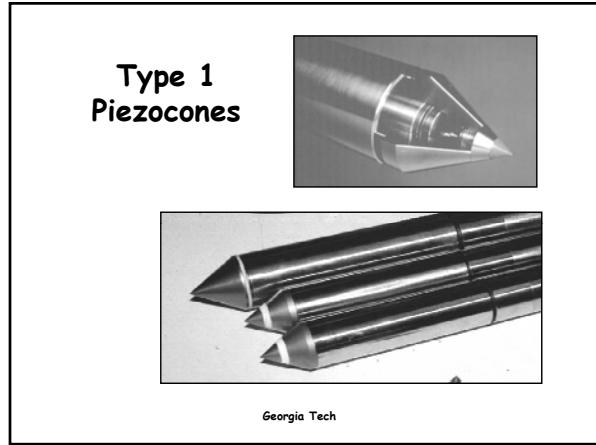
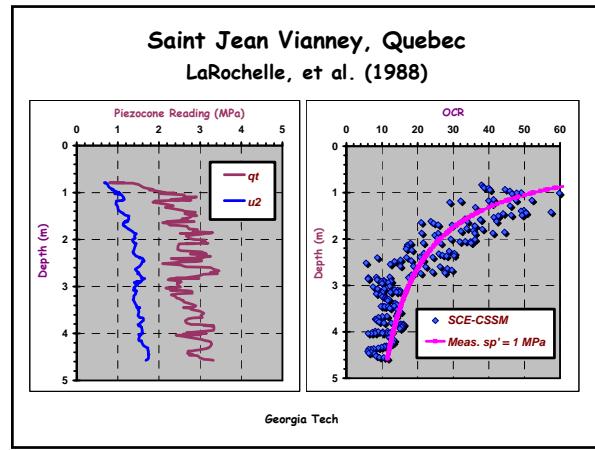
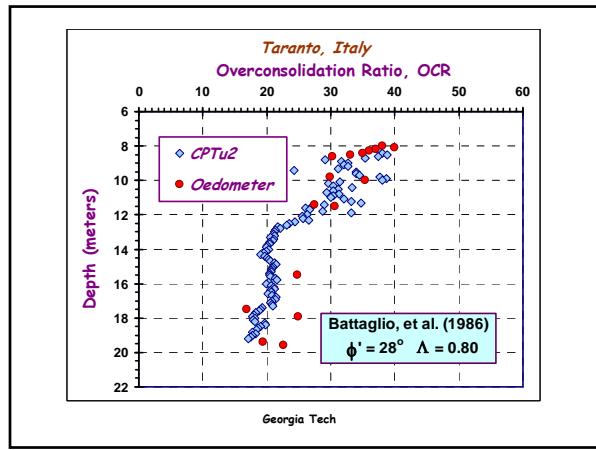
Where $M = 6 \cdot \sin\phi' / (3 - \sin\phi')$, $I_R = G/s_u$, and $\Lambda = 1 - C_s/C_c$

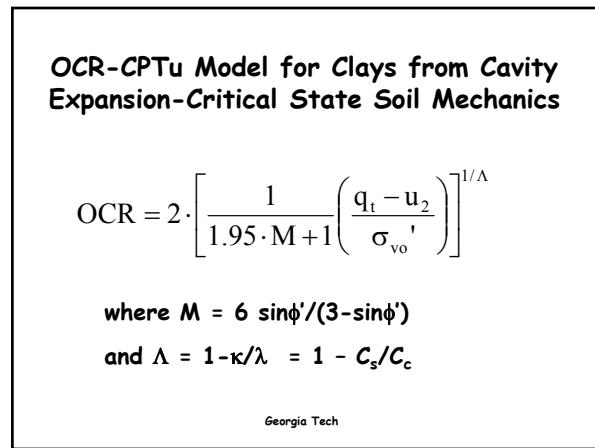
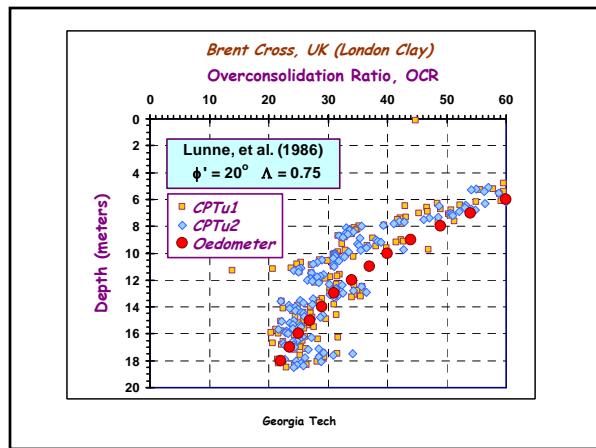
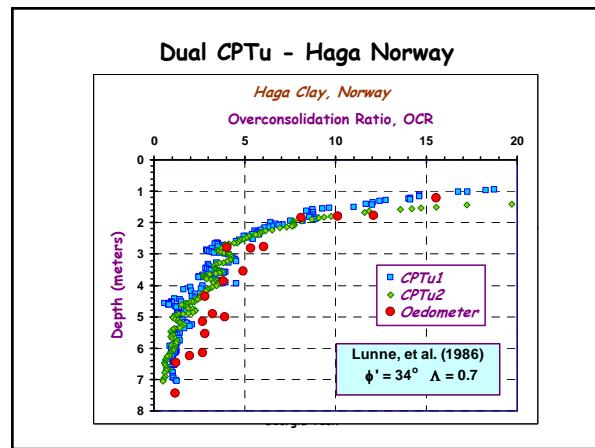
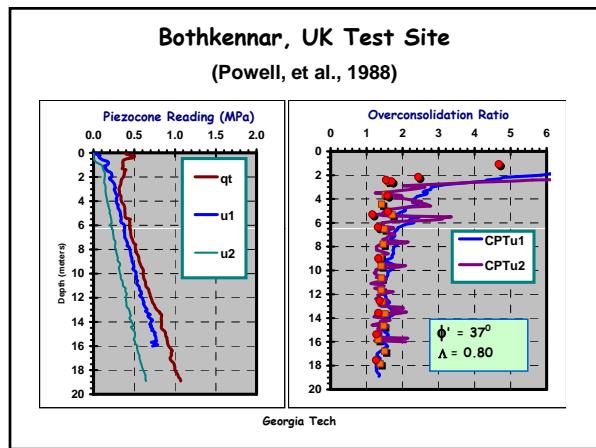
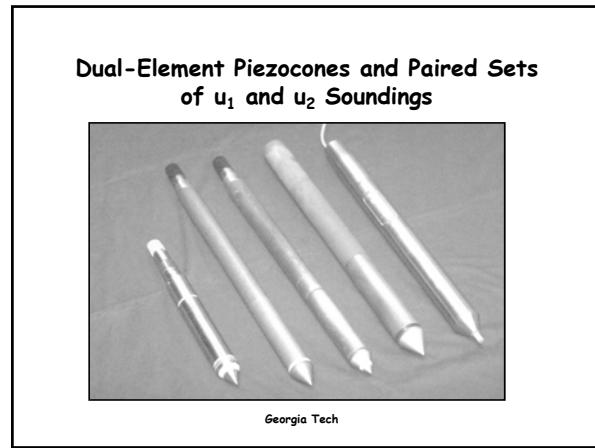
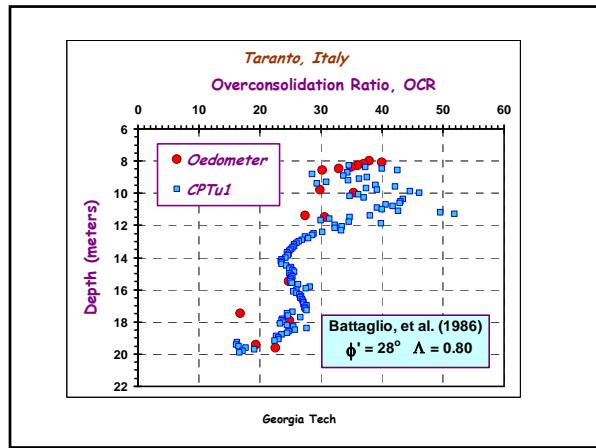
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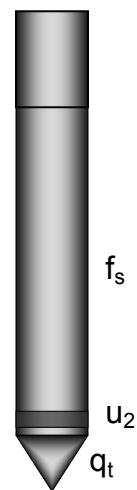


SCE-CSSM Analytical Method

$$OCR = 2 \left[\frac{(2/M)(q_t - \sigma_{vo}') / \sigma_{vo}'}{(4/3)(\ln I_R + 1) + \pi/2 + 1} \right]^{(1/\Lambda)}$$

$$OCR = 2 \left[\frac{(\Delta u_2 / \sigma_{vo}')}{(2M/3)\ln(I_R)} \right]^{(1/\Lambda)}$$

$$OCR = 2 \cdot \left[\frac{1}{1.95M+1} \left(\frac{q_t - u_2}{\sigma_{vo}'} \right) \right]^{1/\Lambda}$$



$$I_R = \exp \left[\left(\frac{1.5}{M} + 2.925 \right) \left(\frac{q_t - \sigma_{vo}}{q_t - u_2} - 2.925 \right) \right]$$

$$\begin{aligned} M &= 6 \sin\phi' / (3 - \sin\phi') \\ \Lambda &= 1 - C_s/C_c \\ I_R &= G/s_u \end{aligned}$$

- Reference: GT Report to NSF (1994)
- Also: Mayne (1991), *Soils & Foundations*
- Mayne (1992), *Predictive Soil Mechanics*, Thomas Telford

School of Civil and Environmental Engineering

Profiling the Overconsolidation Ratio of Clays by Piezocone Tests

Barry Shiyo Chen, PhD, P.E.

Paul W. Mayne, PhD, P.E.

National Science Foundation

August 1994

Report No. GIT-CEE GEO-94-1

SCE-CSSM Analytical Method

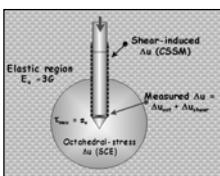
$$OCR = 2 \left[\frac{(2/M)(q_t - \sigma_{vo}')/\sigma_{vo}'}{(4/3)(\ln I_R + 1) + \pi/2 + 1} \right]^{(1/\Lambda)}$$

$$OCR = 2 \left[\frac{(\Delta u / \sigma_{vo}')}{(2M/3)\ln(I_R)} \right]^{(1/\Lambda)}$$

$$OCR = 2 \left[\frac{1}{1.95M + 1} \left(\frac{q_t - u_b}{\sigma_{vo}'} \right) \right]^{(1/\Lambda)}$$

$$I_R = \exp \left[\left(\frac{1.5}{M} + 2.925 \right) \left(\frac{q_t - \sigma_{vo}}{q_t - u_2} - 2.925 \right) \right]$$

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Conclusions

- Theoretical basis for relating OCR to CPTu normalized parameters:

- Cone Tip Resistance: $(q_t - \sigma_{vo})/\sigma_{vo}'$

- Porewater Pressures: $U^* = \Delta u/\sigma_{vo}'$

- Hybrid Cavity Expansion + Critical State Soil Mechanics shows function of effective friction angle ϕ' , $\Lambda = 1 - C_s/C_c$, and rigidity index, $I_R = G/s_u$

- Calibration with Well-Documented Sites

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**GT Report by
Chen & Mayne
(1994)
to NSF**



Georgia Tech

Overconsolidation Ratio of Clays From Flat Dilatometer Tests

Paul W. Mayne
Geosystems Engineering

Georgia Tech

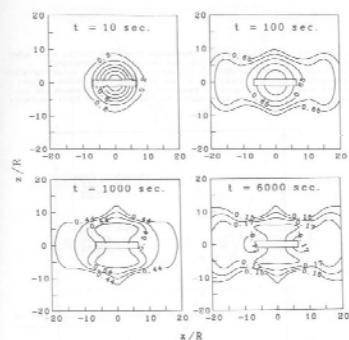
Cavity Expansion-Critical State Method for Flat Dilatometer in Clays

Essentially, the lift-off pressure is dominated by porewater effects induced during penetration

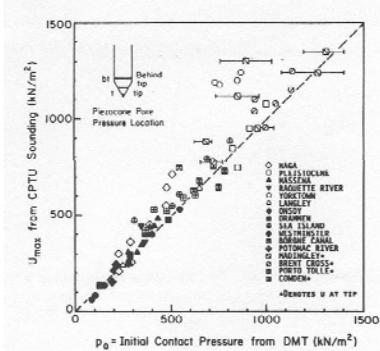
$$p_0 \approx u_{\max}$$

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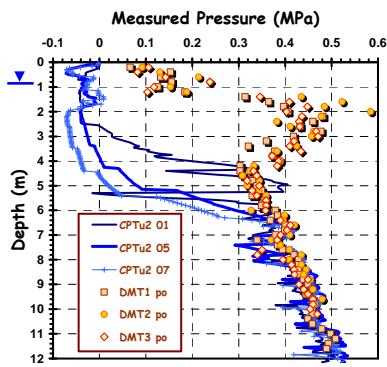
Instrumented DMT Chamber Tests (Huang, 1991 - Calibration Chamber Testing, Elsevier)



CPTu-DMT Database: $p_0 \approx u_{\max}$



CPTu₂ and DMTs at Amherst NGES



Ford Design Center Northwestern University

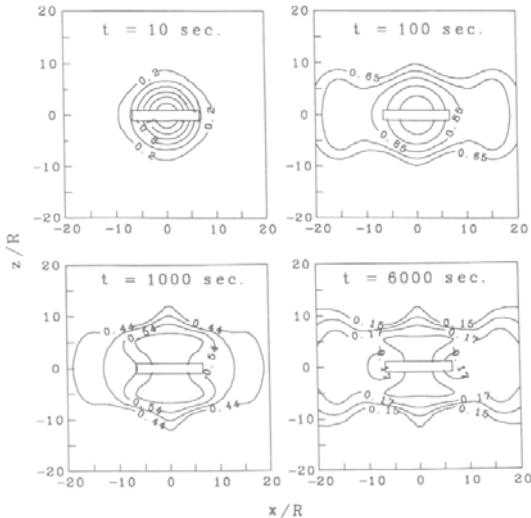


Near NGES Evanston, Illinois (Finno, et al. 2000)

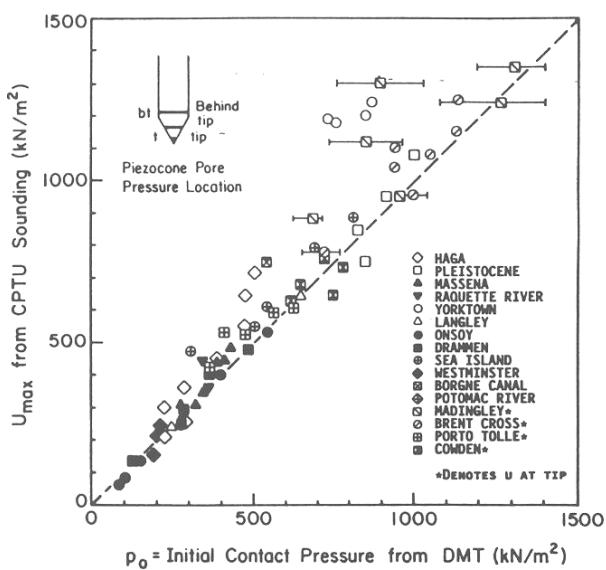
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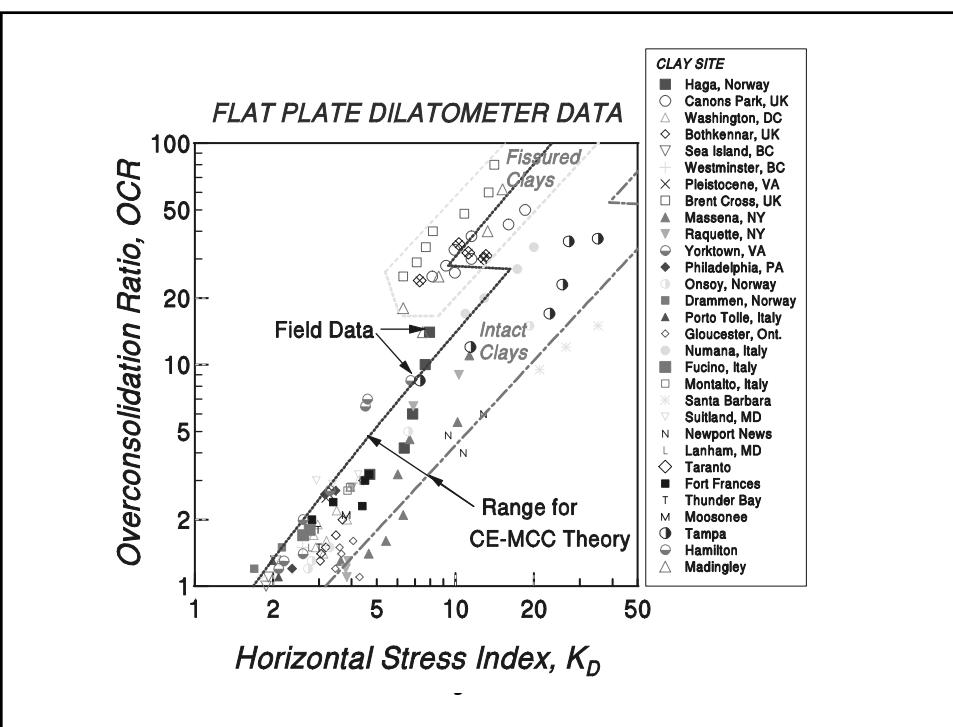
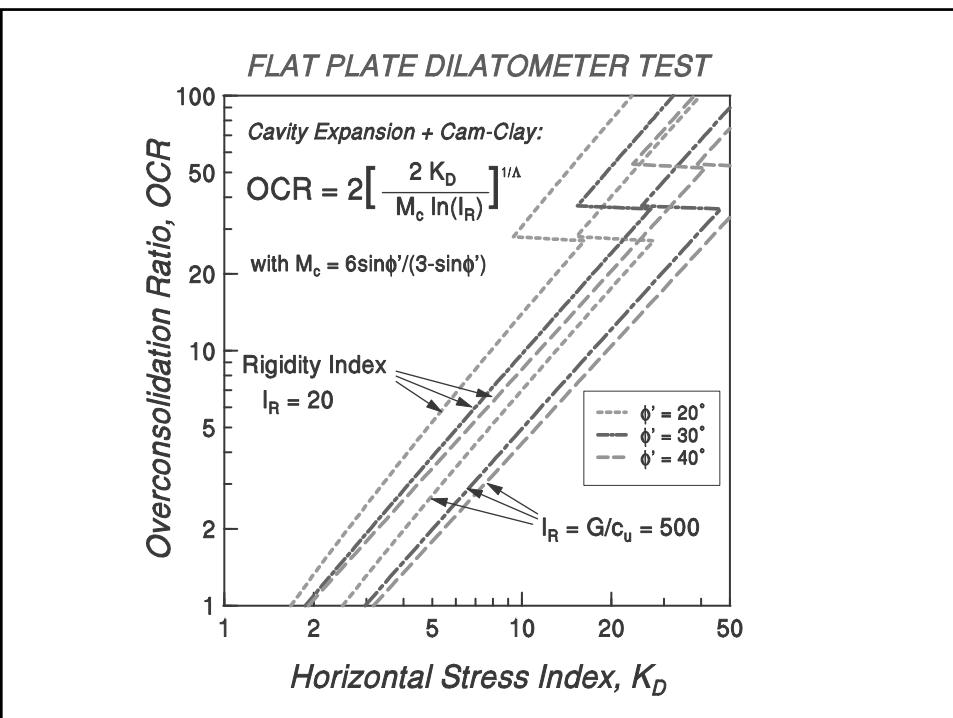
Instrumented DMT Chamber Tests in Clay

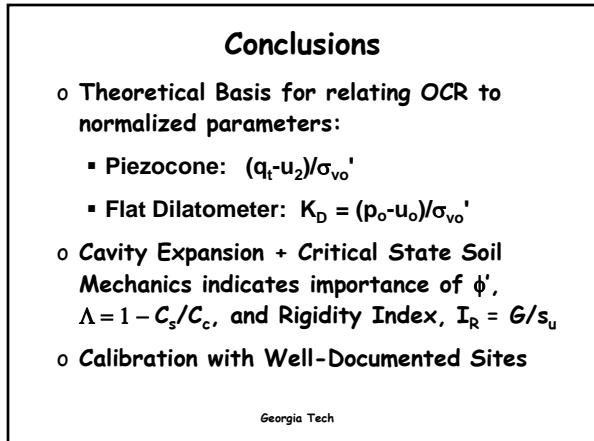
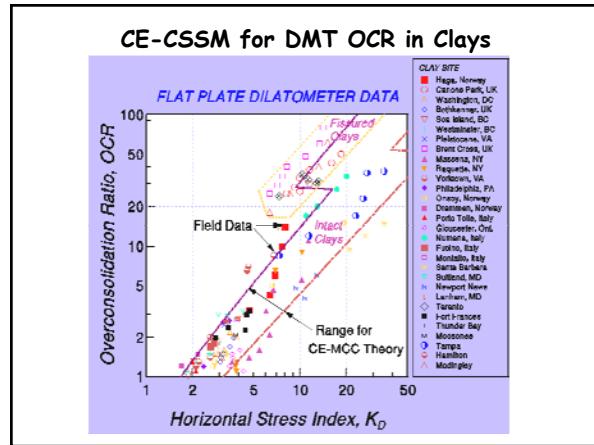
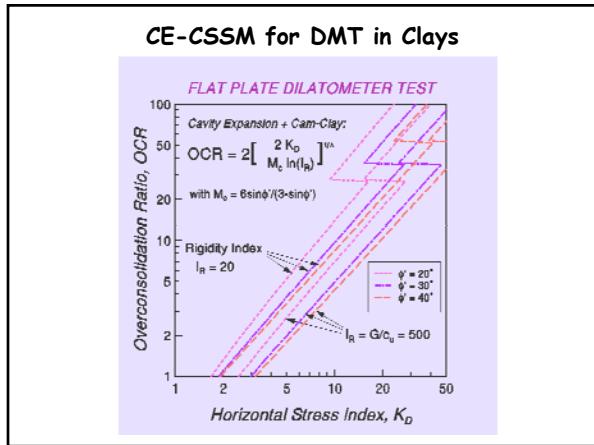
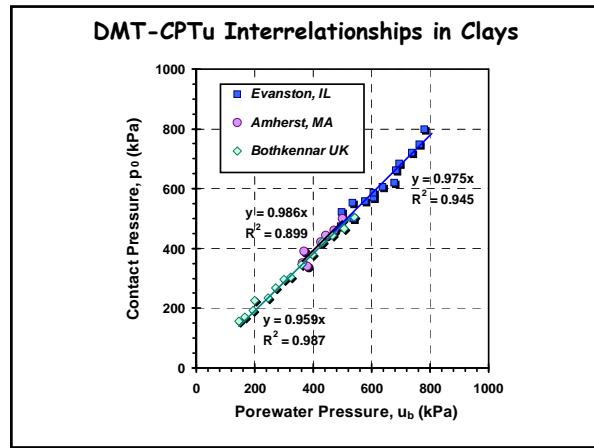
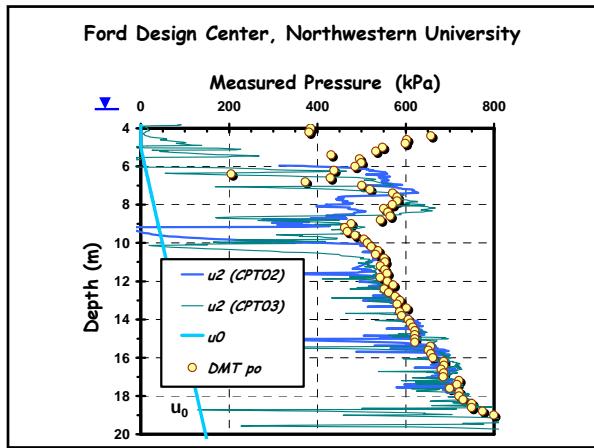
(Huang, ed. 1991 - *Calibration Chamber Testing*, Elsevier)



CPTu-DMT Database in Clays: $p_0 \approx u_{\text{meas}}$
 (Mayne & Bachus, 1989, ICSMFE, Vol. 1, Rio)





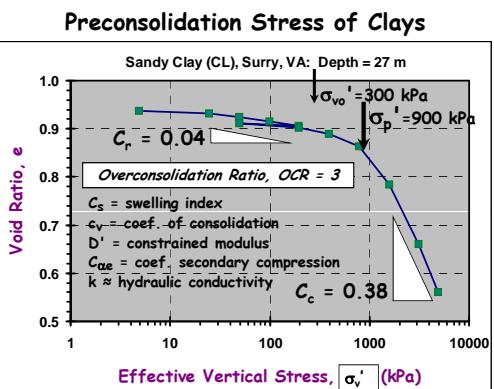


First-Order Evaluation of Preconsolidation Stress from In-Situ Tests (CPT, VST, V_s)

Paul W. Mayne
Georgia Institute of Technology

Stress History of Clays

- Preconsolidation stress ($P_c' = \sigma_{v\max}' = \sigma_p'$)
- Normalized form: $OCR = (\sigma_p'/\sigma_{vo}')$
- Preconsolidation is yield stress separating overconsolidated region ("elastic") from normally-consolidated states ("plastic")
- OCR controls the undrained shear strength in terms of s_u/σ_{vo}' ratio.
- Geostatic stress state: $K_0 - OCR$ relationships in soils.



Preconsolidation Stress of Clays

- Laboratory oedometer (one-dimensional consolidation) is reference test for σ_p'
- Incremental Load Steps (one per day) $\Rightarrow 2$ weeks per specimen
- Constant-rate-of-strain: 2 days
- Costs per test $\approx \$300$ to $\$500$
- Need several tests but only discrete points
- Supplement with in-situ test data

Preconsolidation from In-Situ Tests

- Theoretical approaches based on cavity expansion and critical state soil mechanics (CPT, CPTu, DMT)
- Numerical simulations using finite elements, strain path method, FLAC
- Empirical methods (statistical databases) including: VST, CPT, CPTu, DMT, PMT, SPT, V_s

$$OCR = 2 \left[\frac{\left(\frac{2}{M} \right) (q_t - \sigma_{vo}') / \sigma_{vo}'}{\left(\frac{4}{3} \right) (\ln I_R + 1) + \frac{\pi}{2} + 1} \right]^{(1/\Lambda)}$$

$$\Rightarrow \sigma_p' = \frac{(q_t - \sigma_{vo}')}{M \cdot [1 + \frac{1}{3} \ln(I_R)]} \Rightarrow \sigma_p' = 0.33 \cdot (q_t - \sigma_{vo}')$$

$$OCR = 2 \left[\frac{(\Delta u / \sigma_{vo}')}{(\frac{2}{3} M) \ln(I_R)} \right]^{(1/\Lambda)}$$

$$\Rightarrow \sigma_p' = \frac{(\Delta u_b - u_b)}{\frac{M}{3} \ln(I_R)} \Rightarrow \sigma_p' = 0.54 \cdot (\Delta u_b)$$

$$OCR = 2 \left[\frac{1}{1.95 M + 1} \left(\frac{q_t - u_b}{\sigma_{vo}'} \right)^{\frac{1}{\Lambda}} \right]$$

$$\Rightarrow \sigma_p' = \frac{(q_t - u_b)}{0.975 M + \frac{1}{2}} \Rightarrow \sigma_p' = 0.60 \cdot (q_t - u_b)$$

SCE-CSSM Theory

For $\Lambda = 1$ $\phi' = 30^\circ$ $I_R = 100$

First-Order Approximations

SCE-CSSM Theory

$$OCR = 2 \left[\frac{\left(\frac{2}{M} \right) (q_t - \sigma_{vo}) / \sigma_{vo}}{\left(\frac{4}{3} \right) (\ln I_R + 1) + \frac{\pi}{2} + 1} \right]^{(1/\Lambda)}$$

For $\Lambda = 1$ $\phi' = 30^\circ$ $I_R = 100$

$$\rightarrow \sigma'_p = \frac{(q_t - \sigma_{vo})}{M \cdot [1 + \frac{1}{3} \ln(I_R)]} \rightarrow \sigma'_p = 0.33 \cdot (q_t - \sigma_{vo})$$

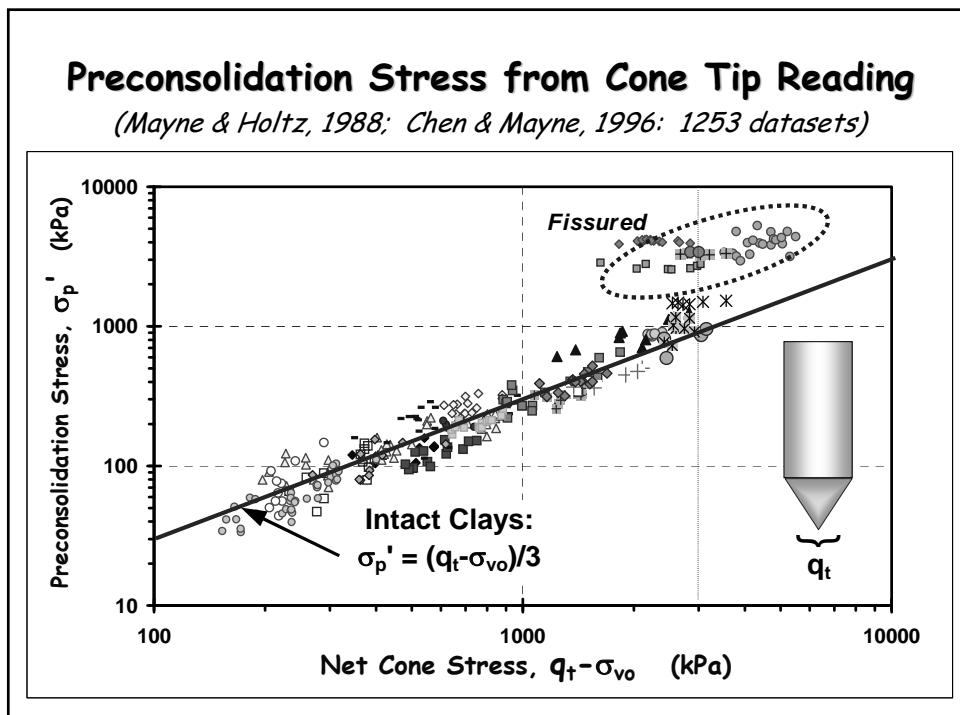
First-Order Approximations

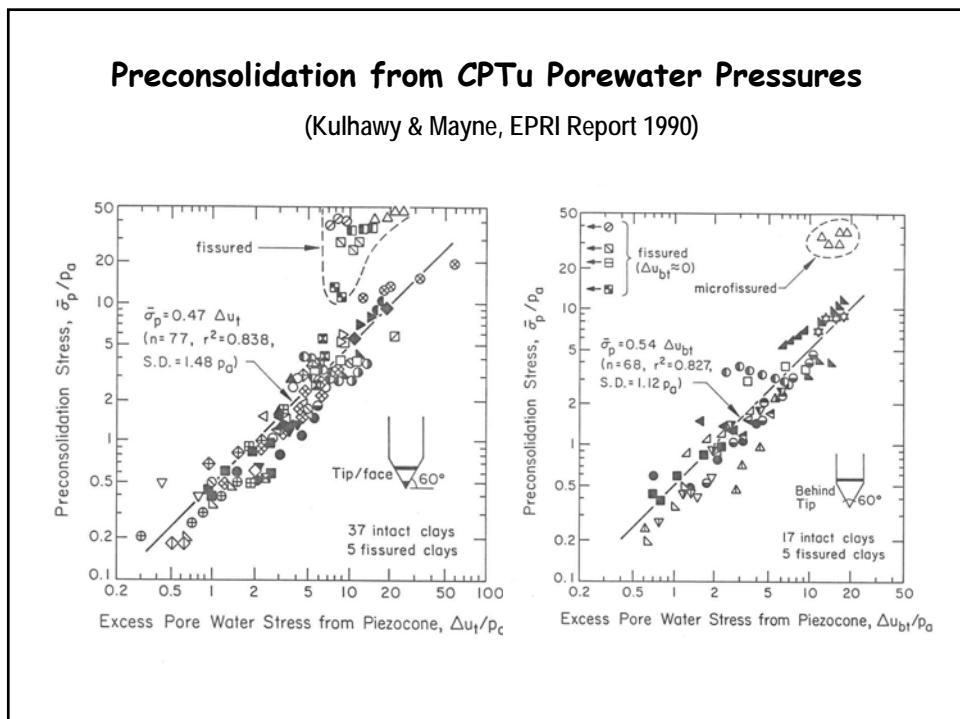
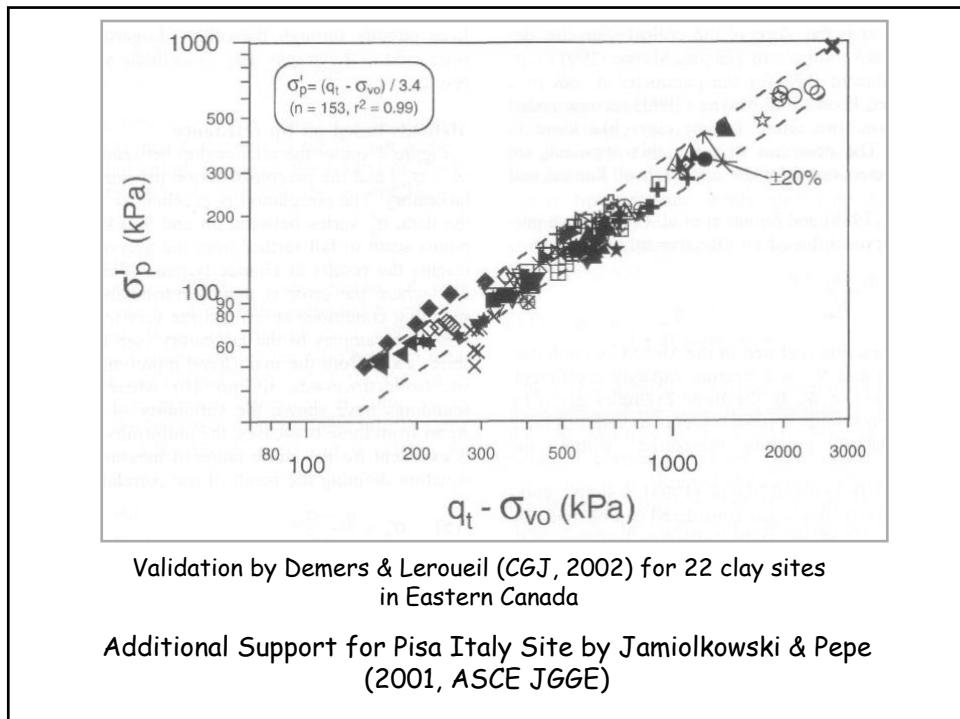
$$OCR = 2 \left[\frac{(\Delta u / \sigma'_{vo})}{\left(\frac{2}{3} M \right) \ln(I_R)} \right]^{(1/\Lambda)}$$

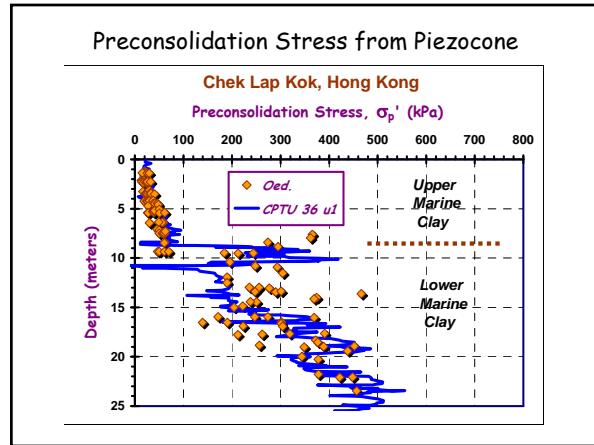
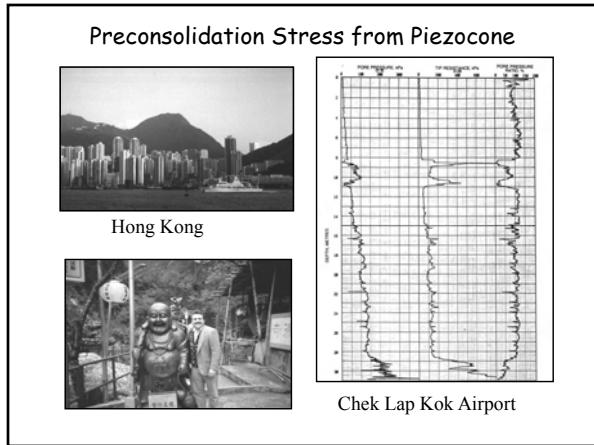
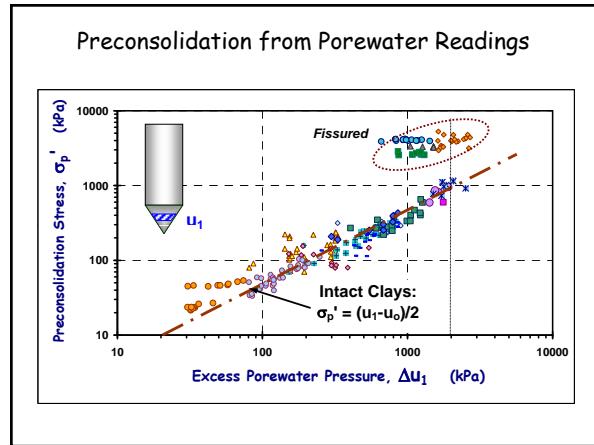
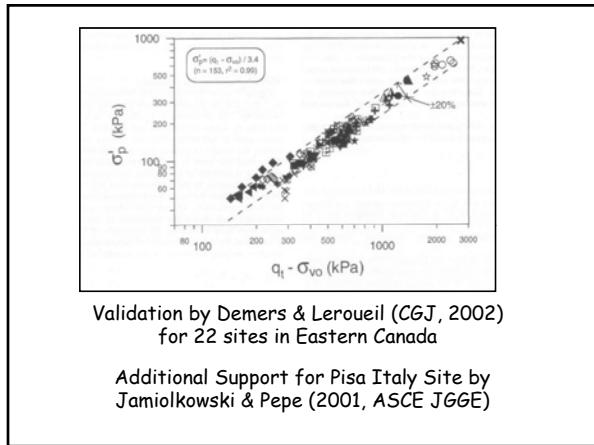
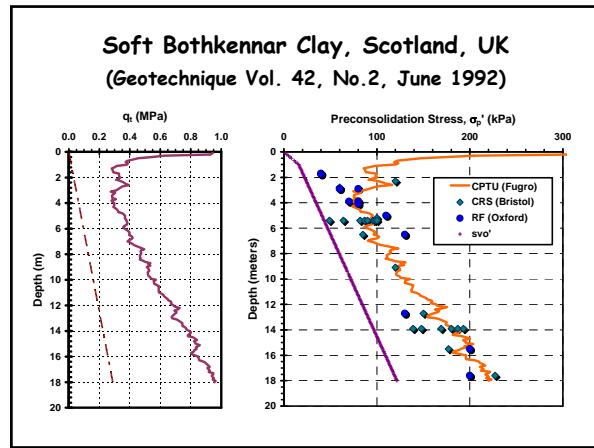
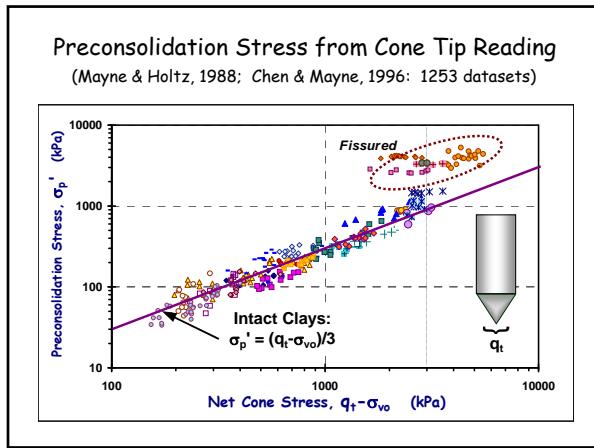
$$\rightarrow \sigma'_p = \frac{(\Delta u_b - u_o)}{\frac{M}{3} \ln(I_R)} \rightarrow \sigma'_p = 0.54 \cdot (\Delta u_b)$$

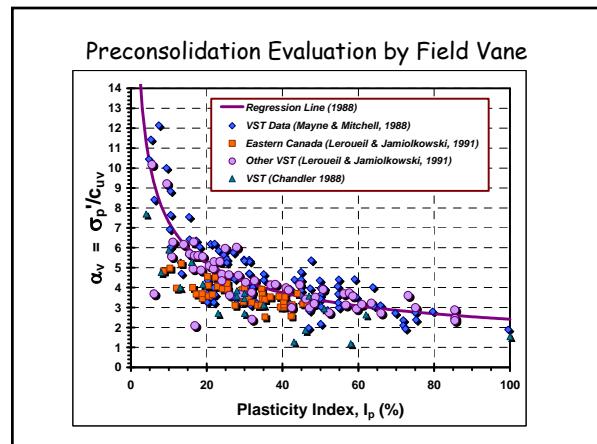
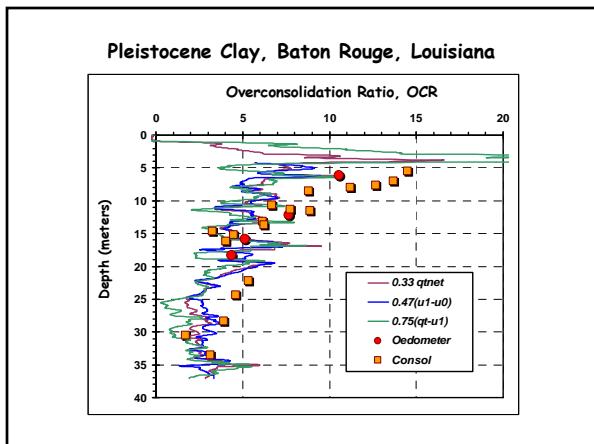
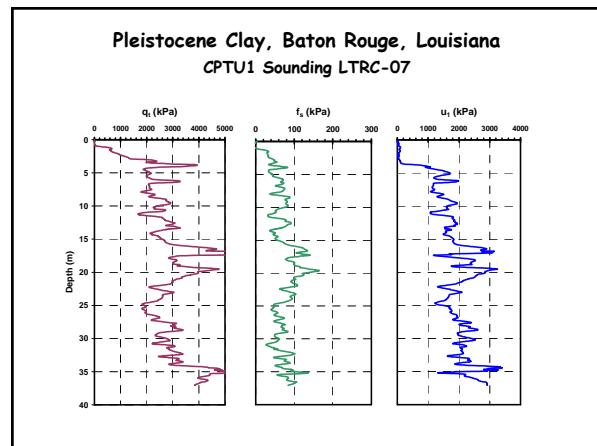
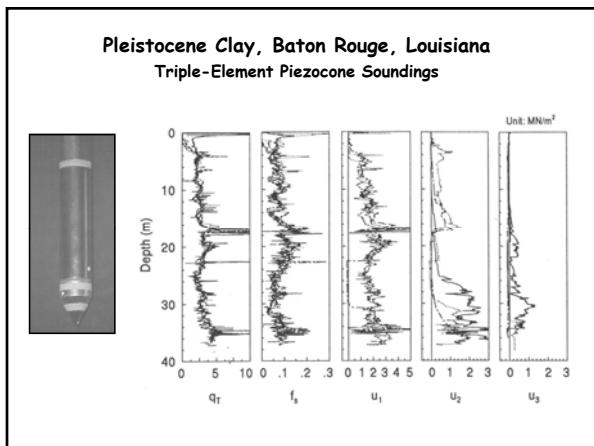
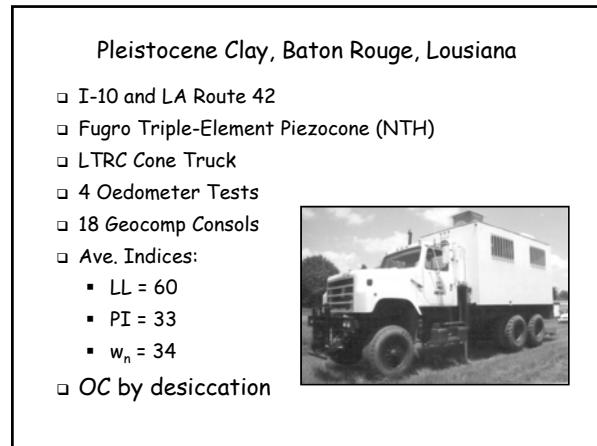
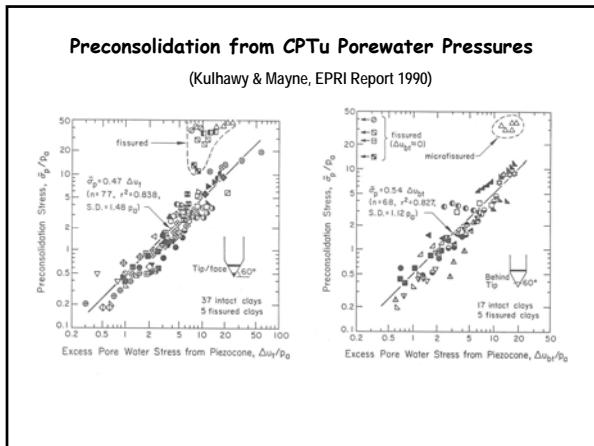
$$\sigma'_p = 2 \cdot \left[\frac{1}{1.95M + 1} \left(\frac{q_t - u_b}{\sigma'_{vo}} \right) \right]^{(1/\Lambda)}$$

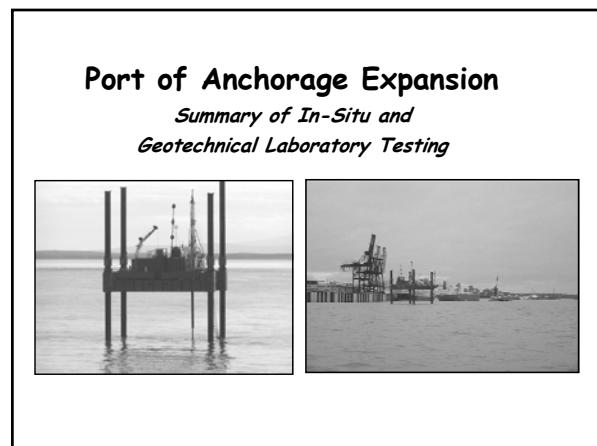
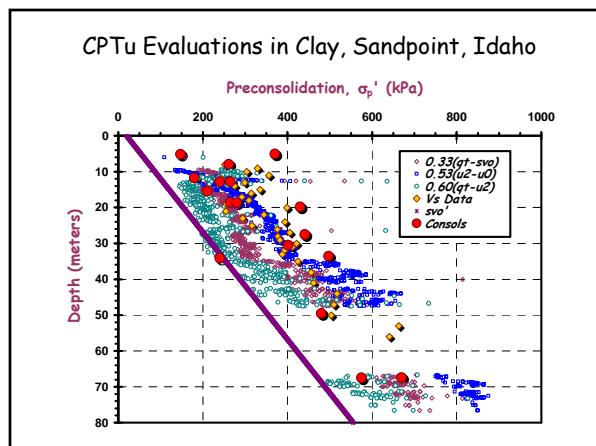
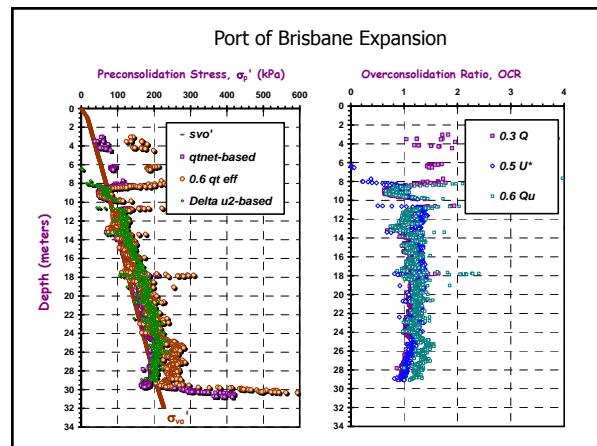
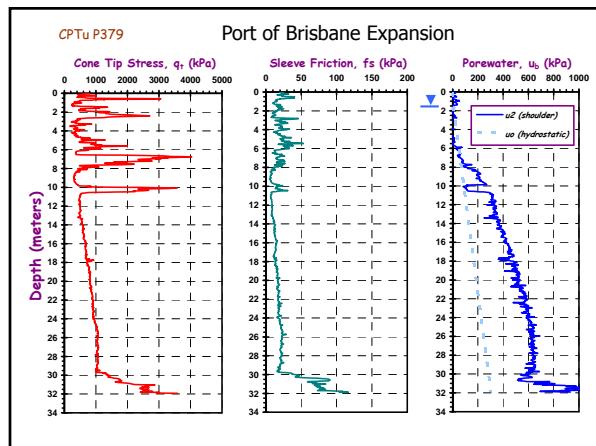
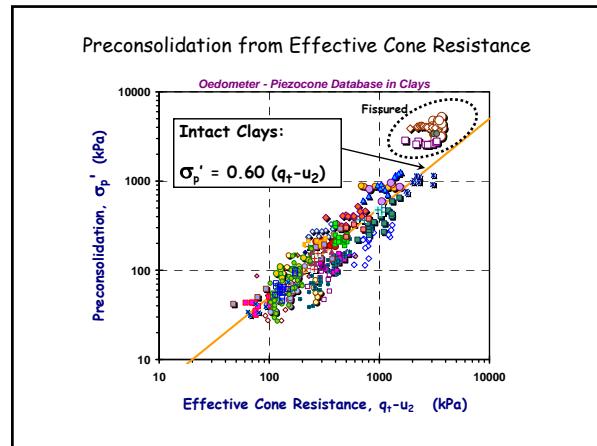
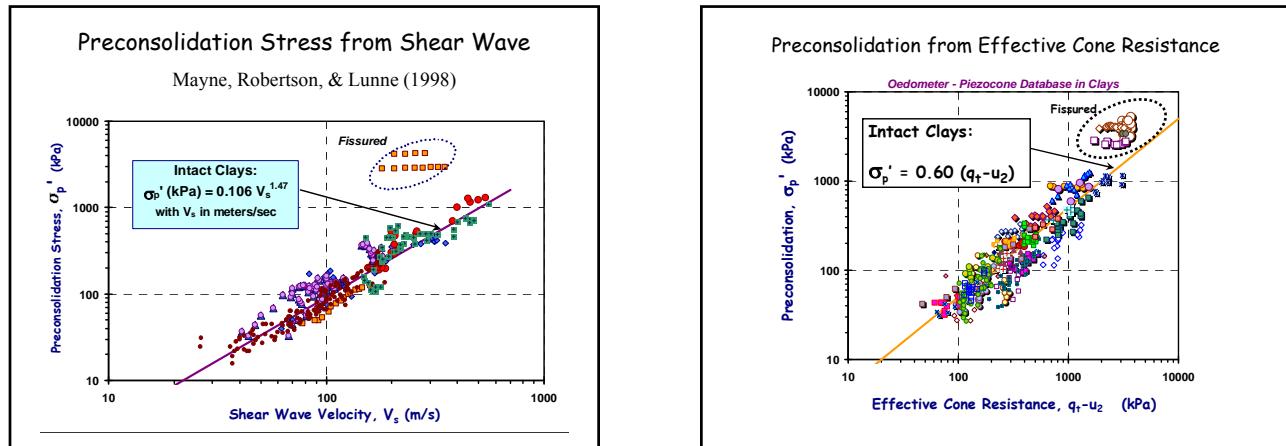
$$\rightarrow \sigma'_p = \frac{(q_t - u_b)}{0.975M + \frac{1}{2}} \rightarrow \sigma'_p = 0.60 \cdot (q_t - u_b)$$









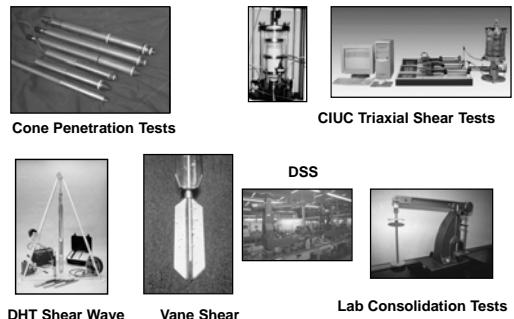


Summary of POA Geotechnical Testing

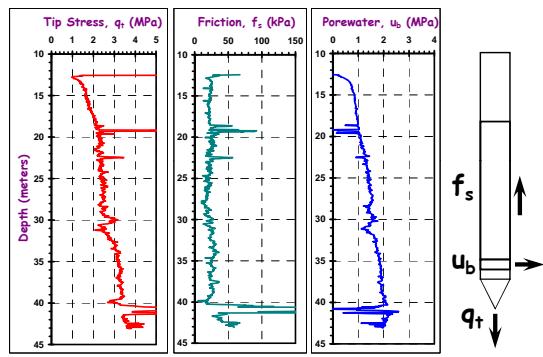
- ❑ Soil Test Borings with Thin-Walled Tube Sampling
- ❑ Lab Testing: Index, Consolidation, Triaxial Shear, Direct Simple Shear
- ❑ Piezocone Soundings
- ❑ Vane Shear Tests
- ❑ Shear Wave Velocity



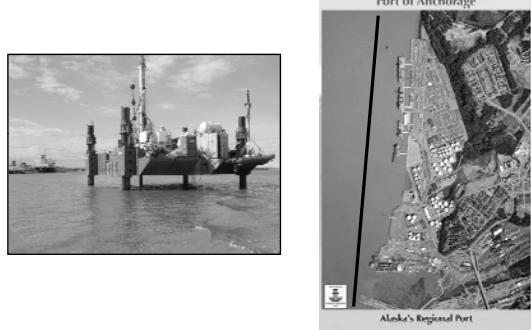
POA In-Situ and Laboratory Test Program



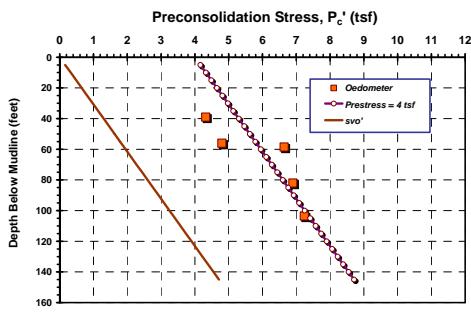
POA Representative Piezocone Sounding



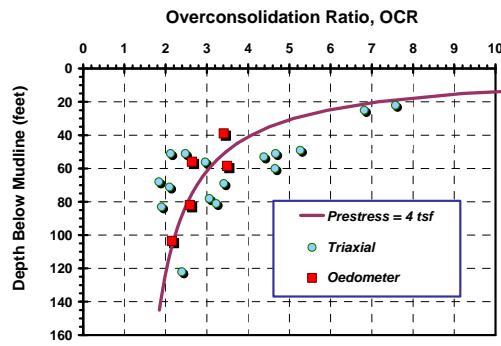
Port of Anchorage Expansion

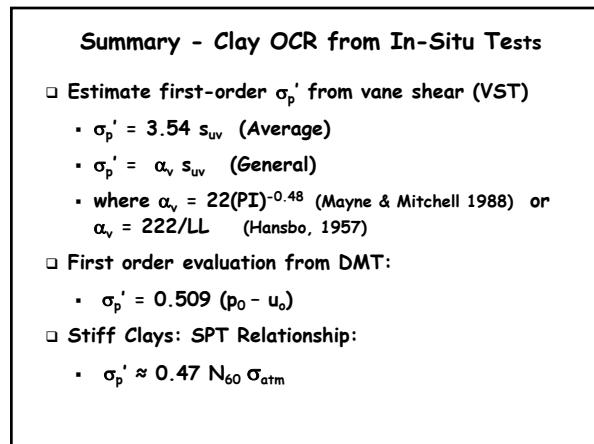
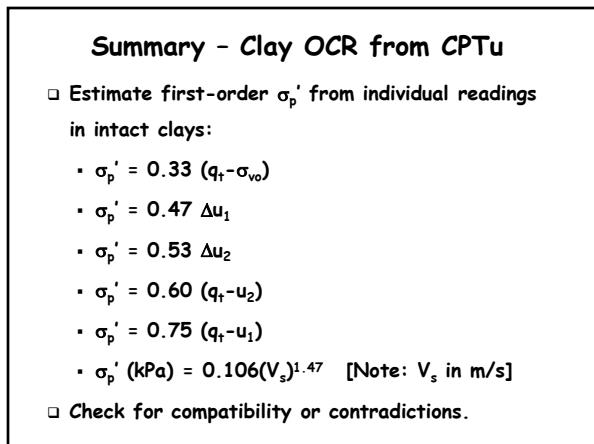
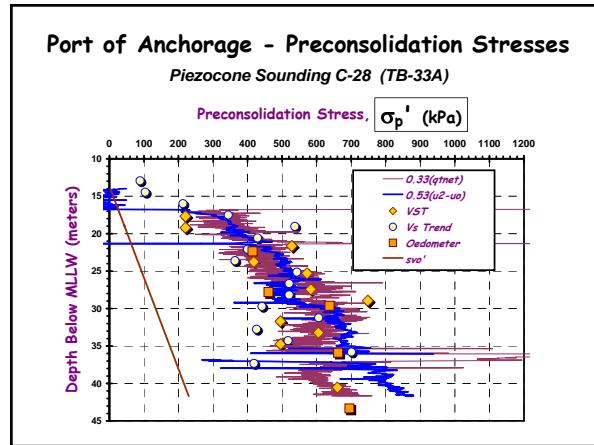
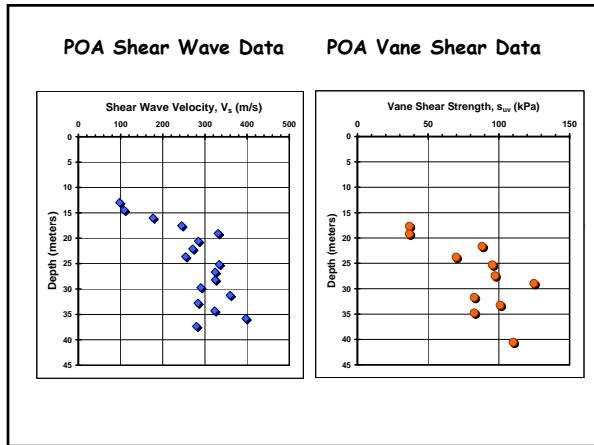
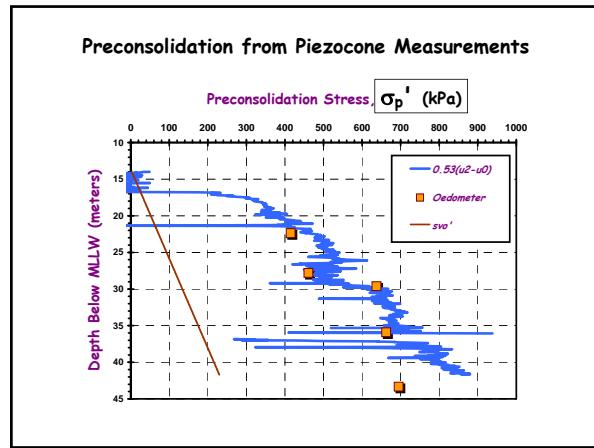
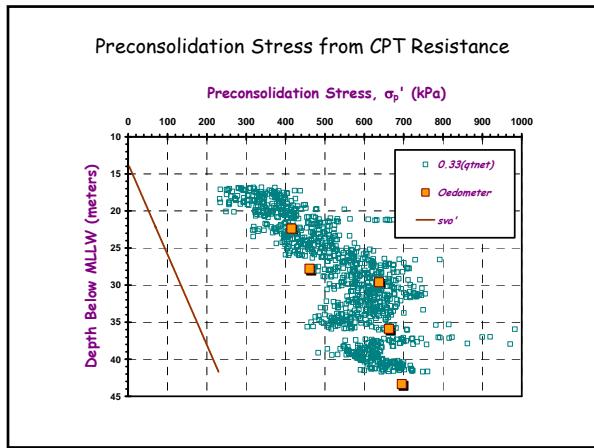


Oedometer Testing on Bootlegger Cove Clay



Lab Testing on Bootlegger Cove Clay





Sample Disturbance

Paul W. Mayne
Georgia Tech
May 2008

Sample Disturbance Effects

Ladd and DeGroot (2003) Soil & Rock America

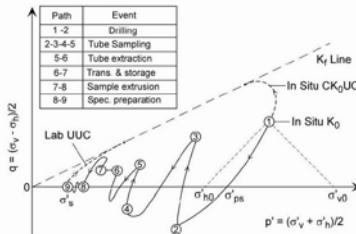


Figure 5-7. Hypothetical stress path during tube sampling and specimen preparation of centerline element of low OCR clay (Ladd and DeGroot, 2003)

Georgia Tech

Sample Disturbance Effects

Table 5-1. Main dimensions and features of soil samplers (modified after Tanaka, 2000)

Sampler	Outside diameter (mm)	Inside diameter (mm)	Sampler length (mm)	Thickness (mm)	Area Ratio (%)**	Piston
JPN	78	75	1000	1.5	7.5	Yes
Laval	216	208	660	4.0	7.3	No
Shelby	75.3	72	610	1.65	8.6	No
NGI54	80	54	768	13	54.4	Yes
ELE100	104.4	101	500	1.7	6.4	Yes
Sherbrooke (Block sampler)	N/A***	350*	250*	N/A***	N/A***	No
NGI95	105.6	95	1000	5.3	14	Yes
Split-barrel	51.1	34.9	450-600	8.1	112	No

* Specimen dimensions

** Area ratio = $\frac{d_{\text{outside}}^2 - d_{\text{internal}}^2}{d_{\text{internal}}^2}$ (US Army Corps of Engineers, 1996)

*** Sherbrooke sampler employs a special sampling technique that does not require a sampling tube. Refer to Lefebvre and Poulin (1979) for more details

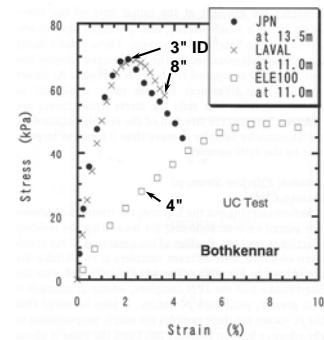
Georgia Tech

Sample Disturbance Effects

Bothkennar Clay, UK

(Tanaka et al. 2000)

UC Lab Tests

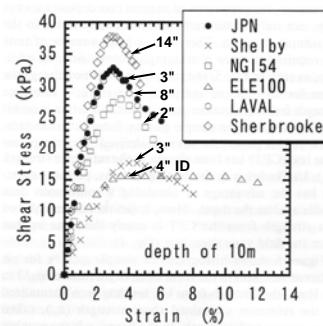


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Sample Disturbance Effects

Ariake Clay, Japan
(Tanaka et al. 2000)

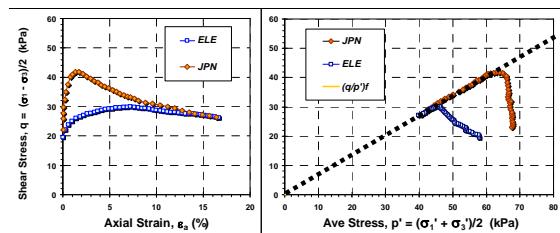
UC Lab Tests



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Sampling Disturbance Effects

Tanaka et al. (2000, 2001)



Sample Disturbance Effects

Ladd and DeGroot (2003) Soil & Rock America (SARA)
Jointly - Proceedings Pan American Conf SMFE at MIT

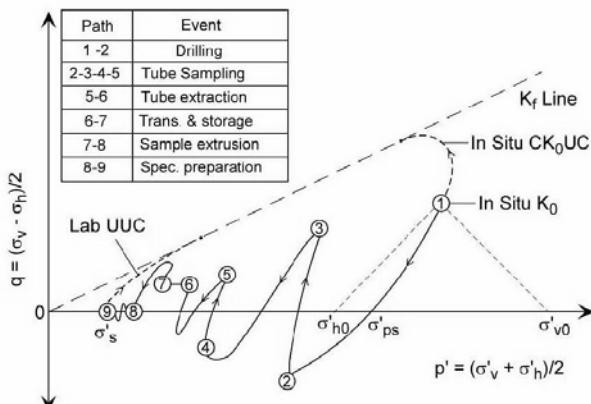


Figure 5-7. Hypothetical stress path during tube sampling and specimen preparation of centerline element of low OCR clay (Ladd and DeGroot, 2003)

Georgia Tech

Sample Disturbance Effects

Undisturbed sampler geometries

Table 5-1. Main dimensions and features of soil samplers (modified after Tanaka, 2000)

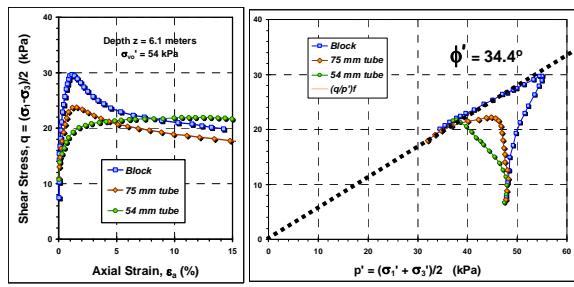
Sampler	Outside diameter (mm)	Inside diameter (mm)	Sampler length (mm)	Thickness (mm)	Area Ratio (%)**	Piston
JPN	78	75	1000	1.5	7.5	Yes
Laval	216	208	660	4.0	7.3	No
Shelby	75.3	72	610	1.65	8.6	No
NGI54	80	54	768	13	54.4	Yes
ELE100	104.4	101	500	1.7	6.4	Yes
Sherbrooke (Block sampler)	N/A***	350*	250*	N/A***	N/A***	No
NGI95	105.6	95	1000	5.3	14	Yes
Split-barrel	51.1	34.9	450-600	8.1	112	No

* Specimen dimensions
** Area ratio = $\frac{(d_{\text{external}}^2 - d_{\text{internal}}^2)}{d_{\text{internal}}^2}$ (US Army Corps of Engineers, 1996)
*** Sherbrooke sampler employs a special sampling technique that does not require a sampling tube. Refer to Lefebvre and Poulin (1979) for more details

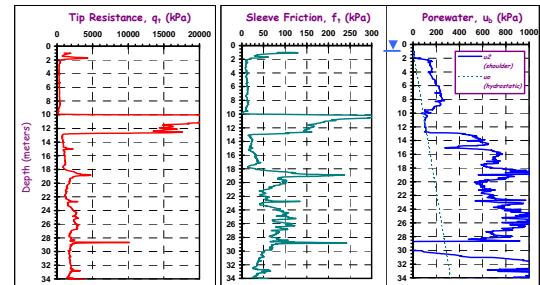
Georgia Tech

Sampling Disturbance Effects

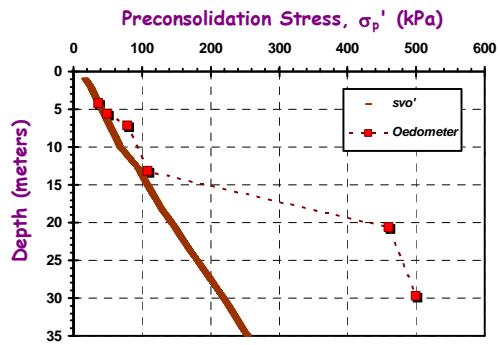
Lunne et al. (2006, CGJ): Lierstranda Clay 6.1 m



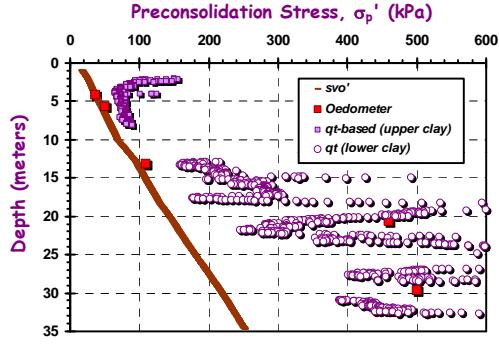
Case Study East Australia



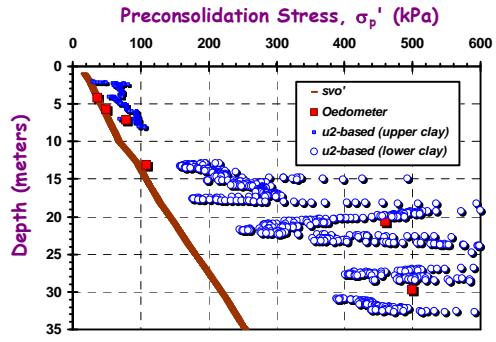
Site Characterization - East Australia



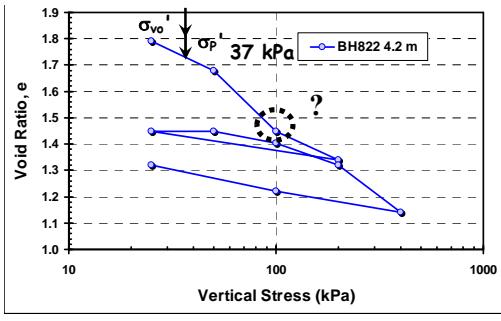
Site Characterization - East Australia

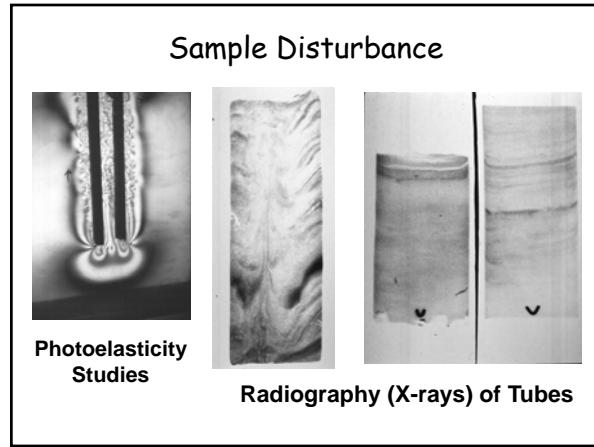
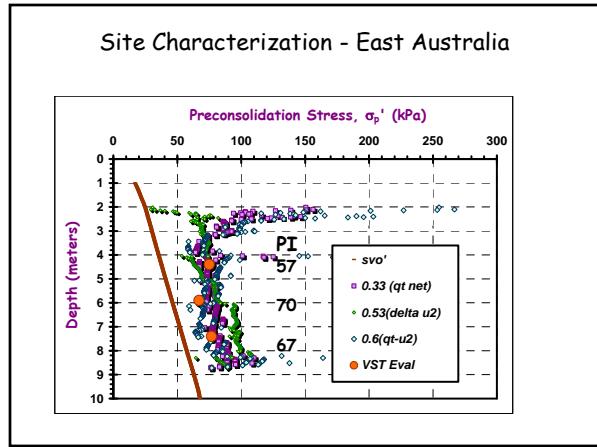
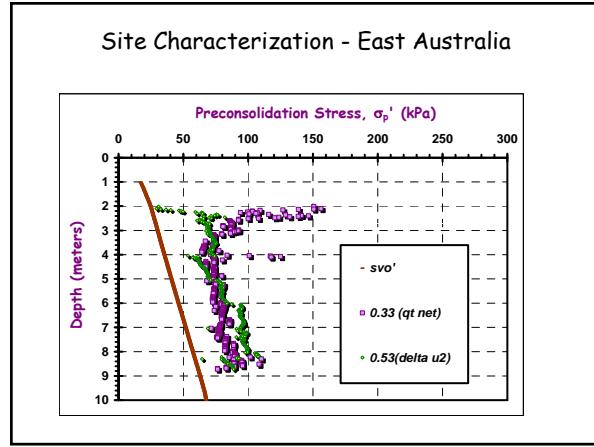
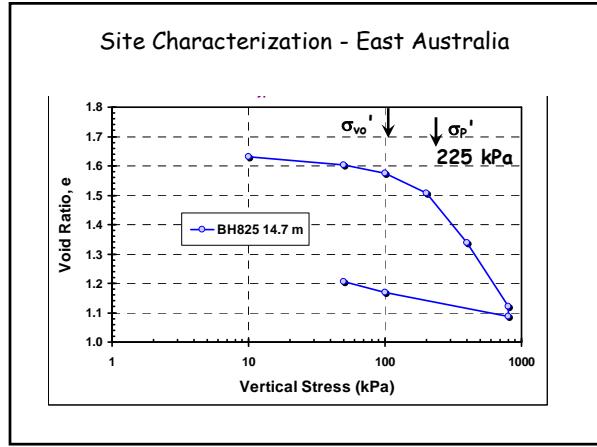
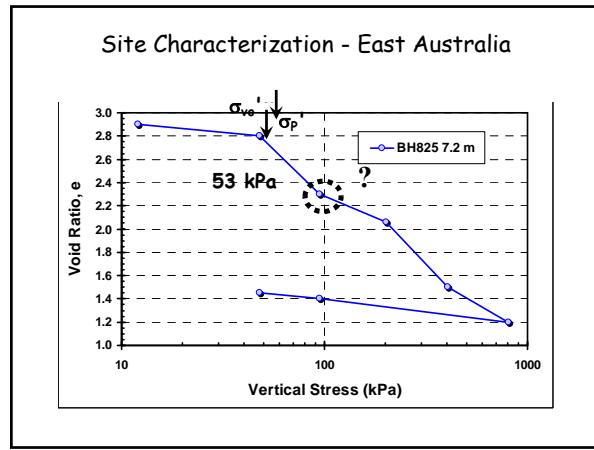
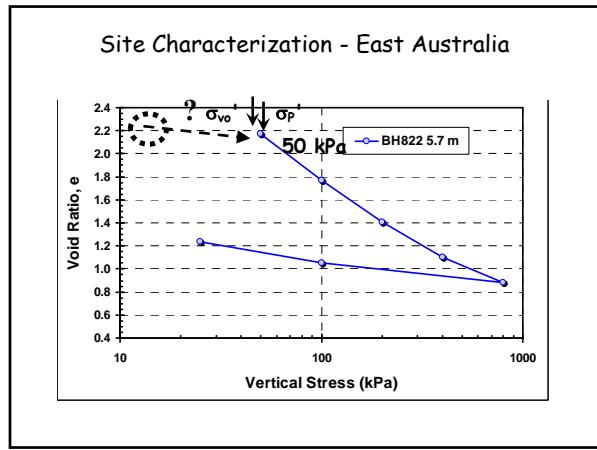


Site Characterization - East Australia



Site Characterization - East Australia





Quantification of Lab Sample Disturbance

Lunne, et al. (2006, CGJ Vol. 43: 725-750)

RATINGS: Ratio of $\Delta e/e_0$ to attain σ_{vo}'

OCR	Excellent to Very Good	Good to Fair	Poor	Very Poor
1 to 2	< 0.04	0.04 to 0.07	0.07 to 0.14	> 0.14
2 to 4	< 0.03	0.03 to 0.05	0.05 to 0.10	> 0.10

References: Sample Disturbance Effects

- Ladd, C.C. and DeGroot, D.J. (2003). Recommended practice for soft ground site characterization: *Proc. 12th Pan American Conference on Soil Mechanics & Geotechnical Engineering*, Vol. 1, Verlag Glückauf GMBH, Essen: 3-57.
- Hight, D.W. and Leroueil, S. (2003). Characterisation of soils for engineering purposes. Proceedings: *Characterization & Engineering Properties of Natural Soils*, Vol. 1 (Singapore Workshop, Balkema/ Rotterdam: 255-360.

References: Sample Disturbance Effects

- Lunne, et al. (2006). Effects of sample disturbance and consolidation procedures. *Canadian Geotechnical Journal* 43: 726-750.
- Tanaka, H. (2000). Sample quality of cohesive soils: Lessons from three sites: Ariake, Bothkennar, and Drammen. *Soils & Foundations*, Vol. 40 (4): 54-74.
- Lacasse, S., Berre, T. and Lefebvre, G. (1985). Block sampling of sensitive clays. *Proceedings, 11th Intl. Conference on Soil Mechanics and Foundations Engineering*, Vol. 2, San Francisco, 887-892.

Undrained Shear Strength

Invited Keynote: 11th Baltic Geotechnical Maritime Conference – Gdansk (15-18 Sept. 2008)

Piezocene Profiling of Clays for Maritime Site Investigations

Paul W. Mayne
Georgia Institute of Technology

Undrained Shear Strength from CPTu

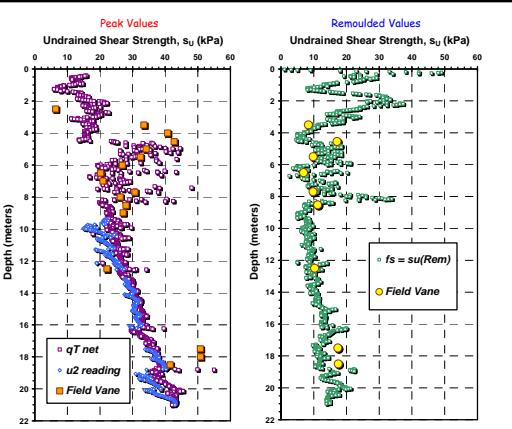
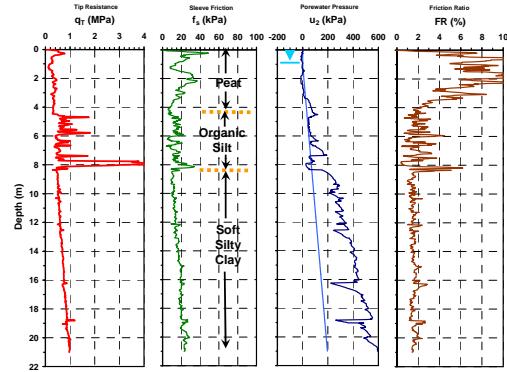
- Piezocene provide three distinct readings - use them
- Undrained shear strength is not unique but exhibits a range of modes
- Stress history (OCR) is a common thread to all s_u modes
- CPTu used to profile OCR; then series of s_u 's
- Case studies involving lab and field data: New Orleans, Bothkennar, Troll, Anchorage

Undrained Shear Strength from CPTu

- $s_u = c_u$ = undrained shear strength
- Independent evaluation by all three readings:

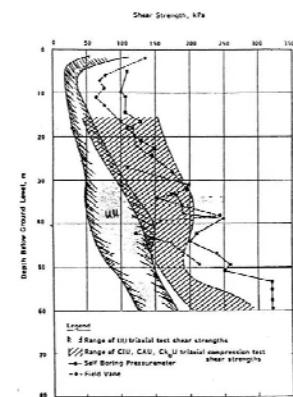
- s_u (remolded) $\approx f_s$
- s_u (peak) $= (u_2 - u_0)/N_{\Delta u}$
- $N_{\Delta u} \approx 10$
- s_u (peak) $= (q_t - \sigma_v)/N_{kt}$
- $N_{kt} \approx 15$

Piezocene Record from New Orleans South



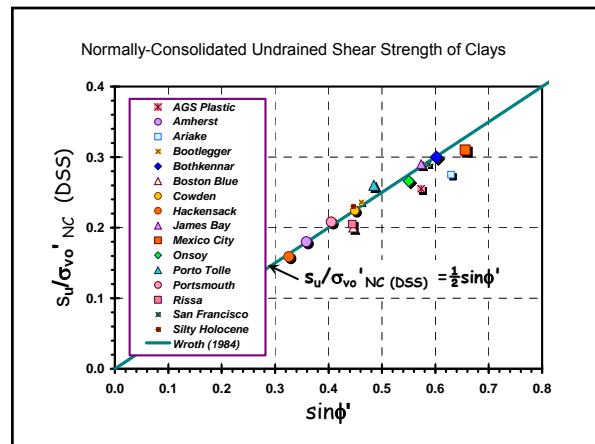
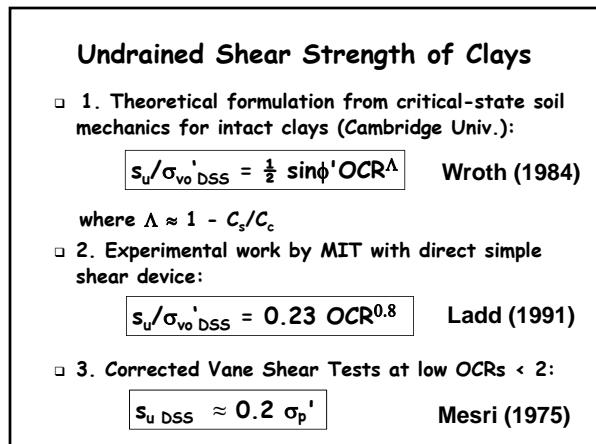
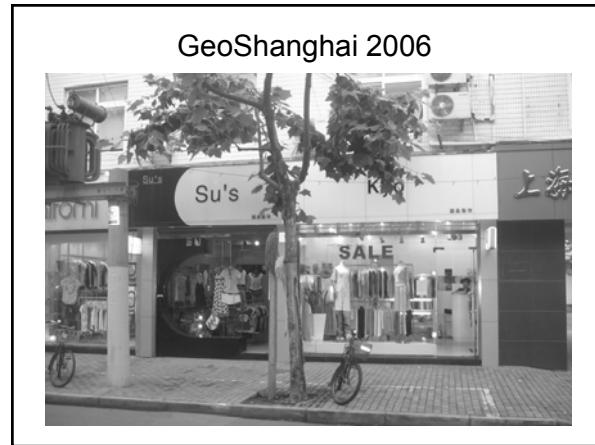
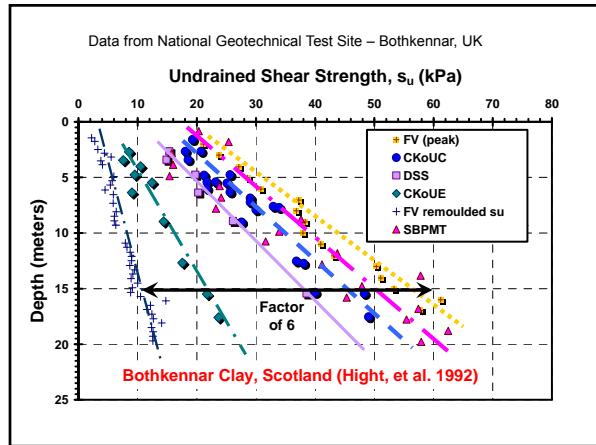
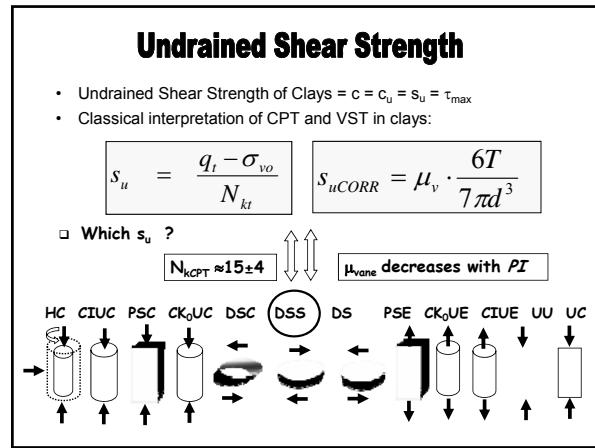
Mix & Match of Undrained Shear Strengths from different tests

Reference: Pentre, UK (Lambson, et al. 1996, Large Scale Pile Tests in Clay, Thomas Telford Ltd, London))



Undrained Shear Strengths for Boston Blue Clay	
Test Method/Mode	$s_u/\sigma_{vo}^{' NC}$
Self-boring pressuremeter (SBPMT)	0.42
Plane strain compression (PSC)	0.34
Triaxial compression ($CK_0 UC$)	0.33
Unconsolidated Undrained (UU)	0.275
Field vane shear test (FV)	0.21
Direct simple shear (DSS)	0.20
Plane strain extension (PSE)	0.19
Triaxial extension ($CK_0 UE$)	0.16
Unconfined compression (UC)	0.14

Ref: MIT Reports; Ladd (1991); Ladd, et al. (1980), Whittle (1993)



Undrained Shear Strength of Clays

Three-Step Hierarchy Approach

- 1. Theoretical formulation from critical-state soil mechanics for intact clays (Cambridge Univ.):

$$s_u / \sigma_{vo}'_{DSS} = \frac{1}{2} \sin\phi' OCR^\Lambda$$

Wroth (1984)
Rankine Lecture
Geotechnique

where $\Lambda \approx 1 - C_s/C_c$

- 2. Experimental work by MIT with direct simple shear device:

$$s_u / \sigma_{vo}'_{DSS} = 0.23 OCR^{0.8}$$

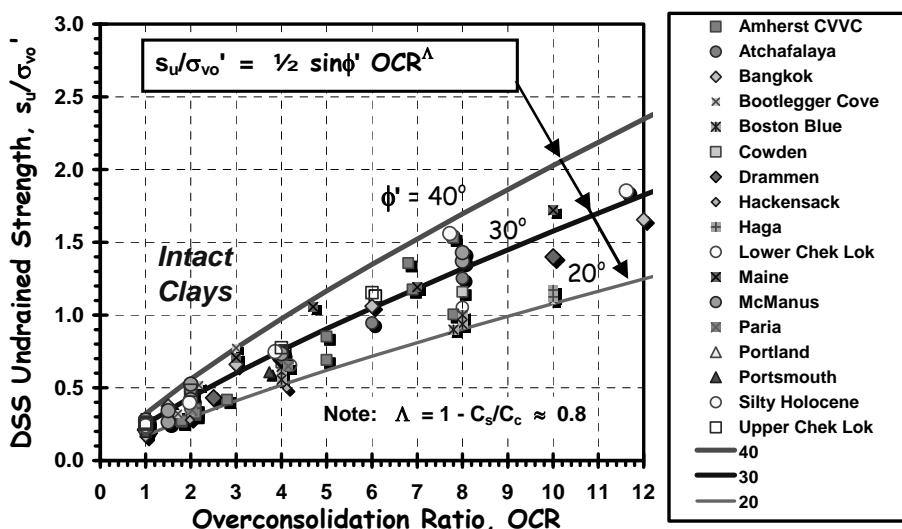
Ladd (1991)
Terzaghi Lecture
ASCE JGE

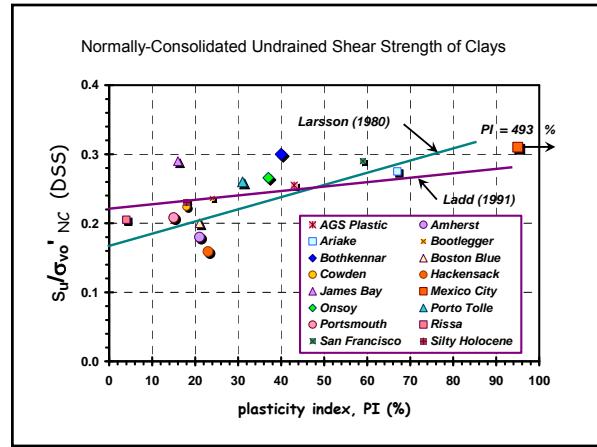
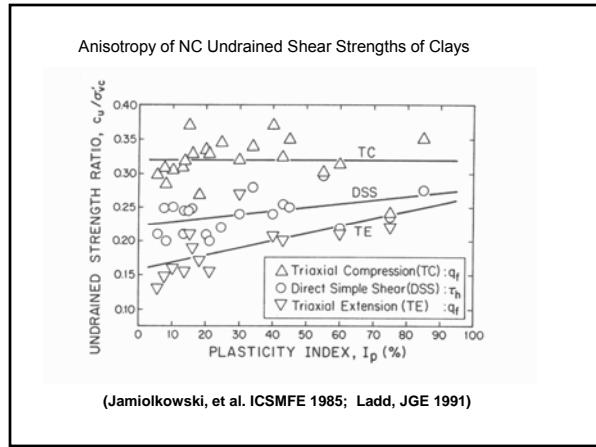
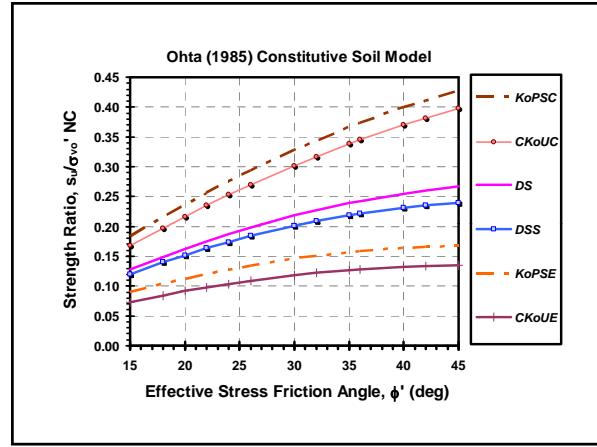
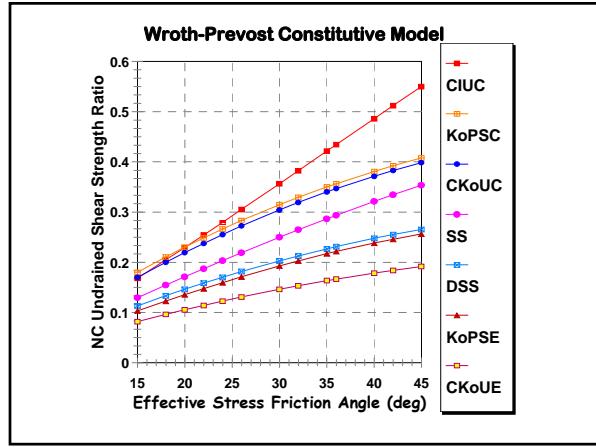
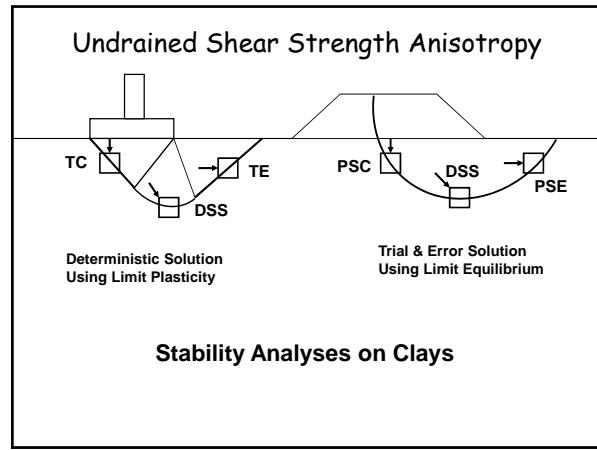
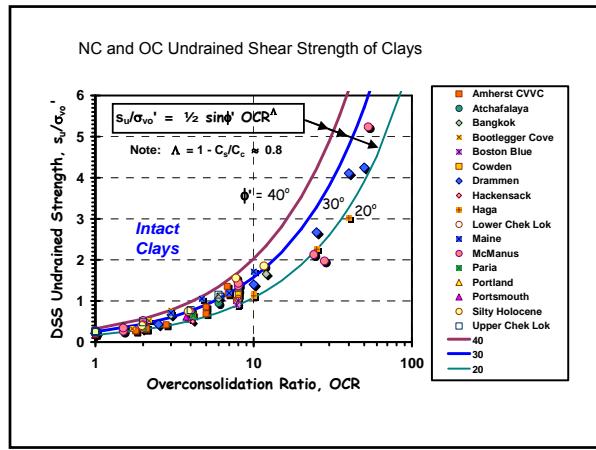
- 3. Corrected Vane Shear Tests at low OCRs < 2:

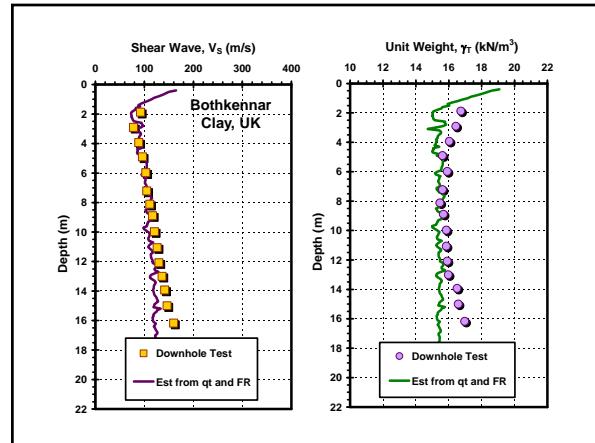
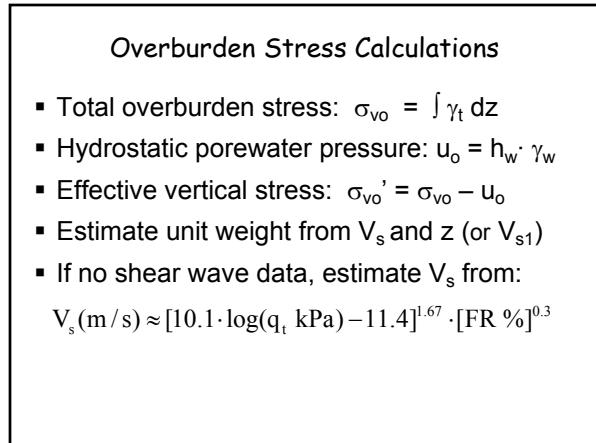
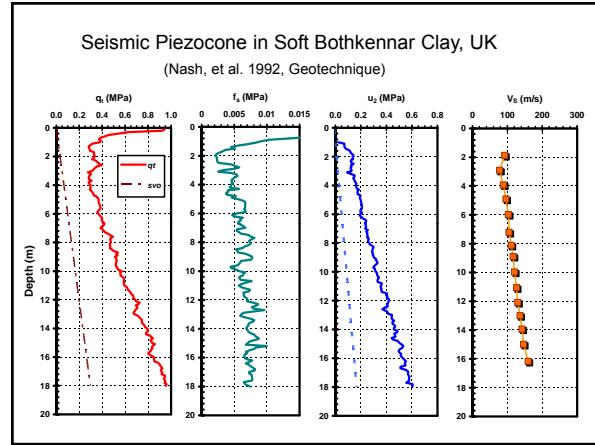
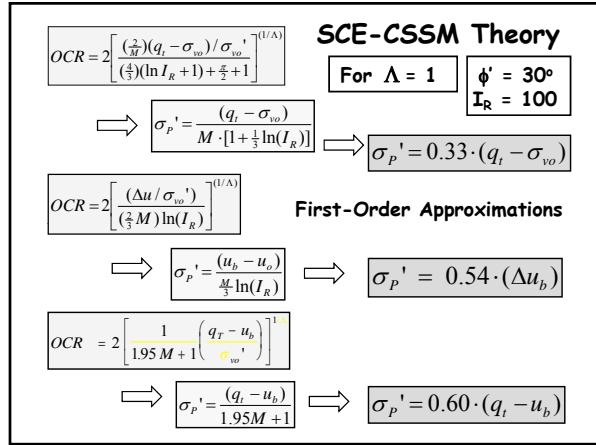
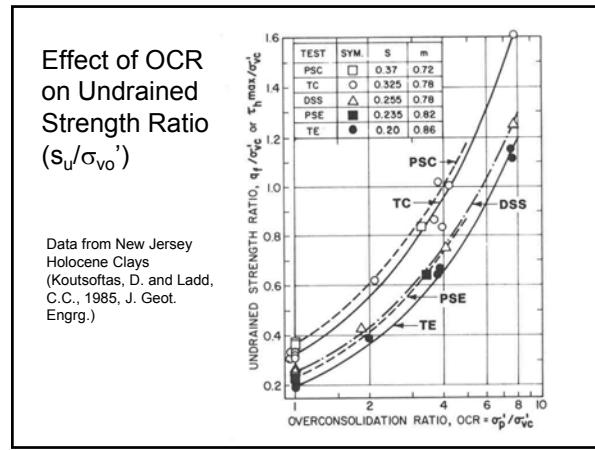
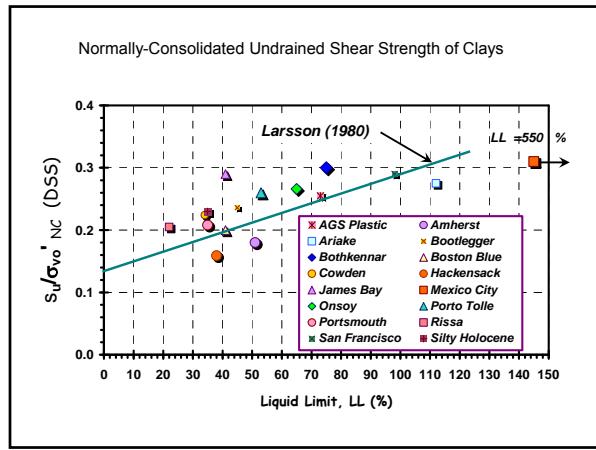
$$s_u DSS \approx 0.2 \sigma_p'$$

Mesri (1975)
ASCE JGE

Normalized NC and OC Undrained Shear Strength of Clays







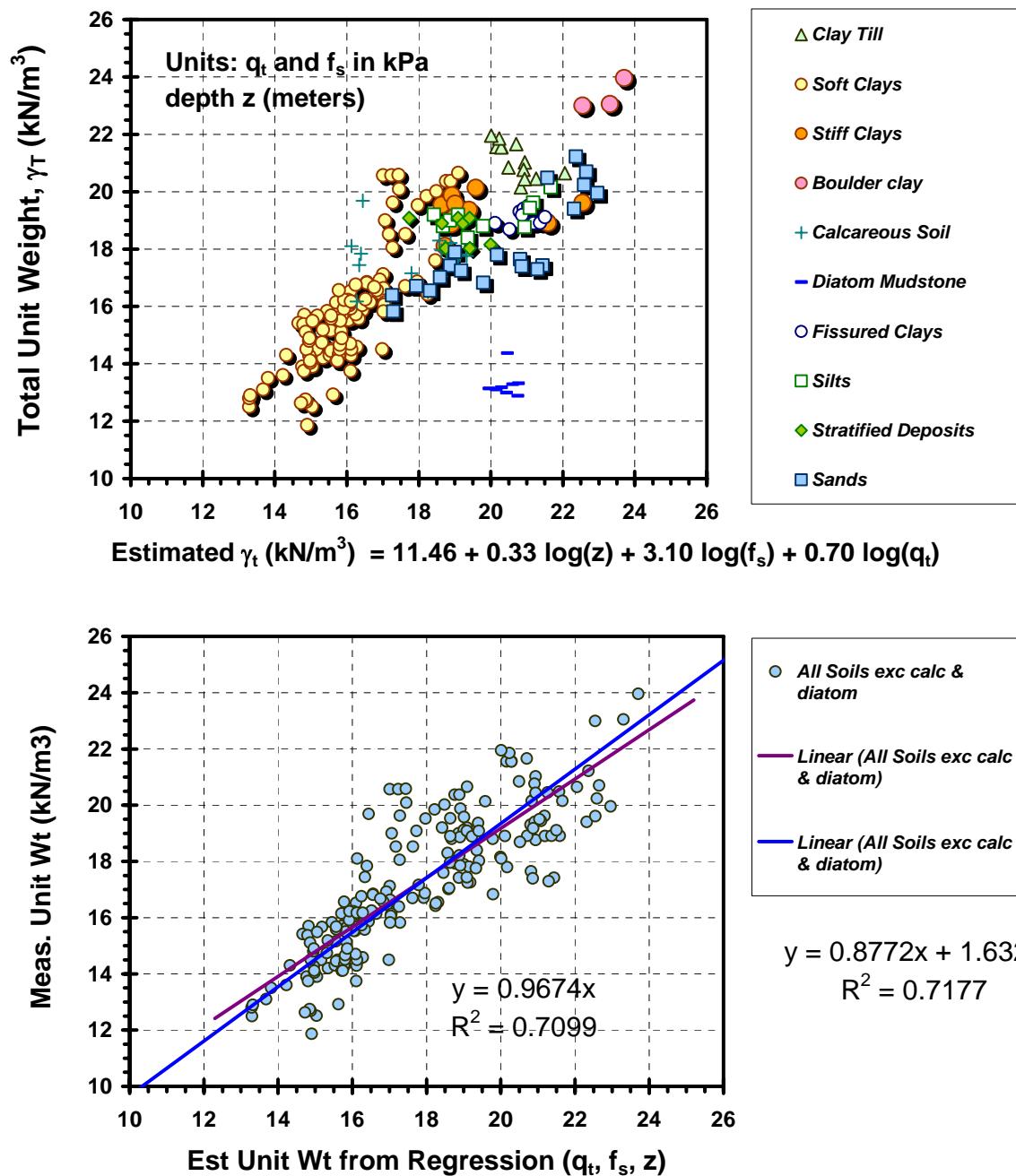


Figure UW-1. Comparison of measured unit weight vs. predicted value using cone tip resistance, sleeve friction, and depth with Table 1 data: (a) grouped soil types; (b) statistical regression analyses.

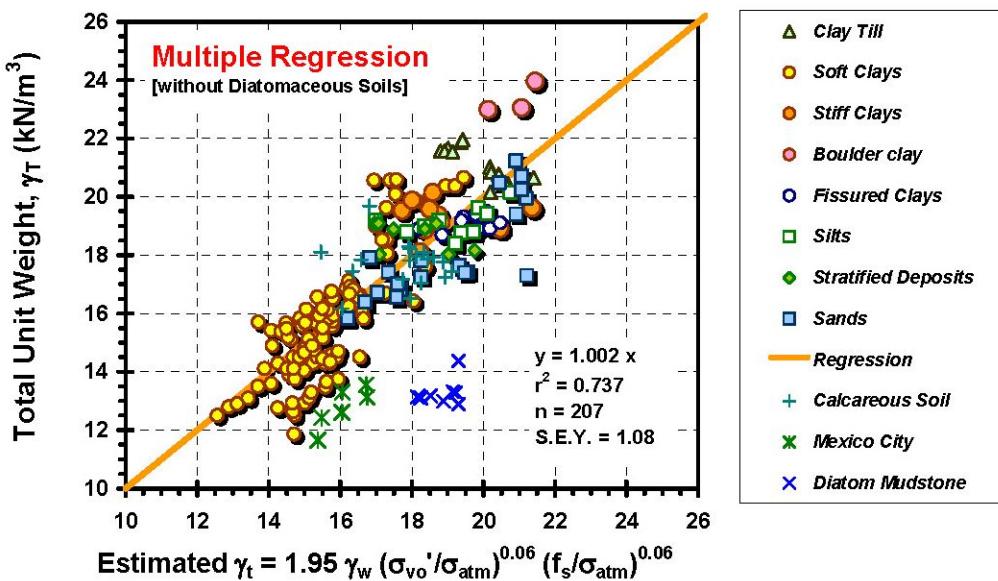


Figure X-2. Multiple regression in power law format for onshore soils.

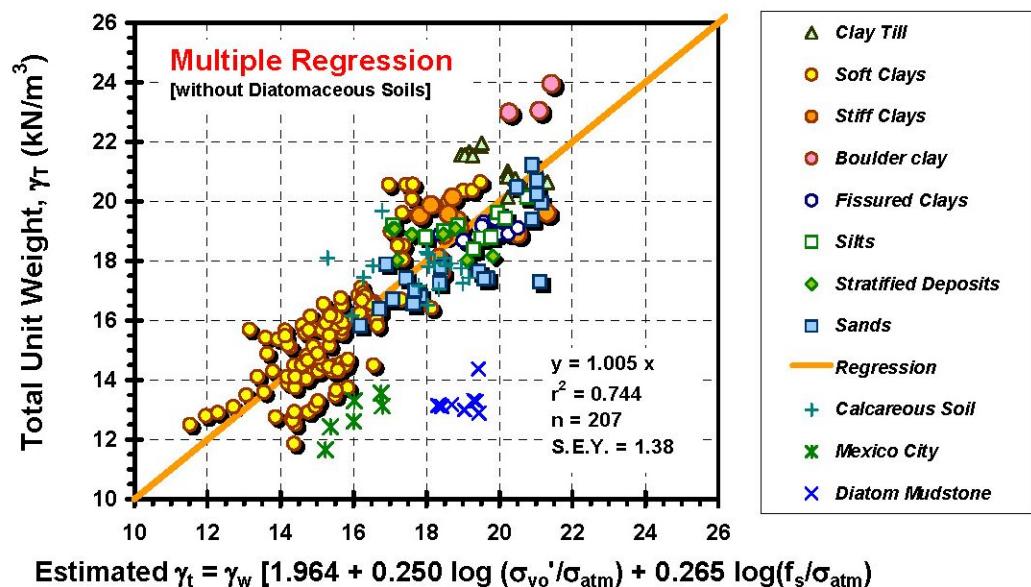
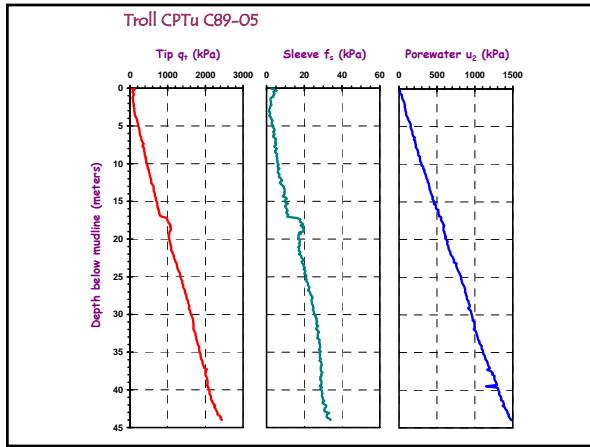
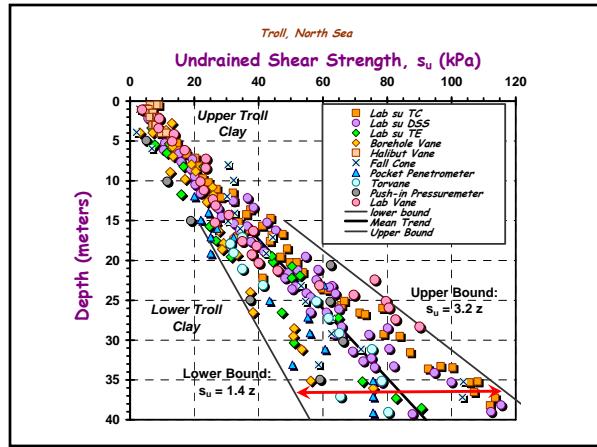
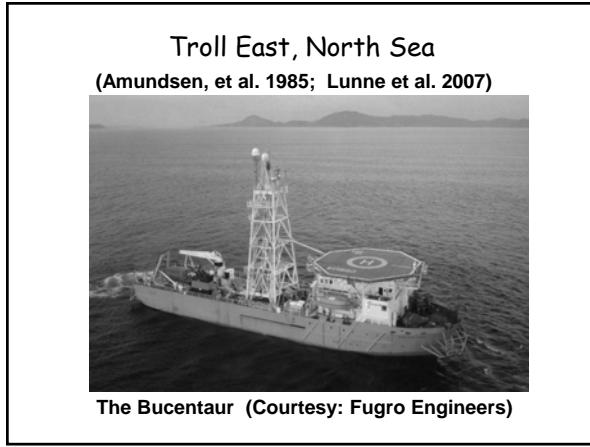
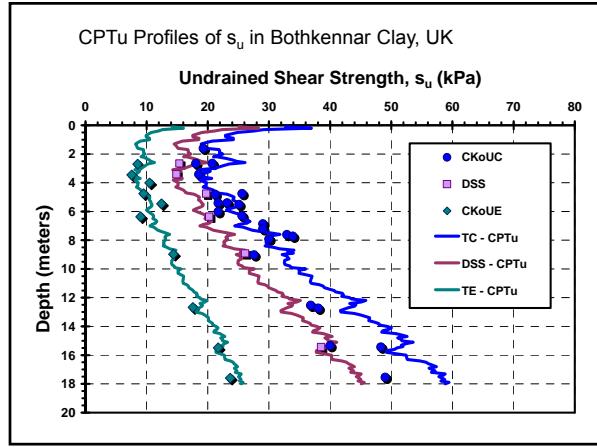
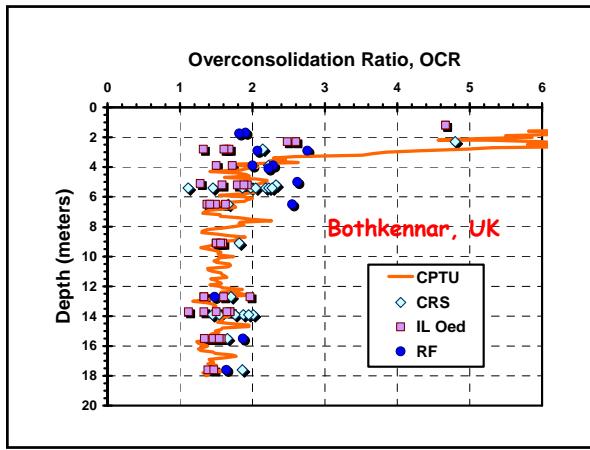
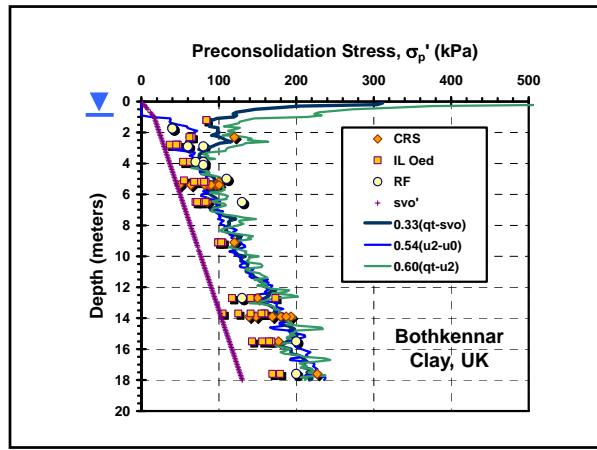
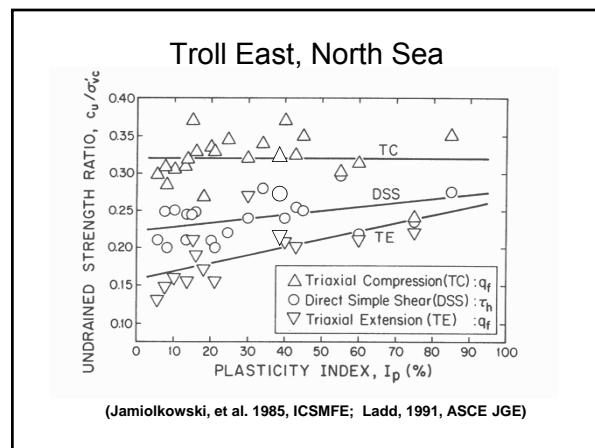
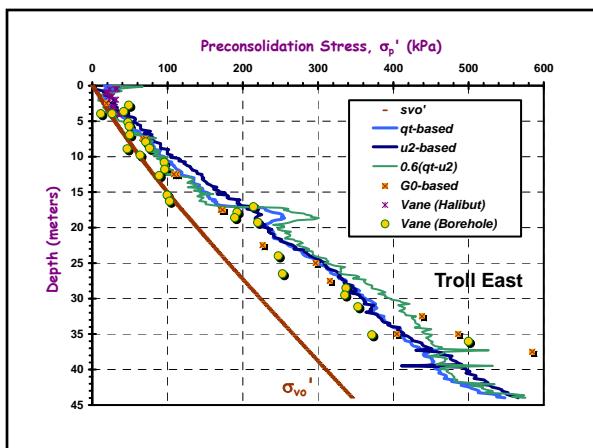
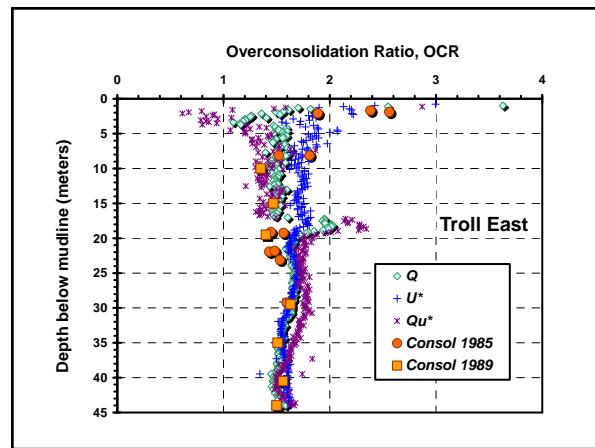
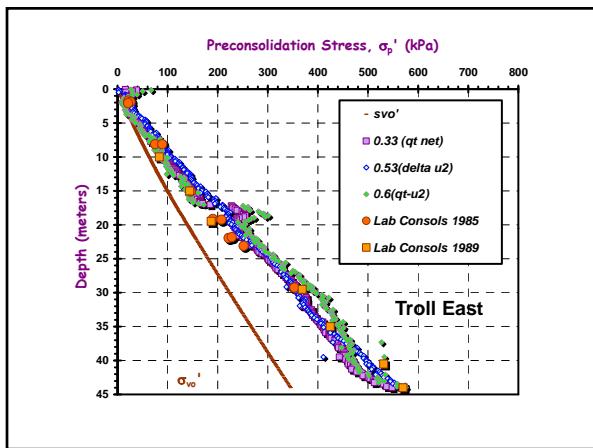
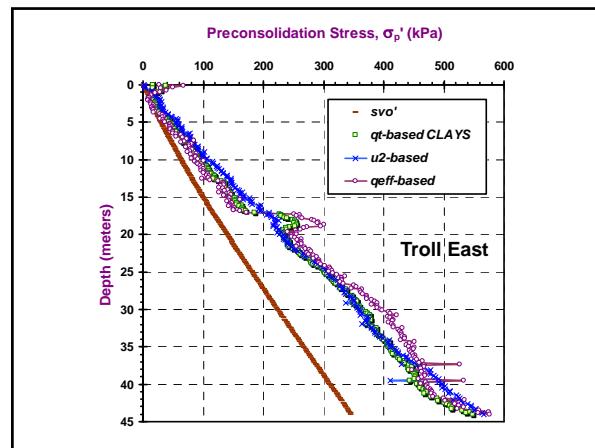
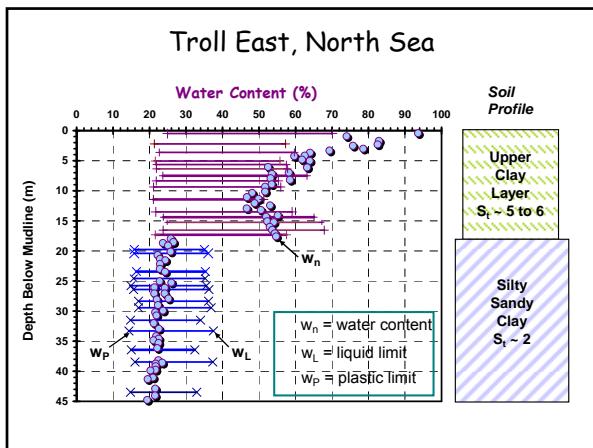
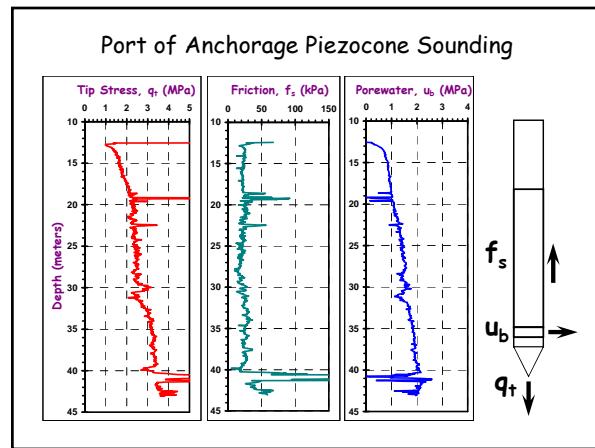
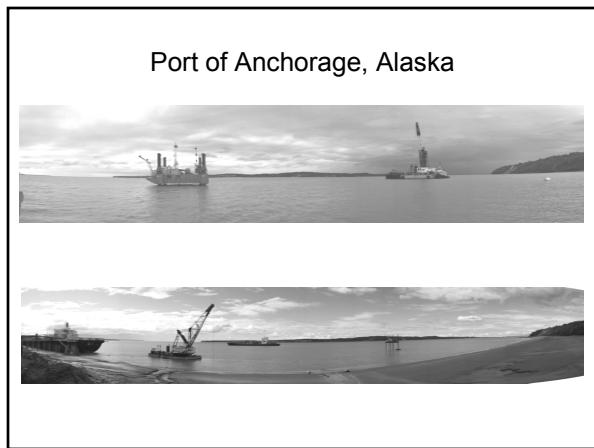
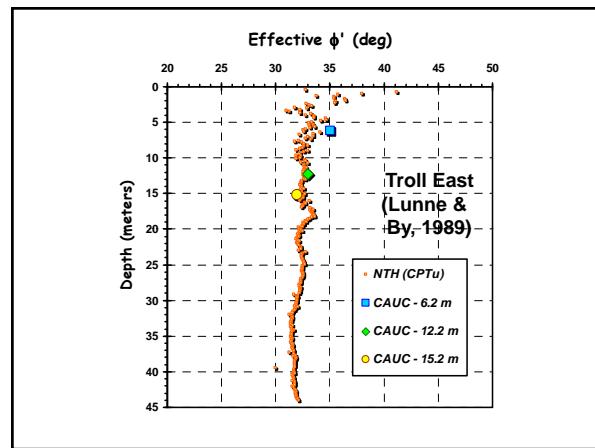
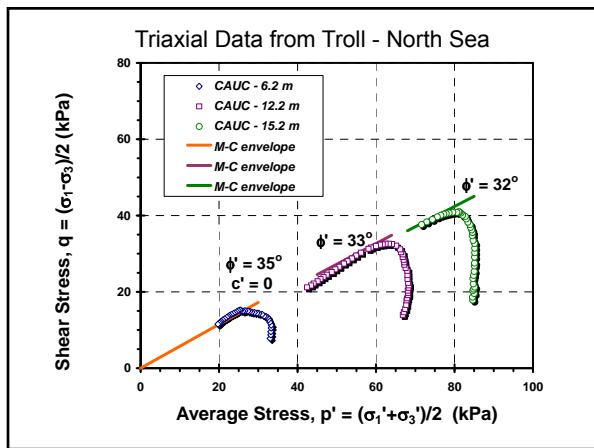
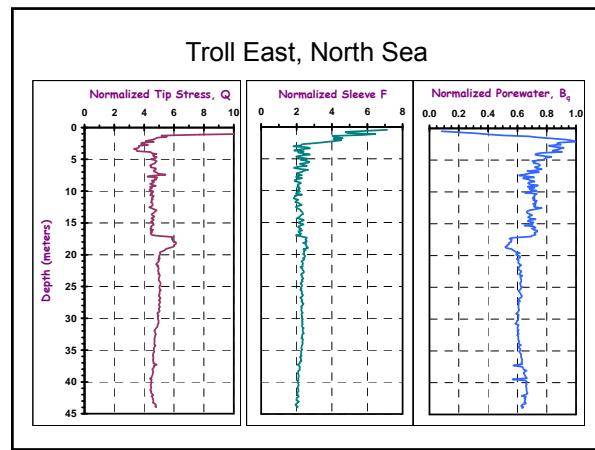
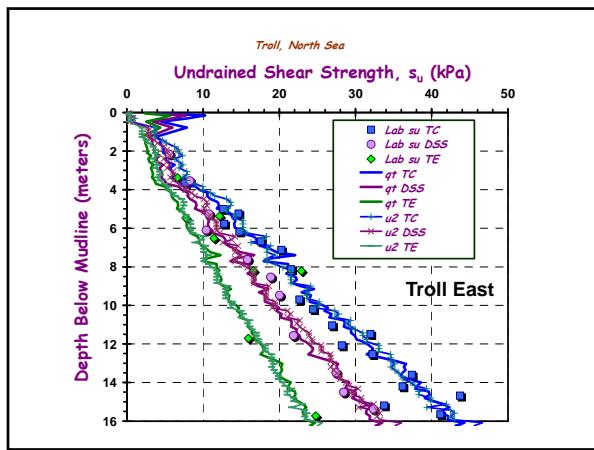
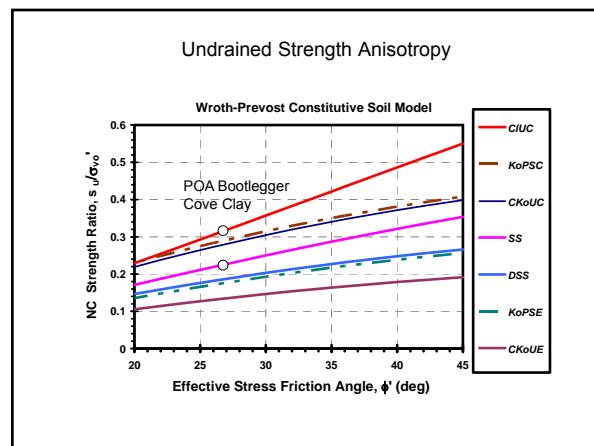
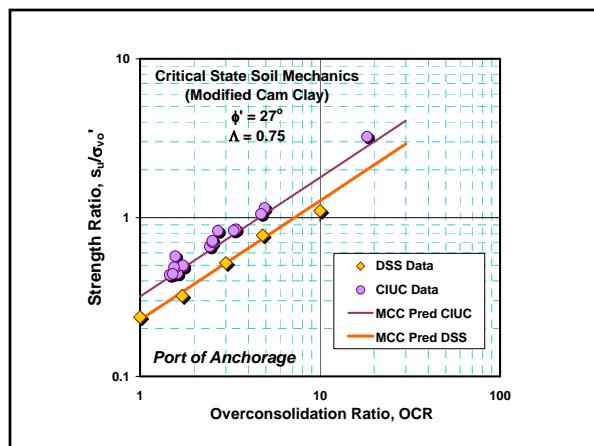
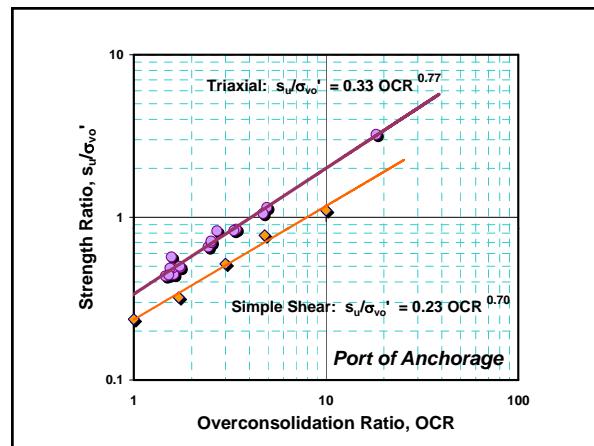
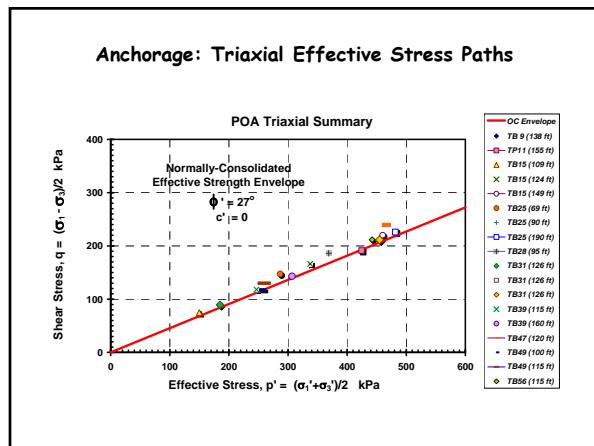
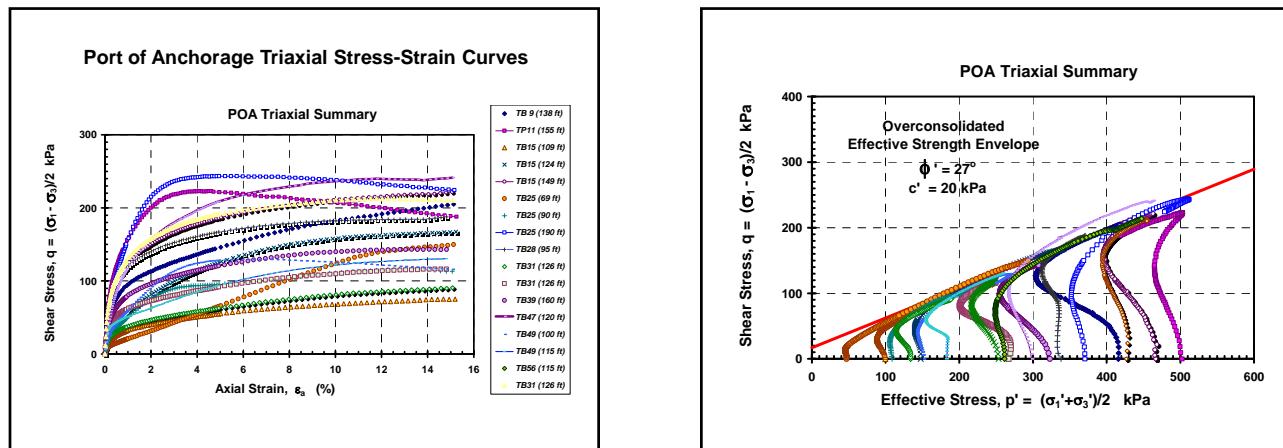


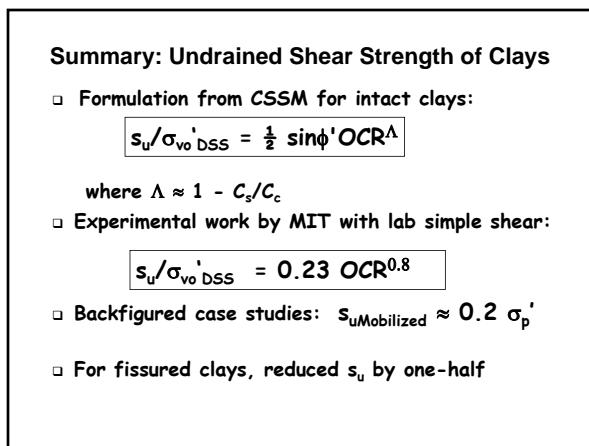
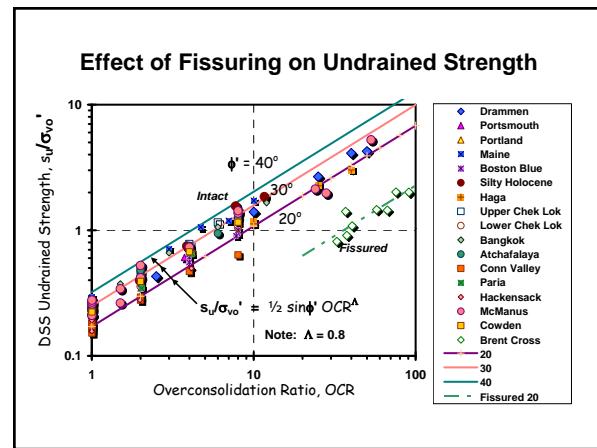
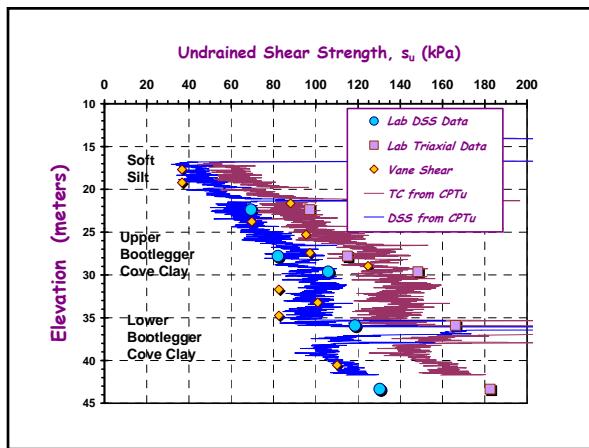
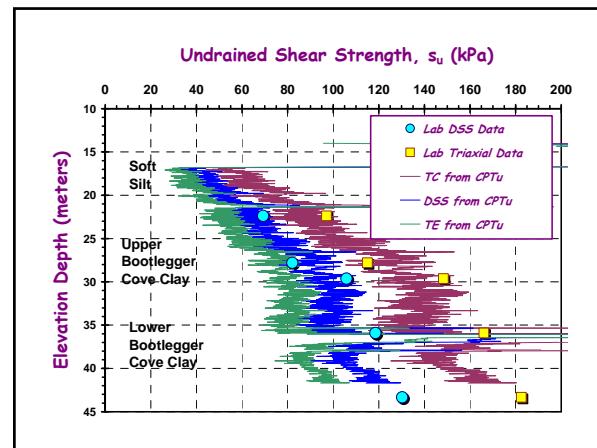
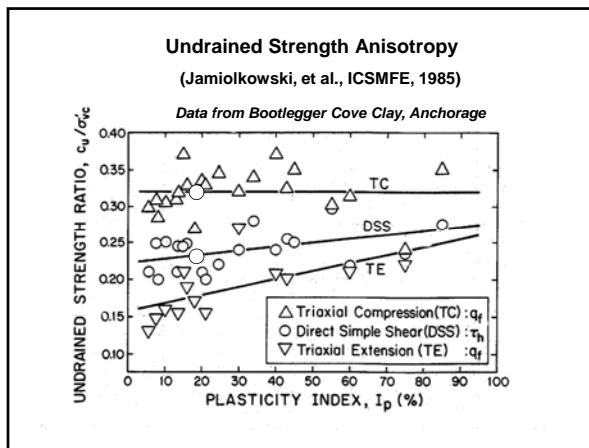
Figure X-3. Multiple regression in assumed semi-log format for onshore soils.











**Case Study: Cooper River Bridge
Charleston, South Carolina**



Paul W. Mayne, PhD, P.E.
Georgia Institute of Technology

Cooper River Bridge, Charleston, South Carolina



Old Pearman and Grace Memorial Bridges



I-17 - Charleston, South Carolina



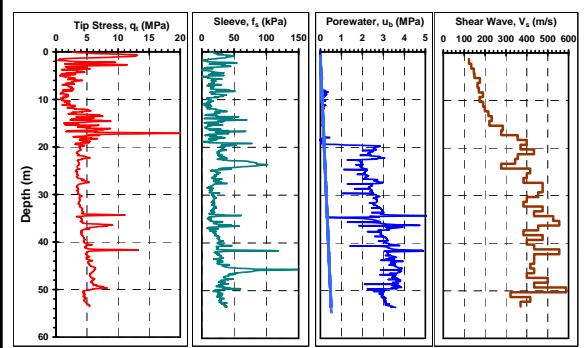
All-Terrain Rotary Drill Rig with Donut Hammer

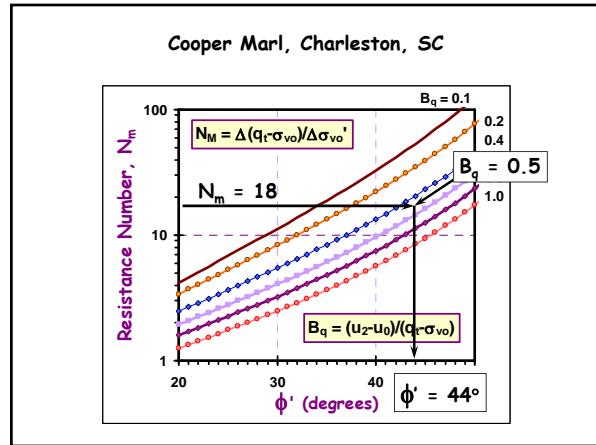
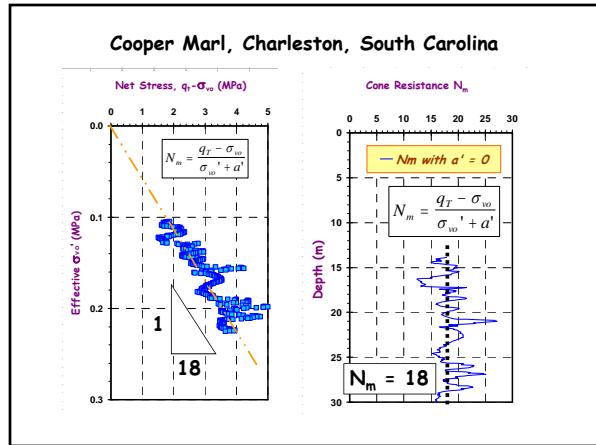
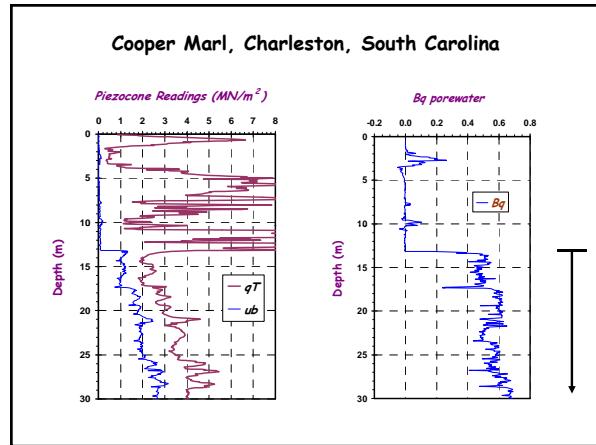
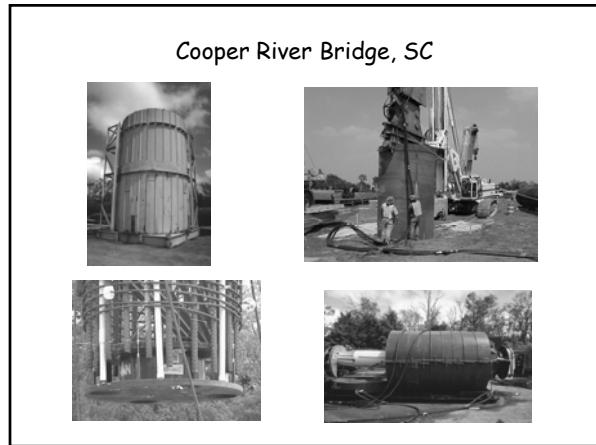
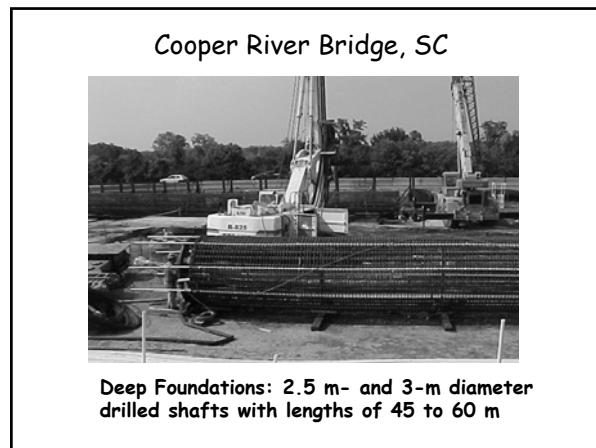
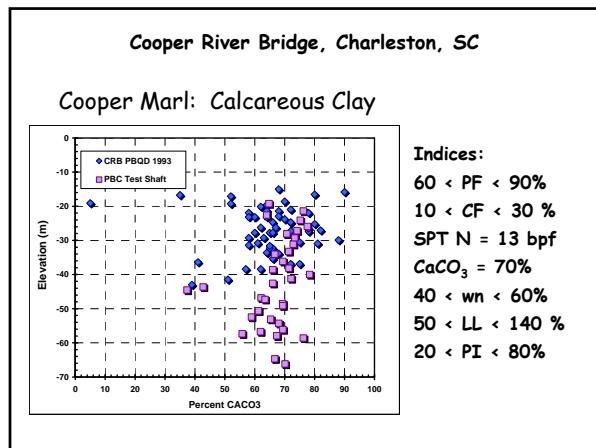
I-17 - Charleston, South Carolina

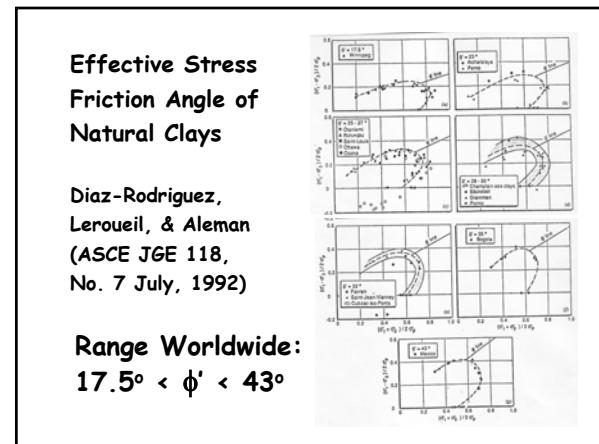
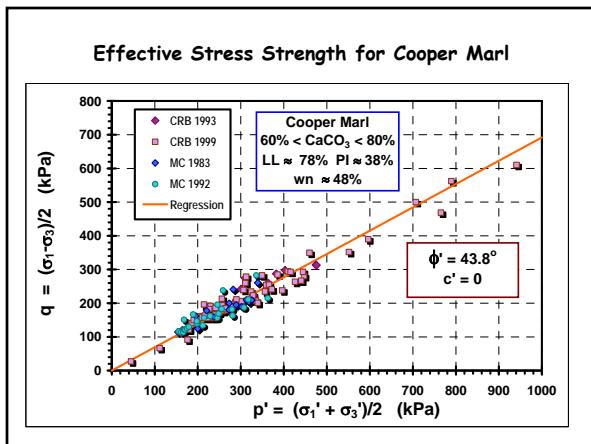
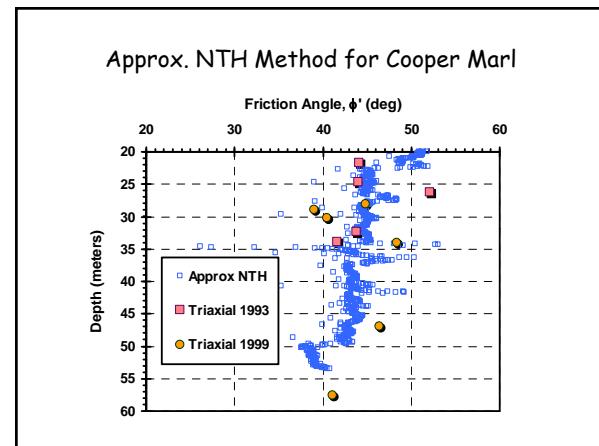
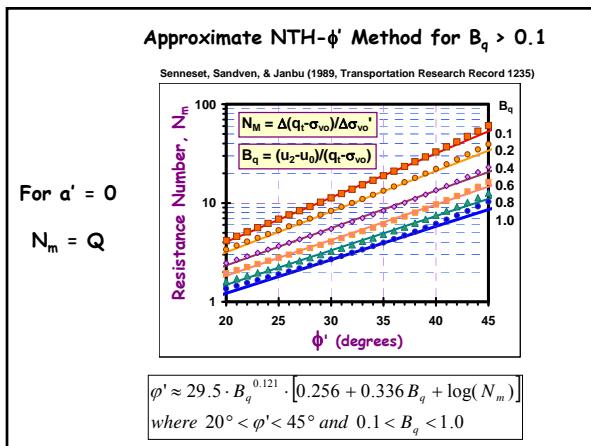
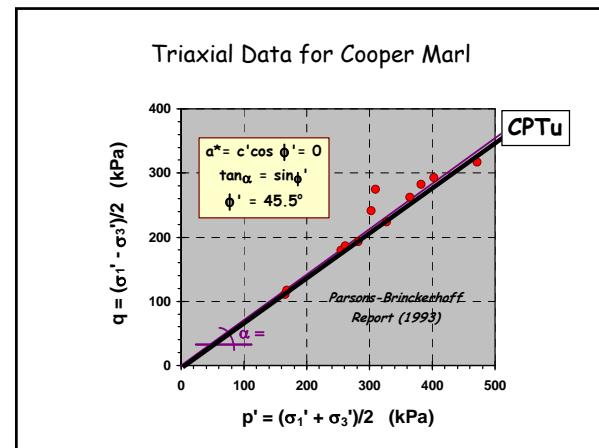
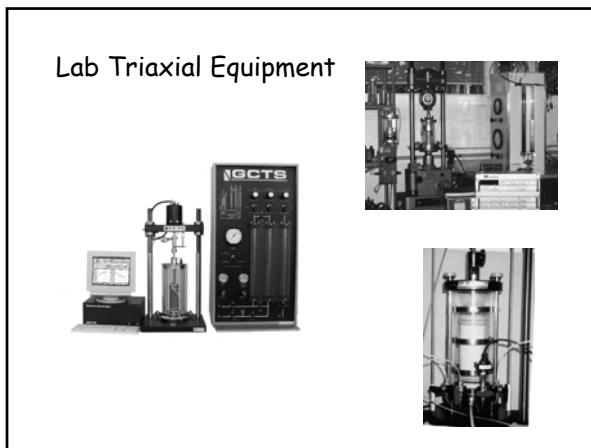


SCPTU soundings advanced by 25-tonne Cone Truck

**Seismic Piezocone Results (C-27)
Cooper River Bridge, Charleston, SC**



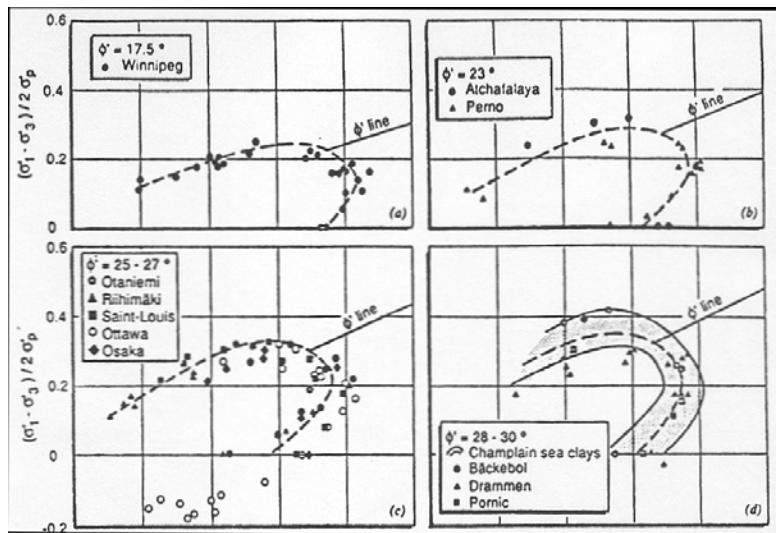




Effective Stress Friction Angle of Natural Clays

Diaz-Rodriguez, Leroueil, & Aleman (ASCE JGE 118, July, 1992)

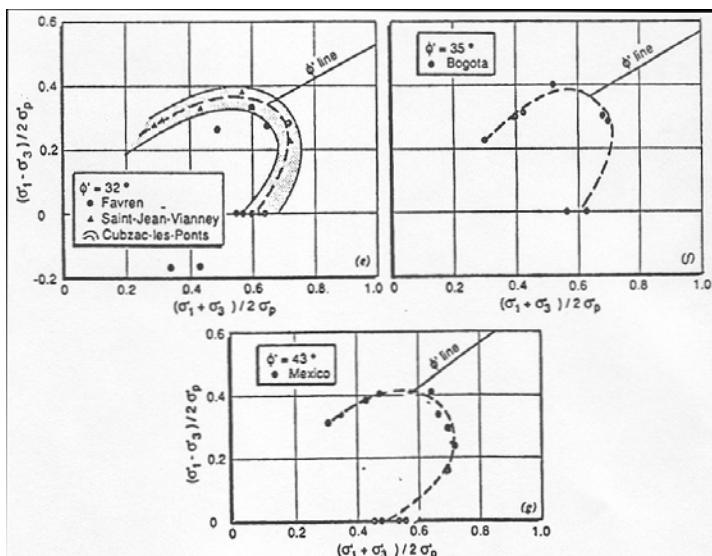
Range Worldwide: $17.5^\circ < \phi' < 43^\circ$

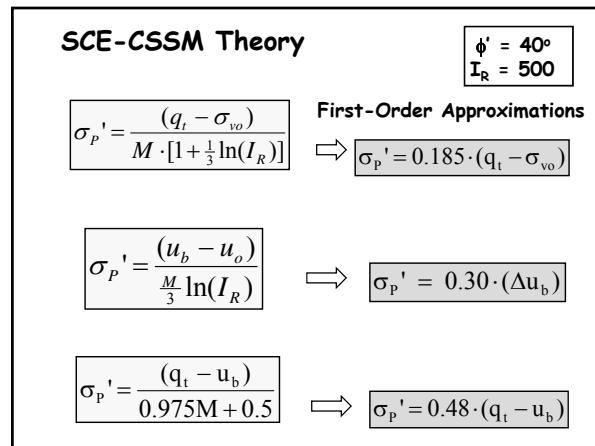
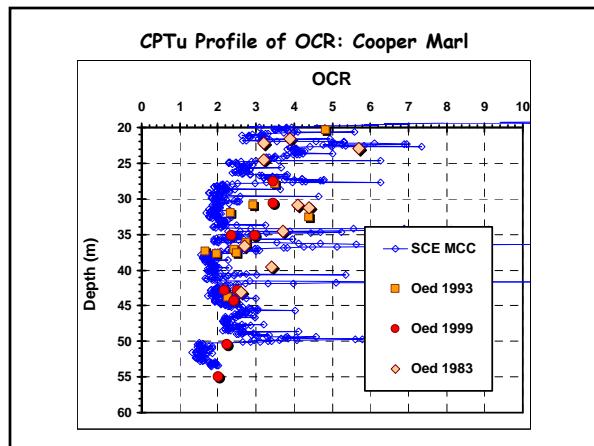
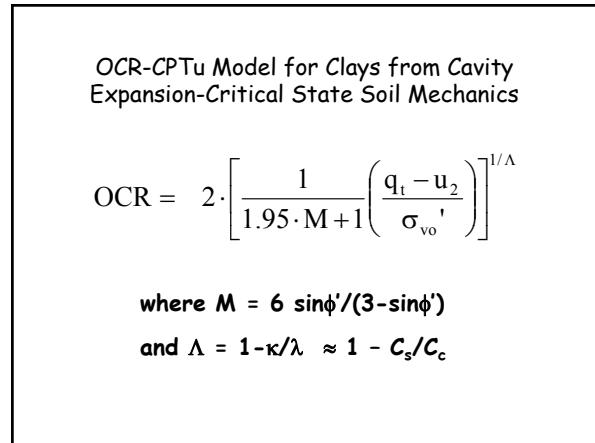
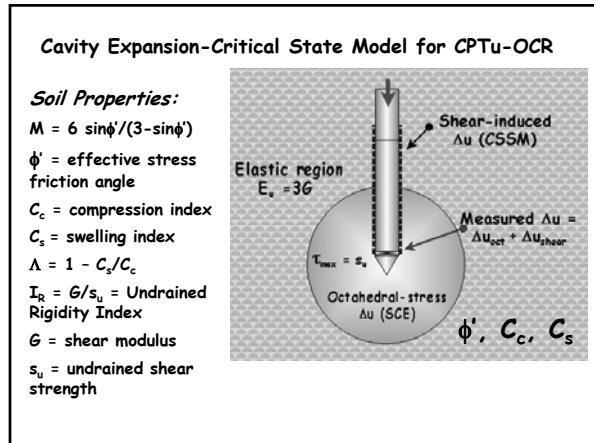
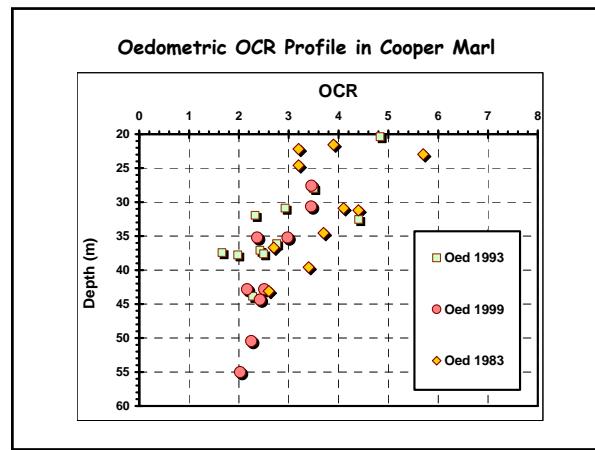
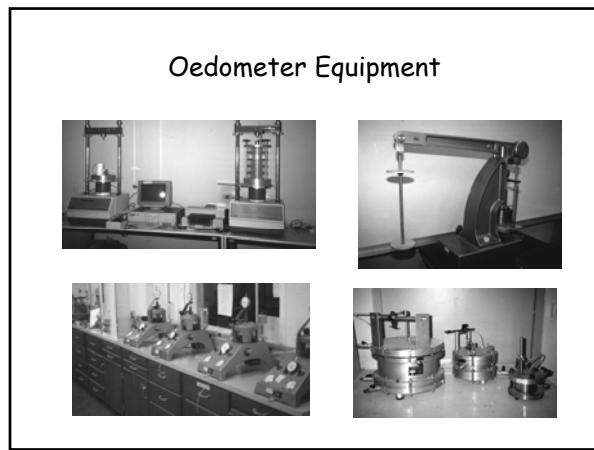


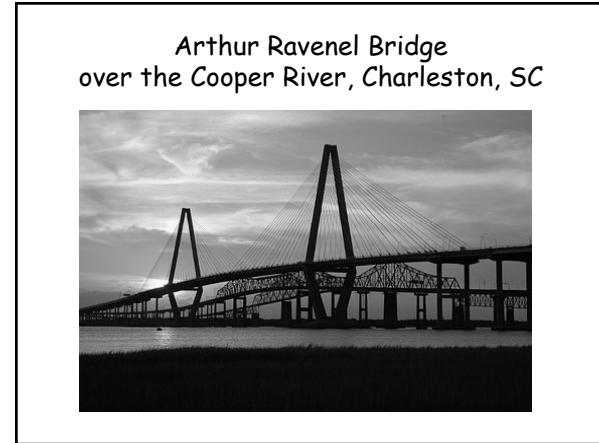
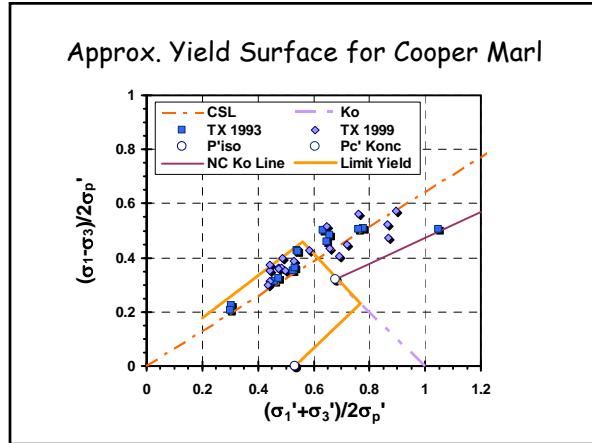
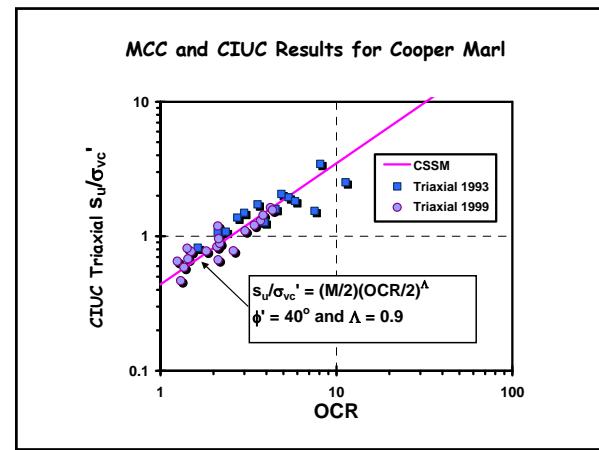
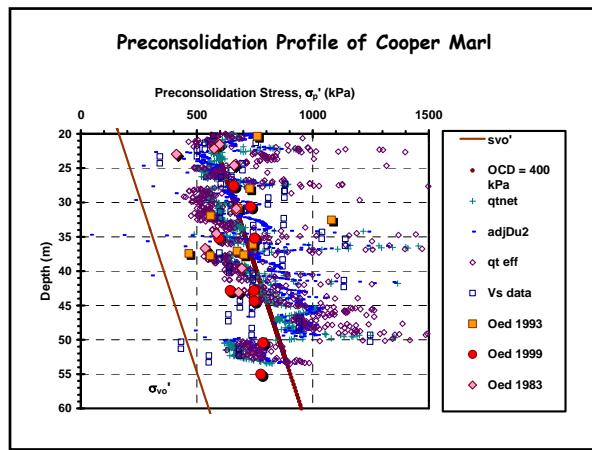
Effective Stress Friction Angle of Natural Clays (Part ii)

Diaz-Rodriguez, Leroueil, & Aleman (ASCE JGE 118, No. 7 1992)

Range Worldwide: $17.5^\circ < \phi' < 43^\circ$







Evaluation of K_o by CPT

Paul W. Mayne
Georgia Institute of Technology

NEWS FLASH (Feb 2004):
NASA Unmanned Rovers "Spirit" and "Opportunity"
find Red Dirt and Stones on Mars (US\$820 Million)



Maui

NASA Polar Lander Missions Searching for water on Mars

LAUNCHED: Jan 3, 1999

LOST: Dec 3, 1999

<http://mars3.jpl.nasa.gov/msp98/lander/>



MARS Spacecraft Weight

Total: 576 kg (1,270 pounds)
Lander: 290 kg (639 pounds)
Propellant: 64 kg (141 pounds)
Cruise Stage: 82 kg (181 pounds)
Aeroshell & Heat Shield: 140 kg (309 pounds)

INTERNATIONAL HERALD TRIBUNE

B8 Sunday, April 1, 2001 ****

Top science stories

ASTRONOMY

1. Lost and found on Mars

The Pentagon's National Imagery and Mapping Agency and Lockheed-Martin lost NASA's Mars Polar Lander, a \$165 million spacecraft lost in 1999 as it tried to land on the Red Planet. NASA and the military assumed the craft had concluded its final descent, but conclusive evidence of Mars' whereabouts was being evaluated by NIMA, which analyzes photos from spy satellites and other sources, appear to show the lander.

Zurich, Saturday-Sunday, October 2-3, 1999

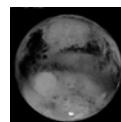
A Little Metric Misstep Cost NASA \$125 Million

Mars Craft Lost to Measurement Blunder

Space agency and Lockheed-Martin officials said the full details of how the mistake occurred were not known. But basically, Lockheed was providing the Jet Propulsion Laboratory with data on the weight of the Mars lander as the spacecraft by its thrusters, which are fired periodically. The data were based in pounds, not kilograms.

However, scientists at the Jet Propulsion Laboratory assumed the figure was in newtons and incorporated it into computer models that were used to calculate the spacecraft's position and direction.

NASA \$165,000,000 Units Blunder



- ❑ "Lockheed was providing the Jet Propulsion Lab with data on the amount of force to the spacecraft by its thrusters."
- ❑ "The data were based in pounds"
- ❑ "However, scientists at the JPL assumed the figures were in Newtons and [used them] in computer models"

$$1 \text{ pound (lb)} = 4.45 \text{ Newtons (N)}$$

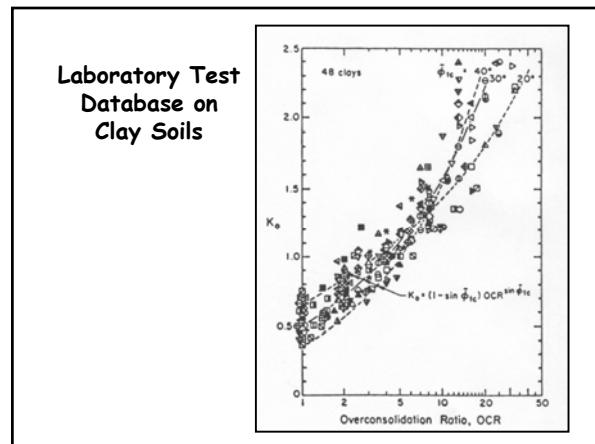
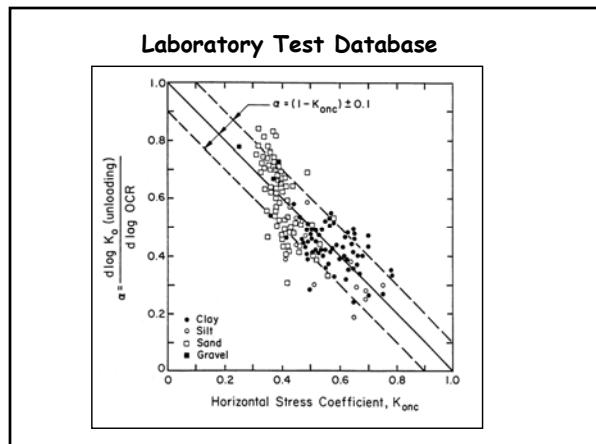
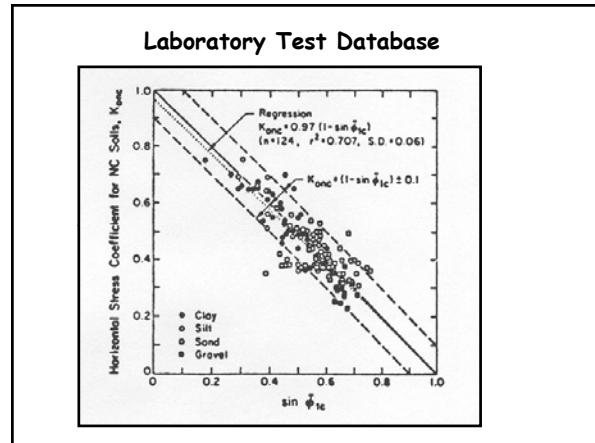
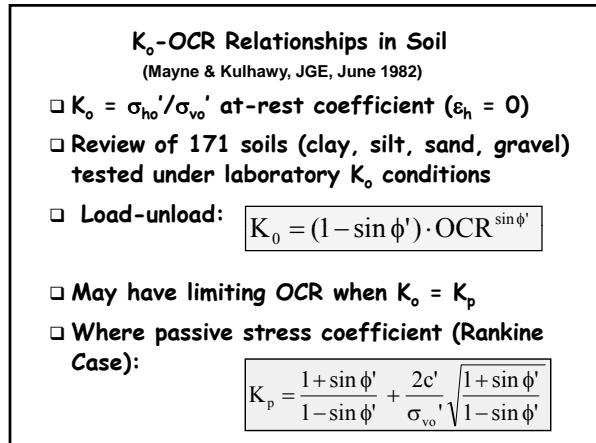
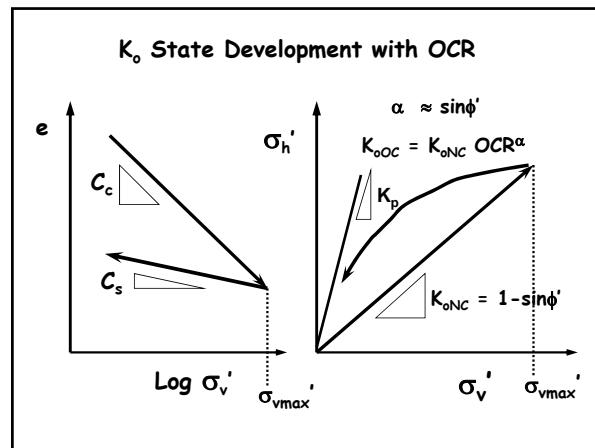
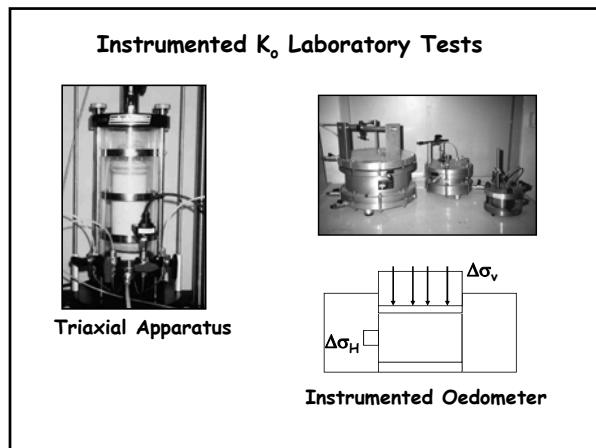
K_o State of Stress in the Ground

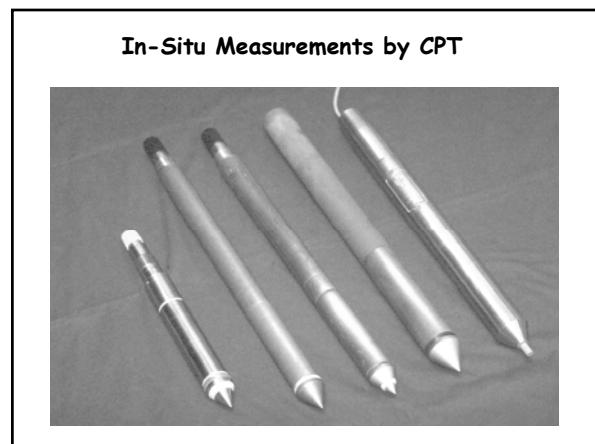
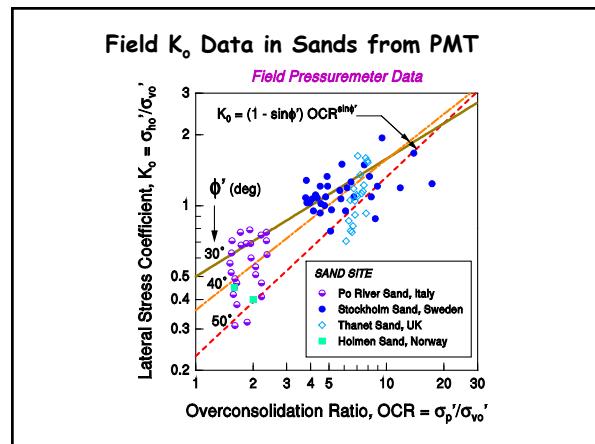
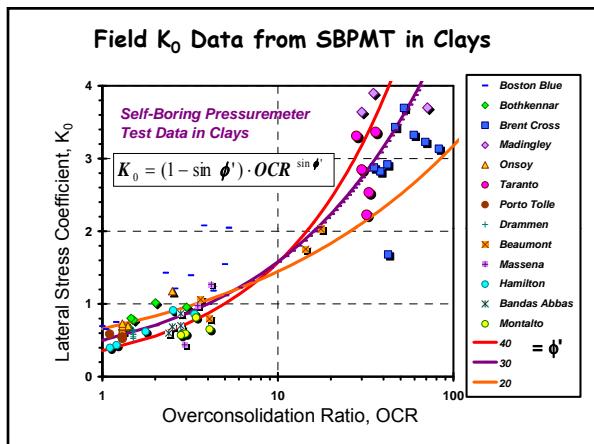
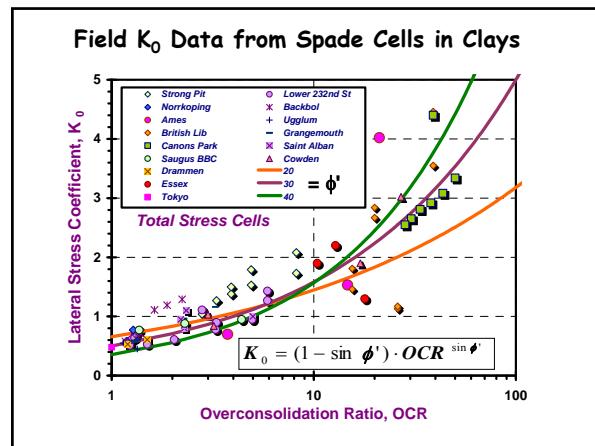
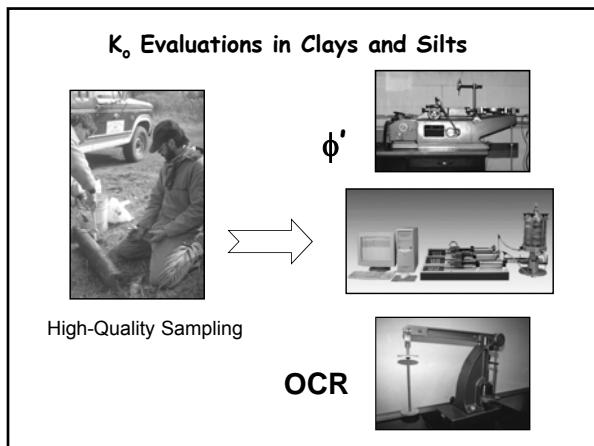


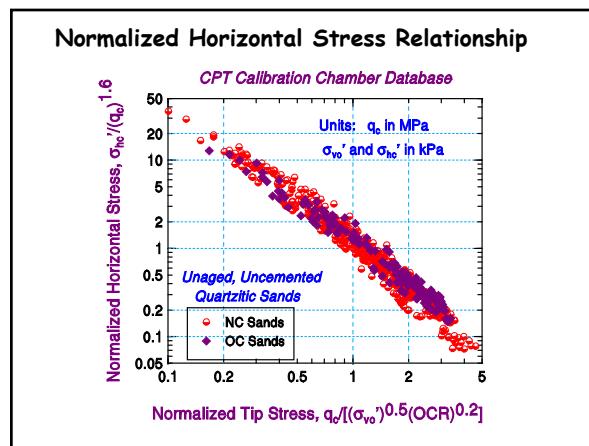
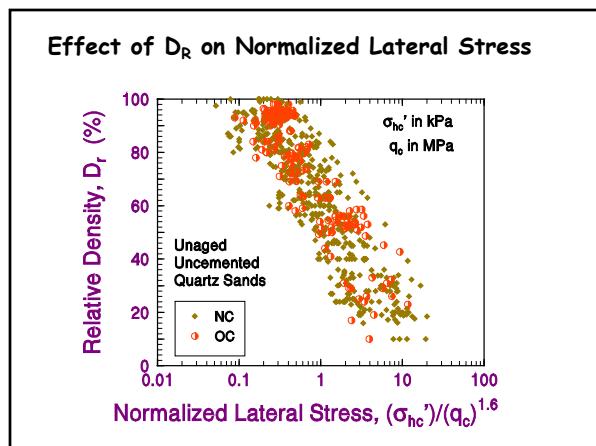
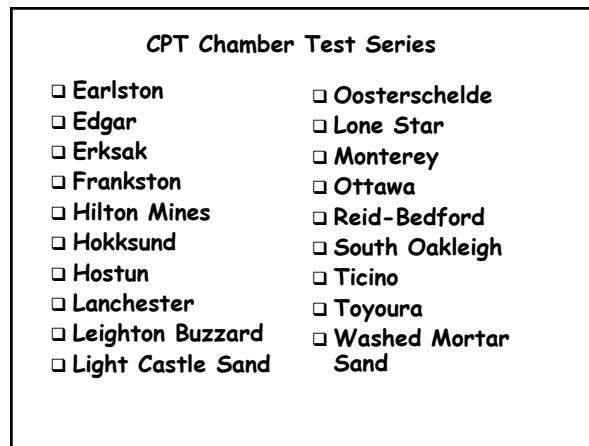
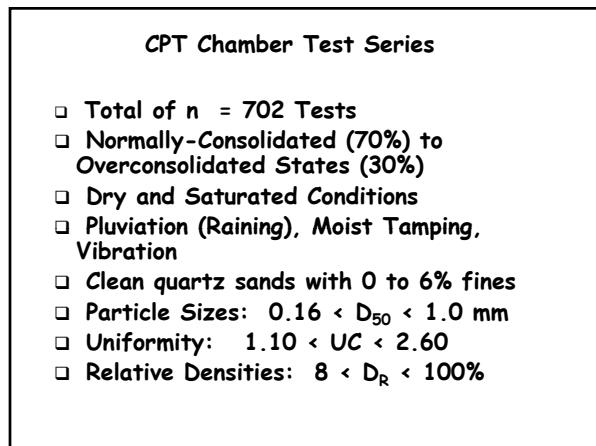
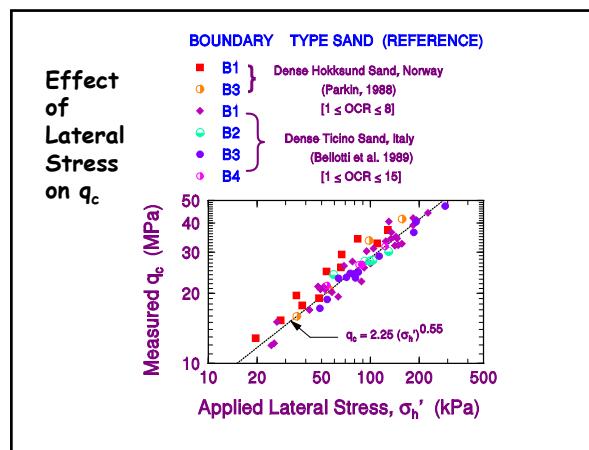
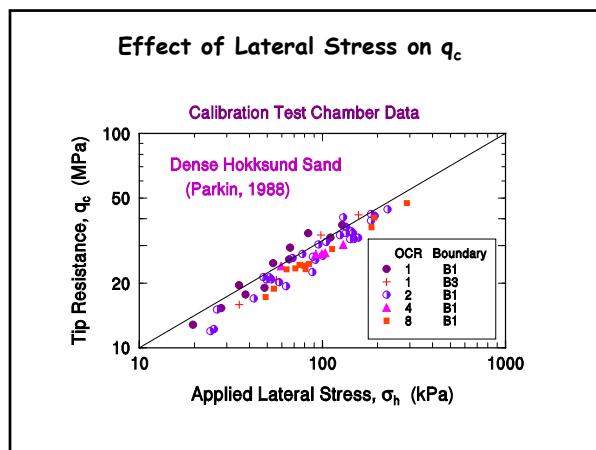
$$\sigma_{vo}' = \sigma_{vo} - u_o$$

$$\sigma_{ho}' = K_o \sigma_{vo}'$$

Geostatic Stress State







CPT Methodology for K_o and OCR in sands

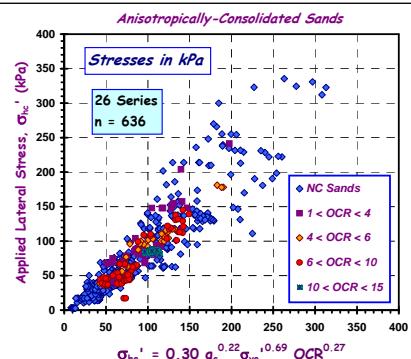
- Calibration Chamber Database Regression
Equation ($n = 705$ data points; $r^2 = 0.871$):

$$K_o = 1.33 q_t^{0.22} (\sigma_{vo}')^{-0.31} OCR^{0.27}$$
- Relationship between K_o and OCR a priori:

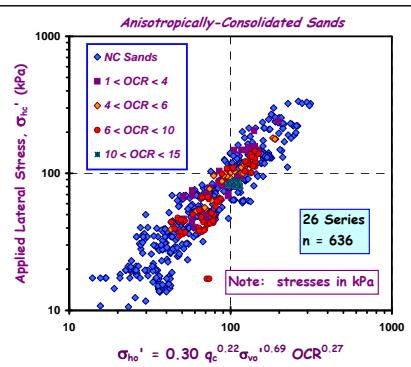
$$K_o = (1 - \sin\phi') OCR \sin\phi'$$
- Evaluate Effective Stress Friction Angle from Kulhawy & Mayne (1990): Soil Properties Manual

$$\phi' = 17.6^\circ + 11 \log [q_c / (\sigma_{vo}')^{0.5}] \quad [\text{bars}]$$

Calibration Chamber Database



Calibration Chamber Database



CPT Methodology in Clean Quartz Sands

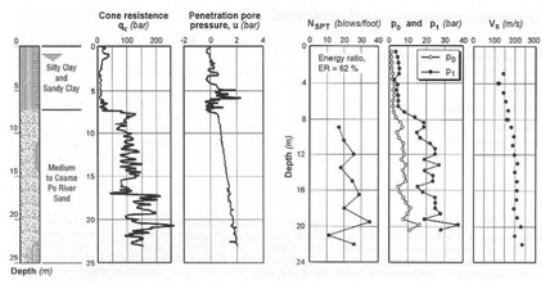
- Combine K_o equations
 - Overconsolidation Ratio, OCR:
- $$OCR = \left[\frac{1.33}{K_{oNC}} \frac{q_t^{0.22}}{(\sigma_{vo}')^{0.31}} \right]^{\frac{1}{\alpha-0.27}}$$
- where $K_{oNC} = 1 - \sin\phi' = 1 - \alpha$
- Tip Stress in MPa and σ_{vo}' in kPa
- Effective Friction Angle from CPT

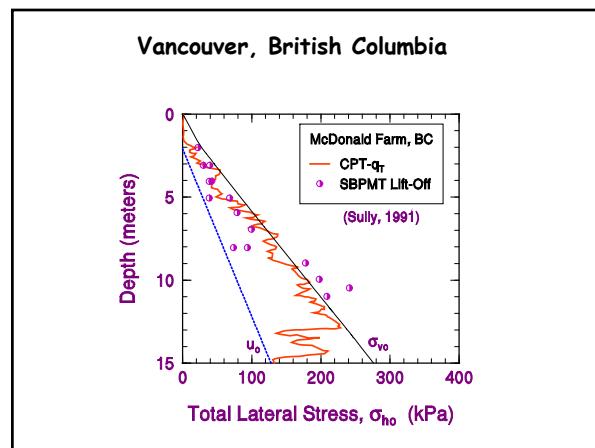
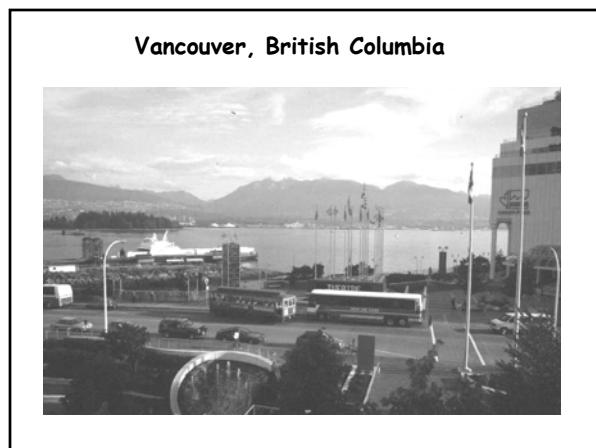
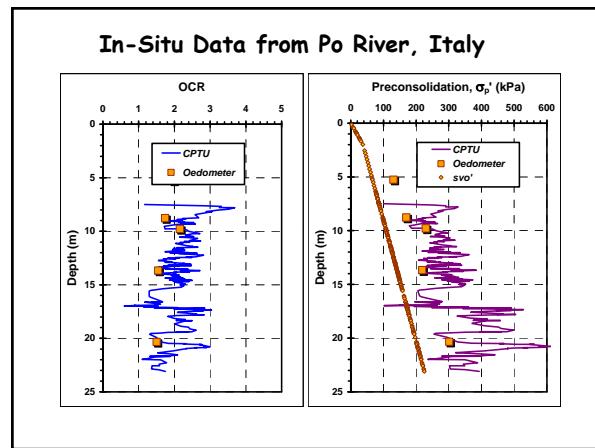
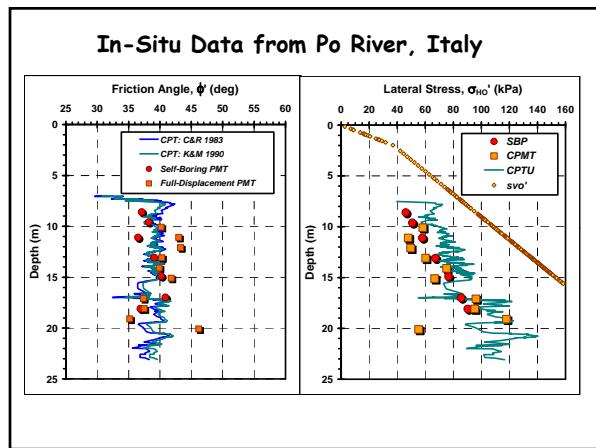
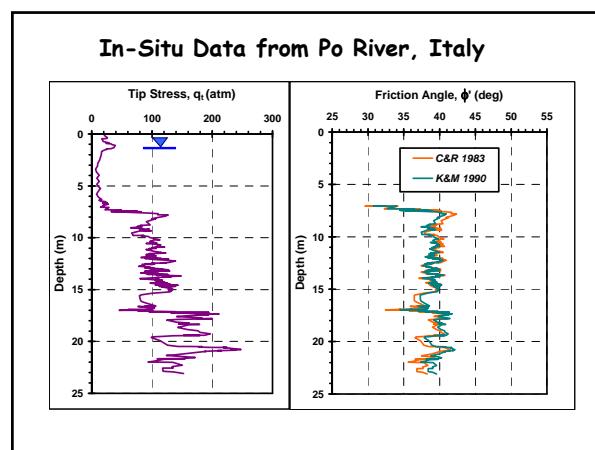
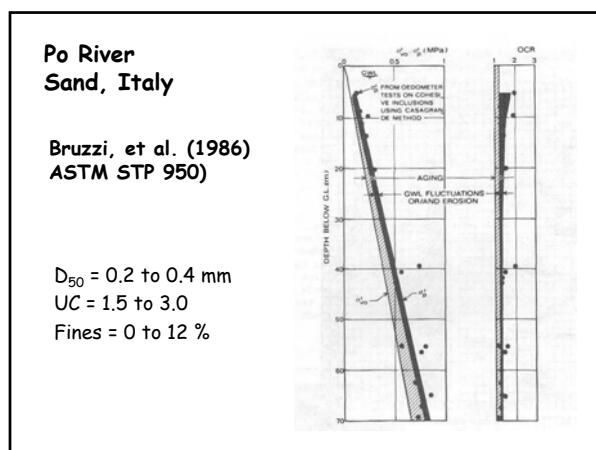
Po River Valley, Italy

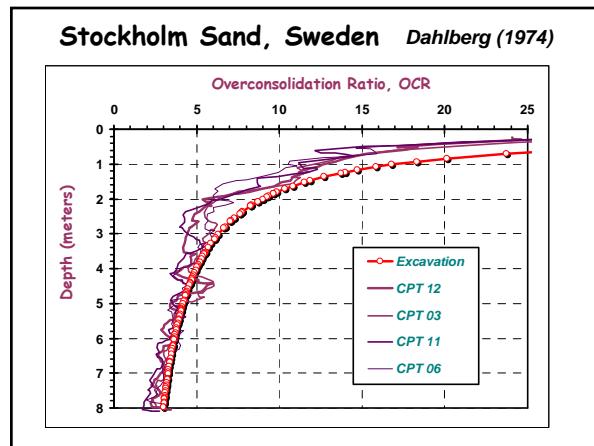
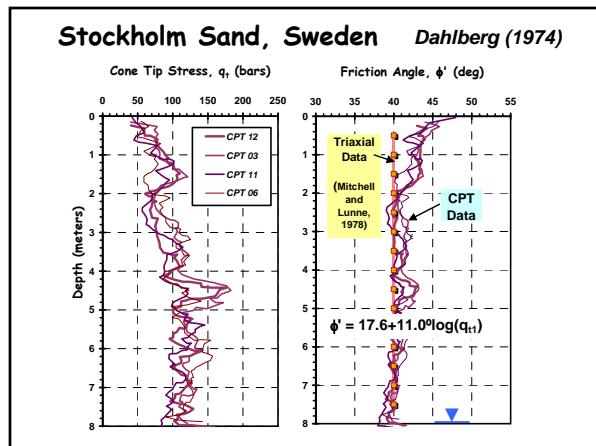
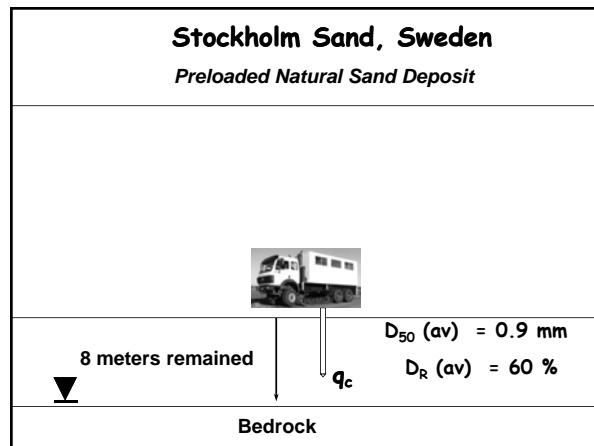
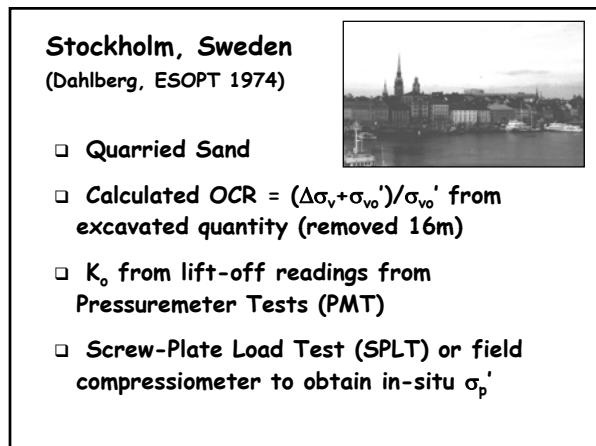
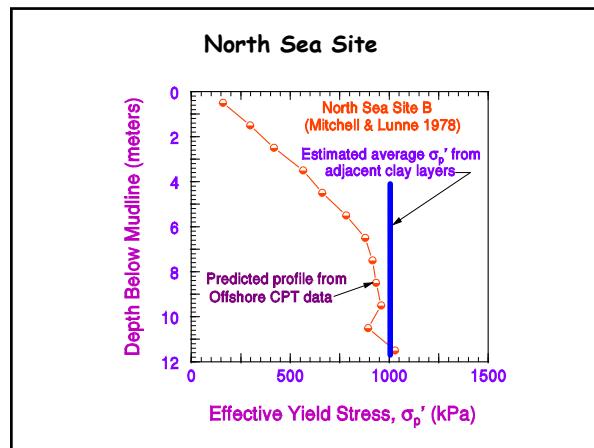
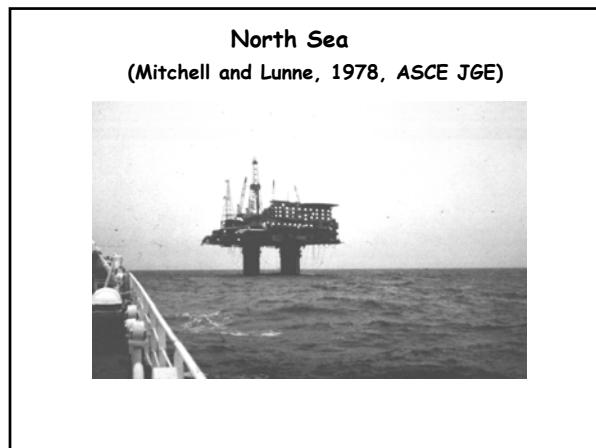


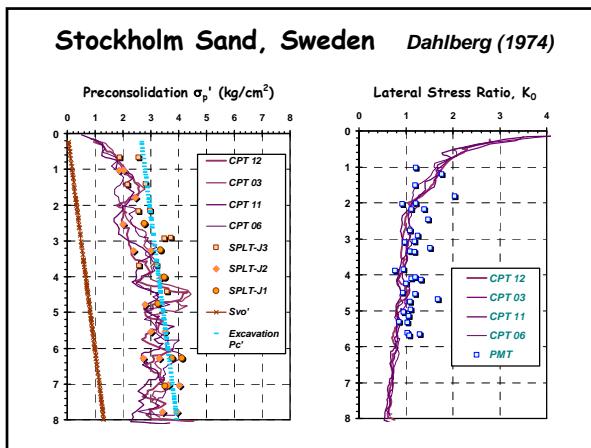
In-Situ Data from Po River, Italy

Ghionna, et al. (1995) Int'l. Symposium Pressuremeters ISP5



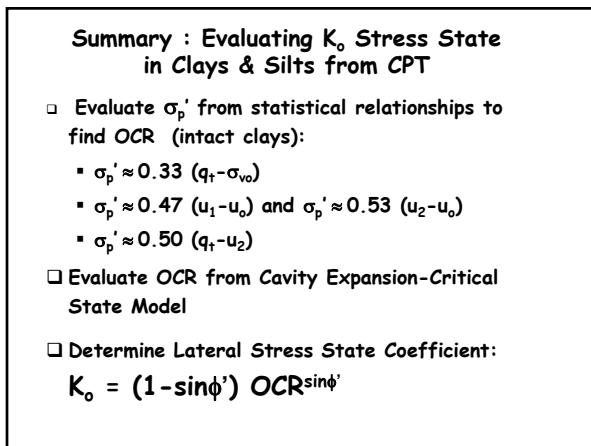






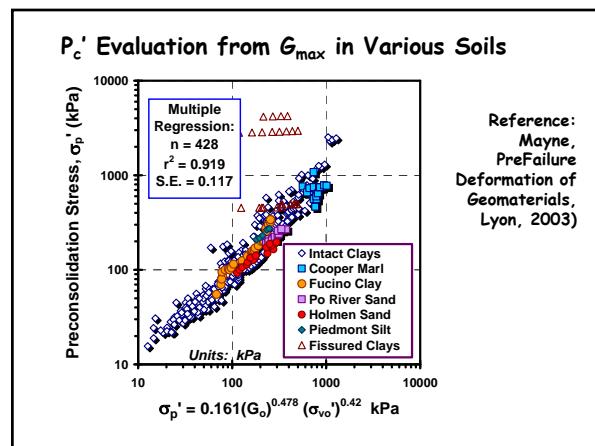
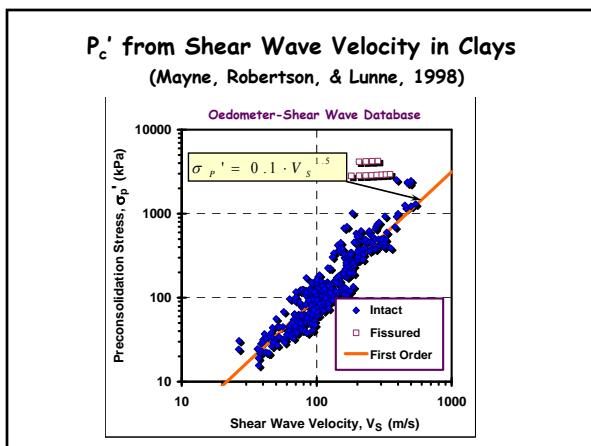
Summary : Evaluating Geostatic Stress State in Clean Quartz Sands from CPT

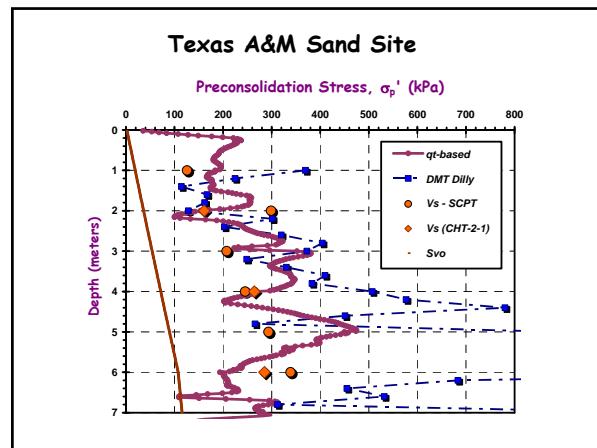
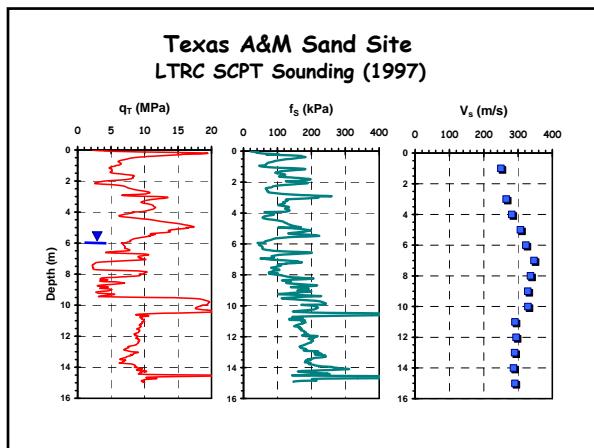
- CPT Database Compiled from 30 Separate Calibration Test Series
- Quartz Sands, unaged, uncemented
- Assume a priori K_o - OCR relationship
- Application to the few field case records (Italy, Sweden, North Sea, Canada)



Future Research & Directions (K_o)

- Database for carbonate sands (cemented sands, coralline sand): Quiou Sand, Dogs Bay Sand
- Use of Sets of Polarized and Directional Shear Waves (V_{shh} , V_{svh} , V_{shv}) since only 2 of 3 principal stresses affect V_s .
- Empirical K_o evaluation from neural network or multiple regression analysis on SCPTu data.





For intact clays, it is useful to utilize all three equations [28], [29], and [30], as redundancy can be helpful in geotechnical site characterization. If the three methods show consensus, then this helps to validate a "well-behaved" clay and encourages the use of these relationships. If the methods show disparities, then a closer examination and scrutiny of the lab and/or field data may be warranted, perhaps justification that additional tests and investigation should be conducted. In the case of fissured clays, the use of [28] and [29] will not apply, leaving [30] as a rough means to evaluate the magnitude of σ_p' . If highly structured clays or geomaterials with large amounts of "unusual" mineralogy exist (i.e., calcite, diatoms, forams, etc.), it may be possible to re-tune these equations for the particular geologic formation attributes (e.g., Mayne 2005).

For the general case of evaluating the preconsolidation stress of natural soils, including sands, silts, clays, and mixed soil types, Figure 31 offers a preliminary method that extends equation [28] to the form:

$$[31] \quad \sigma_p' = 0.33 (q_t - \sigma_{vo})^{m'} (\sigma_{atm}/100)^{1-m'}$$

where the *exponent m'* apparently increases with fines content and/or decreases with mean grain size. Based on available observations, the parameter $m' \approx 0.72$ in clean quartz sands, 0.8 in silty

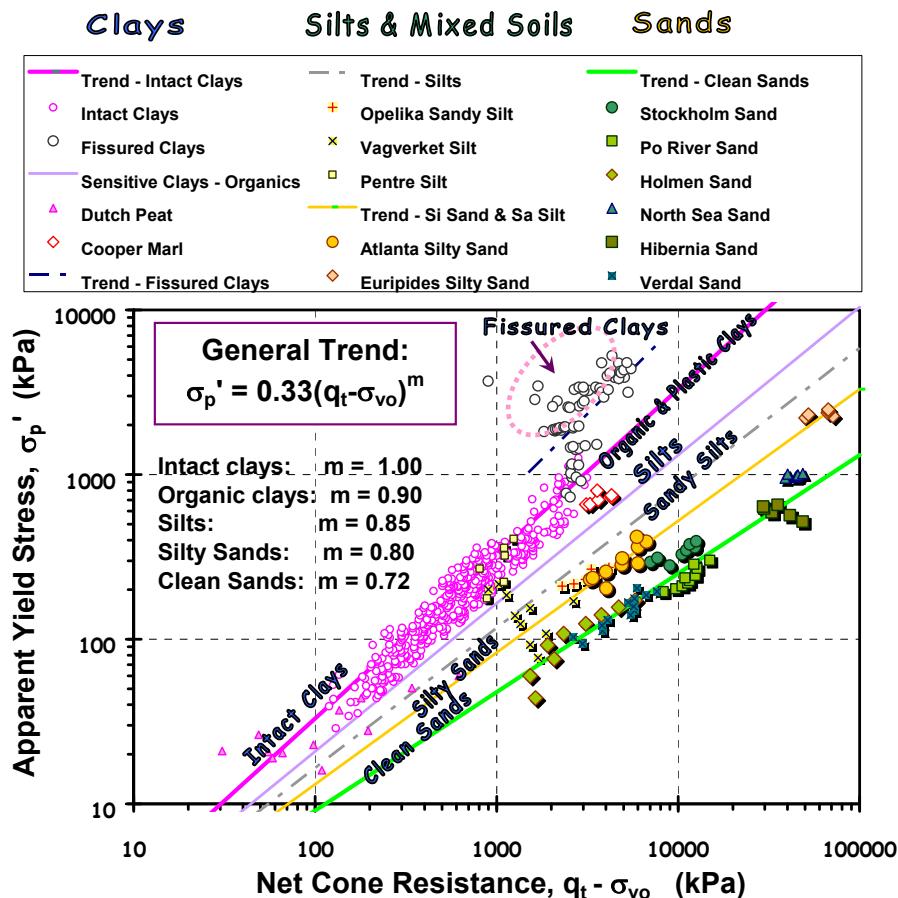
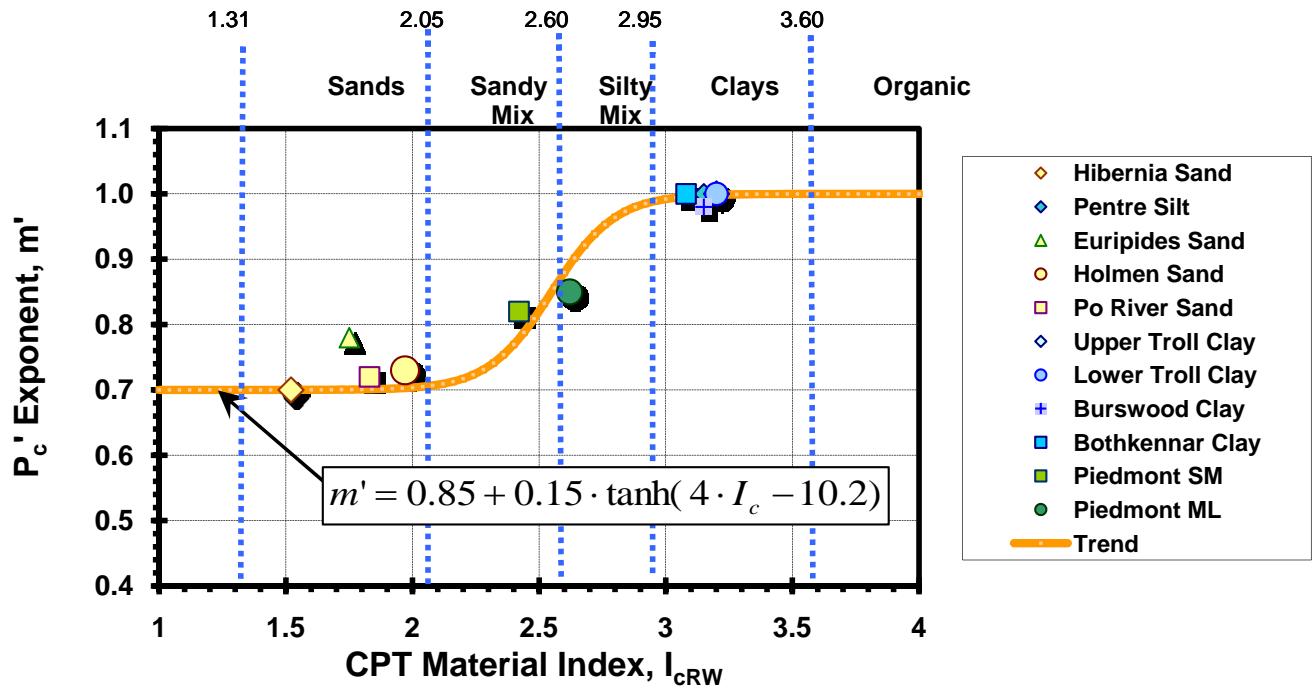


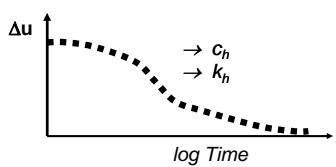
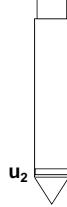
Figure 31. General Approach to P_c' interpretation of soils by CPT net cone resistance.
(Mayne, et al. 2009, SOA-1 Proc. 17th ICSMGE)



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

Coefficient of Consolidation and Permeability from Piezo-Dissipations

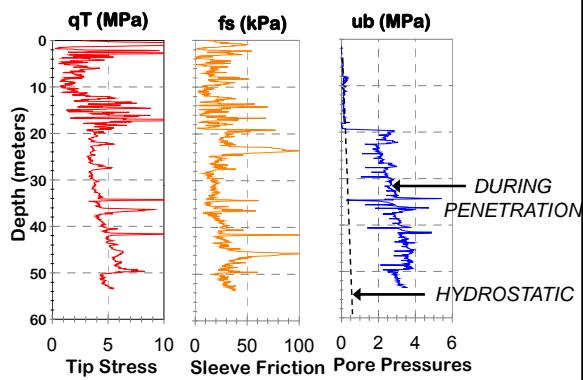
Paul W. Mayne, Georgia Tech
Susan E. Burns, Univ. of Virginia



Porewater Pressure Dissipation

- Stop in penetration (rod break ~ 1 meter)
- Measure pore pressure = fctn (time)
- Excess PWPs decrease to zero ($\Delta u \rightarrow 0$)
[total $u_{\text{meas}} \rightarrow u_0$ hydrostatic]
- Time to completion depends on coefficient of consolidation (c_h) and permeability (k_h)
- Monitor either u_1 or u_2 position

Cooper River Bridge, Charleston, South Carolina



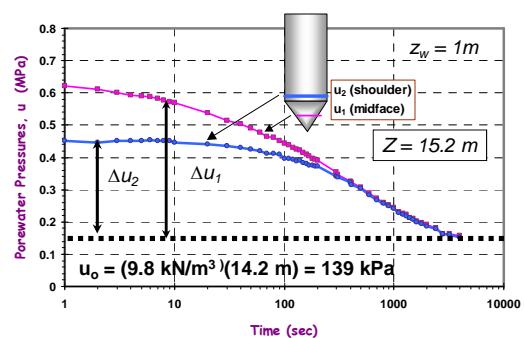
Theoretical Solutions to PiezoDissipation

- Tortensson (1977) - Cavity Expansion
- Randolph & Wroth (1979) - Cavity Expansion
- Battaglio et al. (1981) - CE + Skempton's A_f
- Jones & Van Zyl (1981) - Empirical Approach
- Senneset et al. (1982) - Consolidation Theory
- Baligh & Levadoux (1986) - Strain Path
- Gupta & Davidson (1986) - Series of cavities
- Housby & Teh (1988, 1991) - Strain Path

Strain Path Method (Housby & Teh, 1988, 1991)

- Position of Element (tip, midface, shoulder, upper shaft)
- Often applied with measured time to 50% completion, t_{50} (single point)
- Need estimate of undrained rigidity index, $I_R = G/s_u$
- Monotonic Decay of Δu with time only.

Dissipations in Soft Varved Clay at Amherst NGES



Strain Path Method

Degree of Consolidation:

$$U = 1 - \Delta u / \Delta u_i$$

where $\Delta u_i = u_{\text{meas}} - u_0$
during penetration;

Δu = remaining excess
porewater pressure.

T^* = modified time
factor (SPM) where

$$c_h = T^* a^2 I_R^{0.5} / t$$

Houlsby & Teh (Geot. 1991)

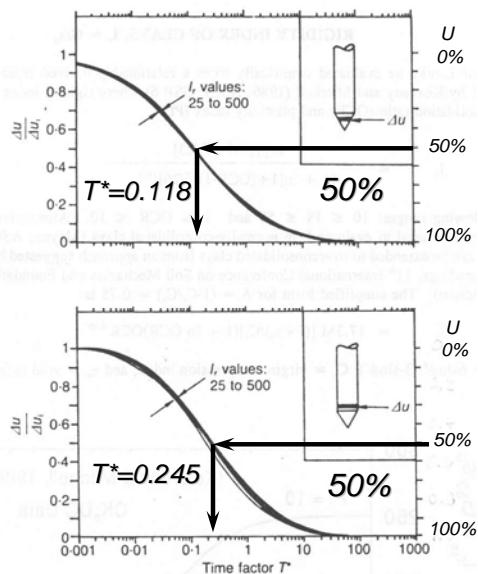


Fig. 19. Normalized dissipation curves plotted against T^*

Monotonic Piezocone Dissipation (SPM)

$$c_h = \frac{(T_{50}^*) a^2 \sqrt{I_R}}{t_{50}}$$

where $T_{50}^* = 0.118$ for Type 1

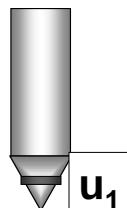
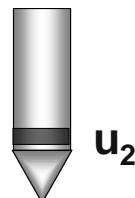
$= 0.245$ for Type 2

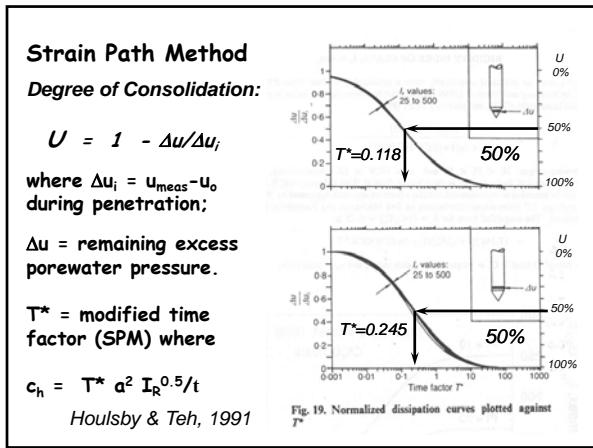
a = probe radius

$= 1.78$ cm for 10-cm² cone

$= 2.20$ cm for 15-cm² cone

$I_R = G/s_u$ = Rigidity Index





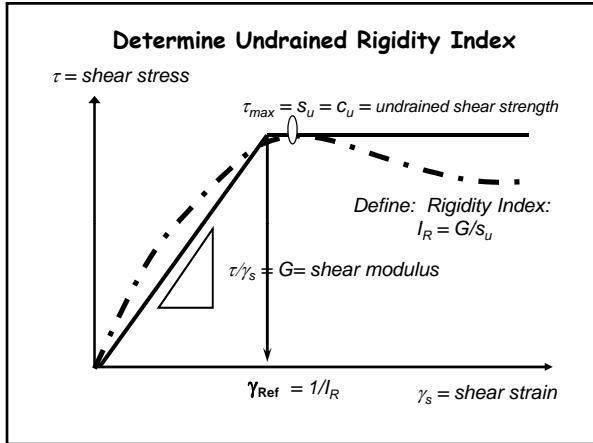
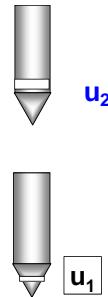
Monotonic Piezocone Dissipation

$$c_h = \frac{(T_{50}^*) a^2 \sqrt{I_R}}{t_{50}}$$

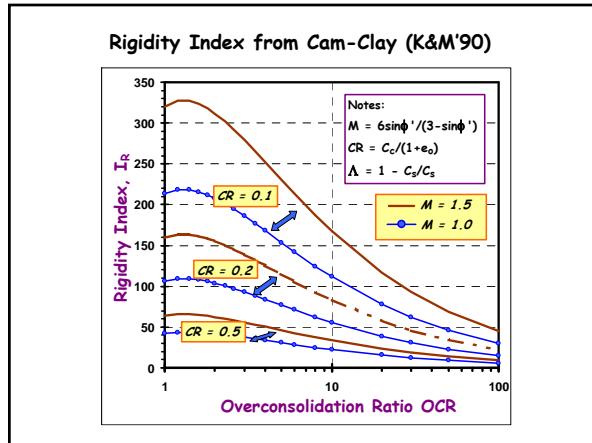
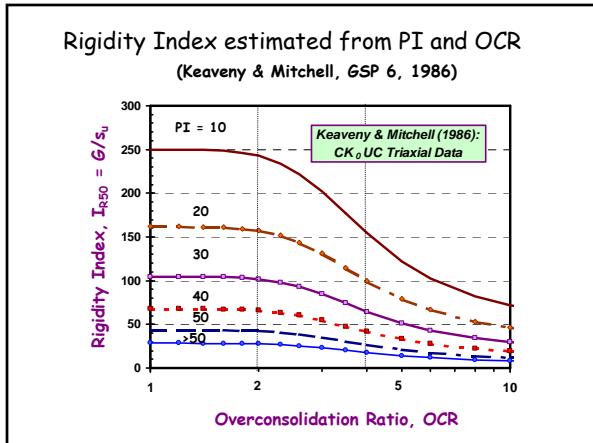
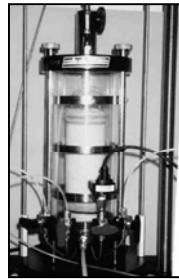
where $T_{50}^* = 0.118$ for Type 1
 $= 0.245$ for Type 2

a = probe radius
 $= 1.78$ cm for 10-cm² cone
 $= 2.20$ cm for 15-cm² cone

$I_R = G/s_u$ = Rigidity Index

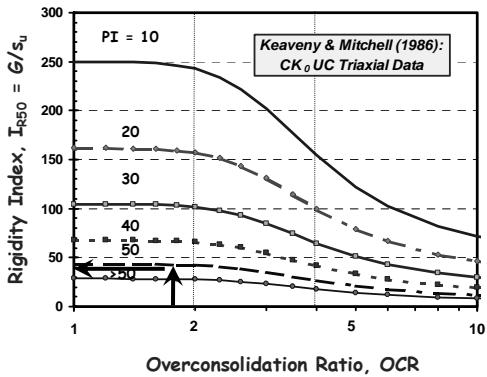


Laboratory Tests for Rigidity Index



Case Study: Bothkennar Dissipations

- Using $OCR = 1.82$ and Plasticity Index $PI = 50$
- Estimated Rigidity Index, $I_R = 38$



Approximation:

$$I_R = \frac{\exp\left[\frac{137 - PI}{23}\right]}{\left\{1 + \ln\left[1 + \frac{(OCR - 1)^{3.2}}{26}\right]\right\}^{0.8}} \quad \text{for } 10 < PI < 50 \quad \text{and } 1 < OCR < 10$$

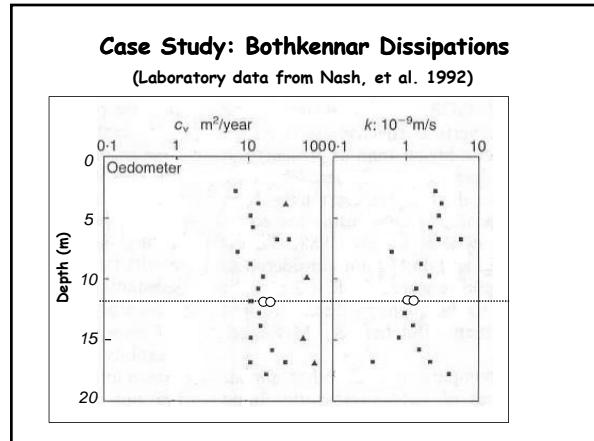
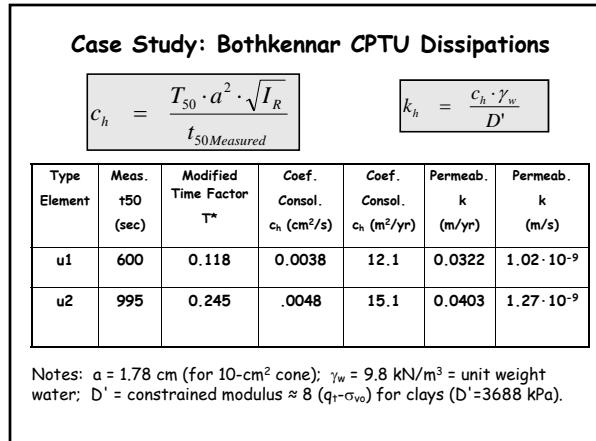
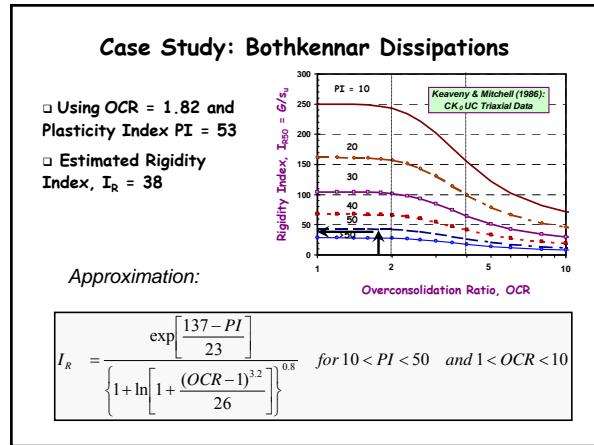
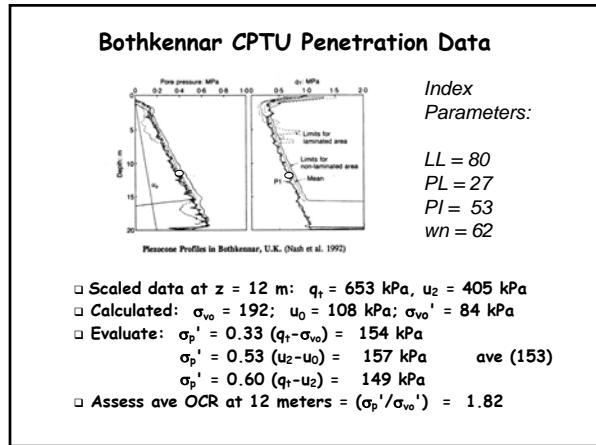
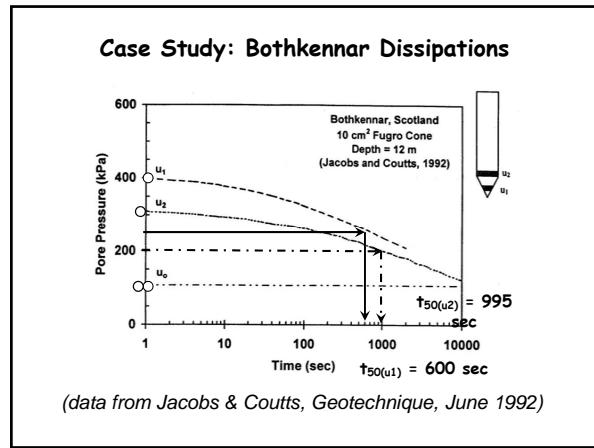
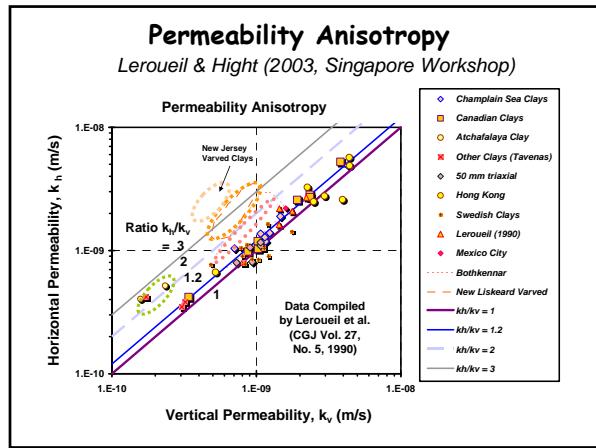
Case Study: Bothkennar CPTU Dissipations

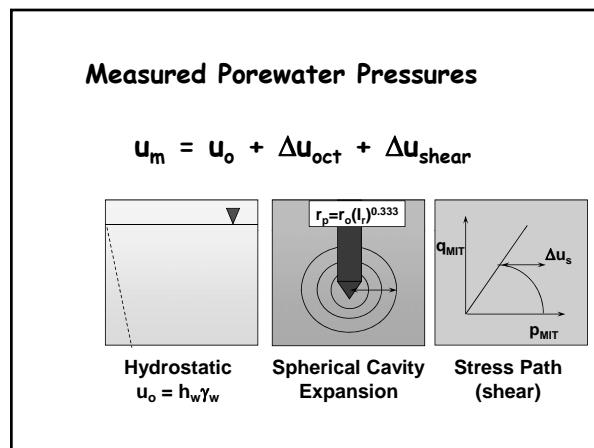
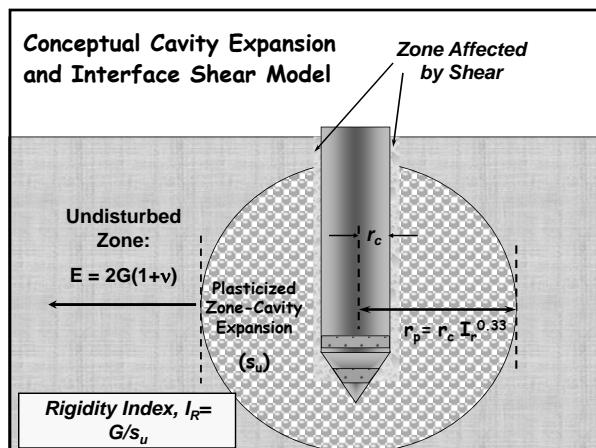
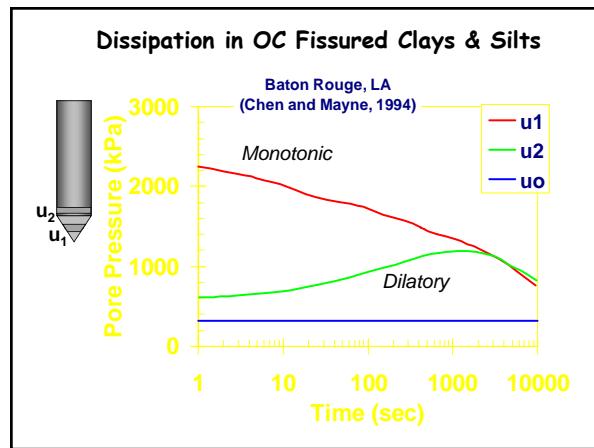
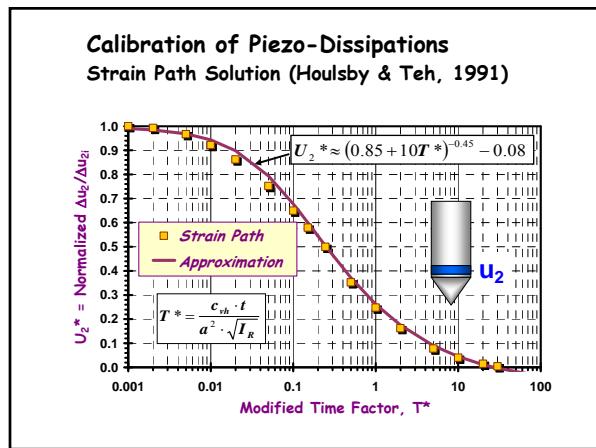
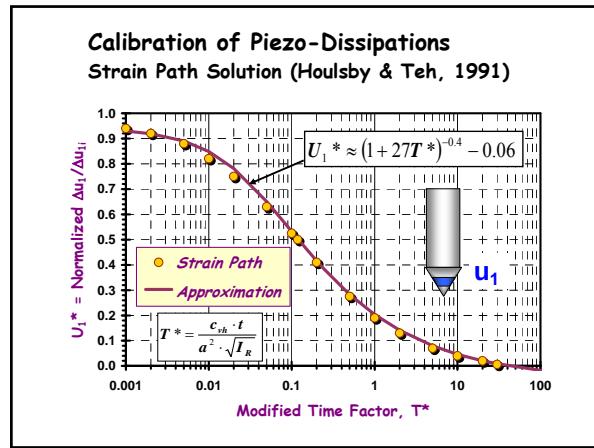
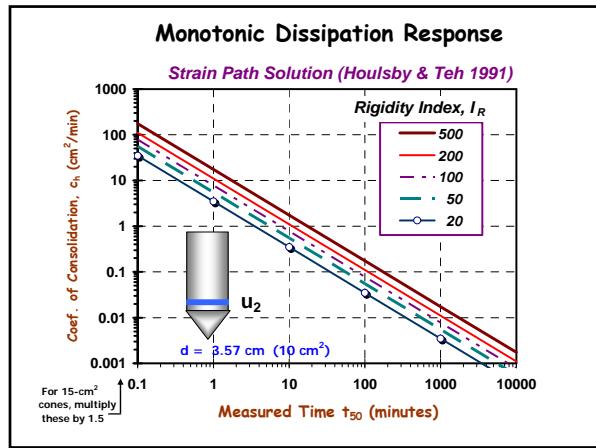
$$c_h = \frac{T_{50} \cdot a^2 \cdot \sqrt{I_R}}{t_{50\text{Measured}}}$$

$$k_h = \frac{c_h \cdot \gamma_w}{D'}$$

Type Element	Meas. t ₅₀ (sec)	Modified Time Factor T*	Coef. Consol. c _h (cm ² /s)	Coef. Consol. c _h (m ² /yr)	Permeab. k (m/yr)	Permeab. k (m/s)
u1	600	0.118	0.0038	12.1	0.0322	$1.02 \cdot 10^{-9}$
u2	995	0.245	.0048	15.1	0.0403	$1.27 \cdot 10^{-9}$

Notes: $a = 1.78$ cm (for 10-cm² cone); $\gamma_w = 9.8$ kN/m³ = unit weight water; D' = constrained modulus ≈ 8 ($q_t - \sigma_{vo}$) for clays ($D' = 3688$ kPa).





Uncoupled Linear Consolidation Analysis for Dissipation

- Utilize analytical solution to the 1-d consolidation equation

$$\frac{\partial u}{\partial t} = c_h \frac{1}{r} \frac{\partial u}{\partial r} + c_h \frac{\partial^2 u}{\partial r^2}$$

Boundary conditions:

- Cone is impermeable
- No pore pressure increase outside plasticized zone

Model Formulation

Initial conditions:

- Initial magnitude of pore pressure calculated using SCE and Cam Clay
- Finite difference (1 hour on Pentium II)
- Closed form (Mathcad in 15 secs.)

$$u = \sum_{n=1}^{\infty} B_n e^{-c\alpha_n^2 t} [-Y_0(\alpha_n r) J_0(\alpha_n r_{plastic}) + Y_0(\alpha_n r_{plastic}) J_0(\alpha_n r)]$$

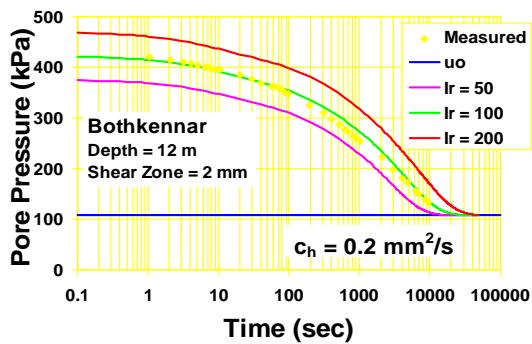
Input Data to CE-CSSM Model

- ϕ' = Effective Stress Friction Angle
- OCR = Overconsolidation Ratio
- σ_{vo}' = Effective Overburden Stress
- u_o = Hydrostatic Pore Pressure
- $I_r = G/s_u$ = Rigidity Index (G = shear modulus and s_u = undrained shear strength)
- r_{cone} = cone radius

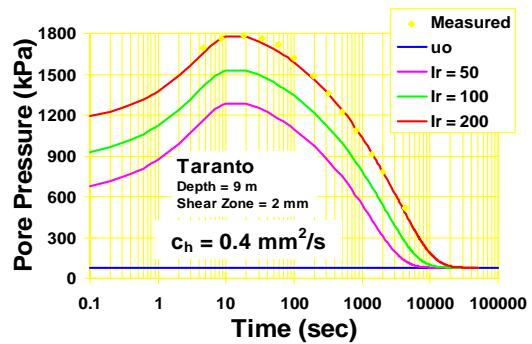
Dissipation Database for Calibration

- 15 clay sites worldwide (Burns & Mayne, Canadian Geotechnical Journal, 1998)
 - 6 soft intact clay sites
 - 9 hard to stiff overconsolidated clay sites (fissured, crusts)
- Applied to Instrumented Driven Pile Foundations (Burns & Mayne, 1999)

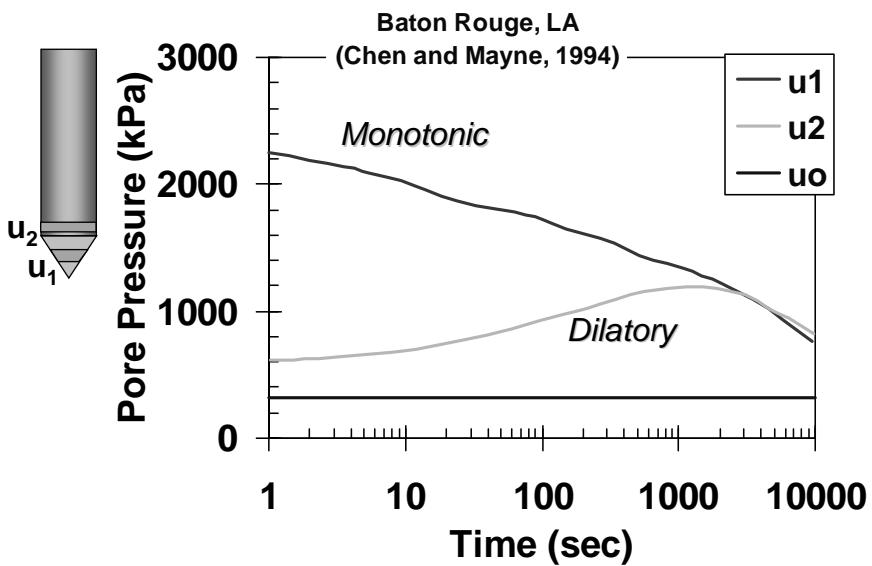
Model Evaluation - Soft Clay



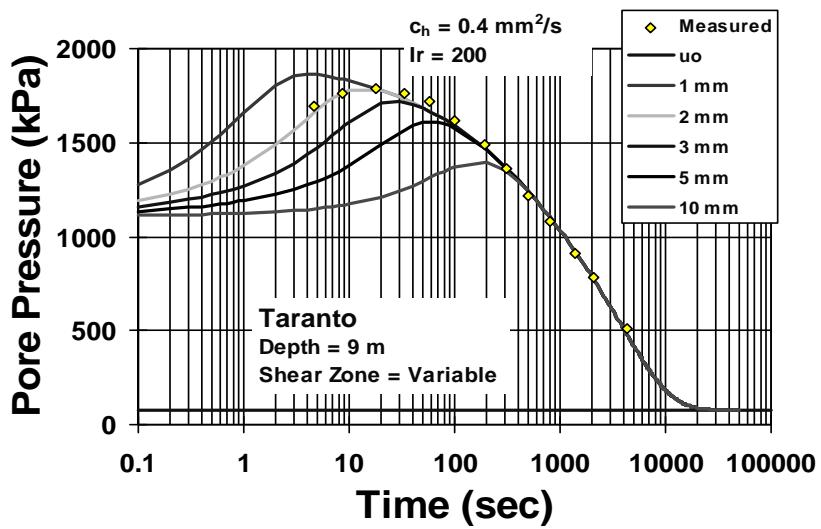
Model Evaluation - Stiff Clay

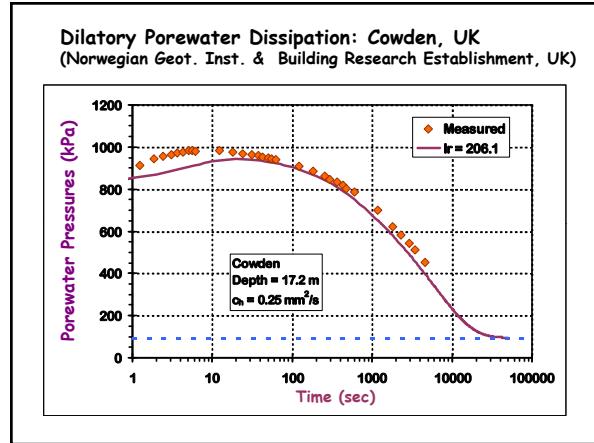
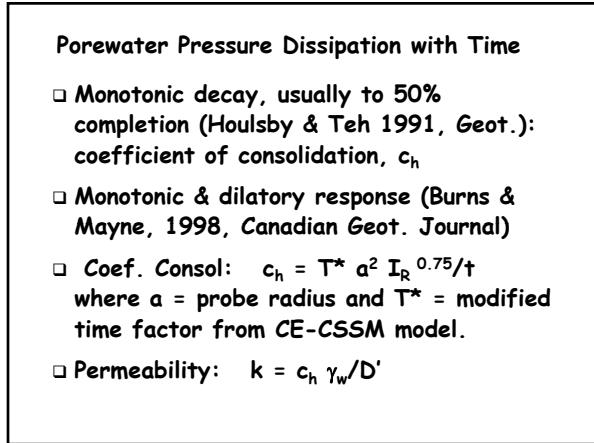
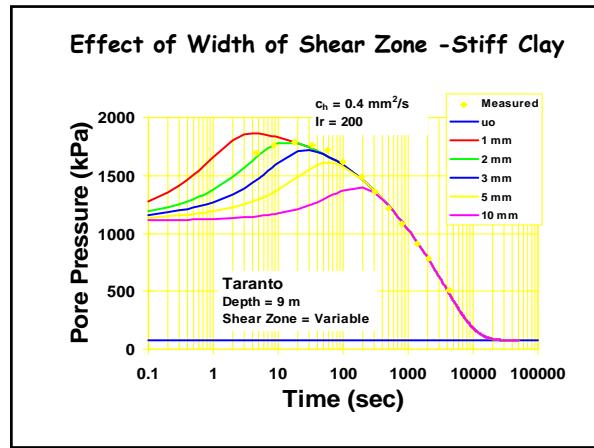
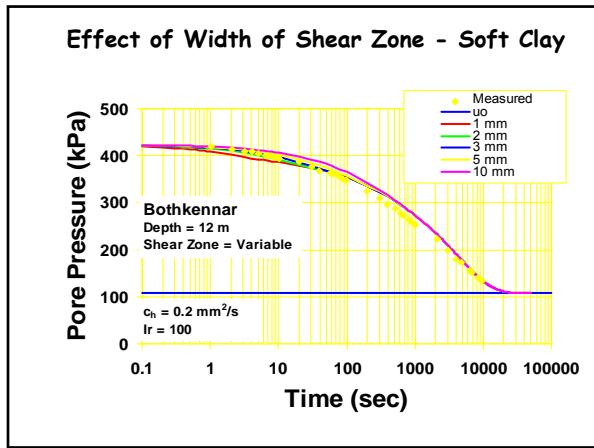


Dissipation in OC Fissured Clays & Silts



Effect of Width of Shear Zone - Stiff Clay

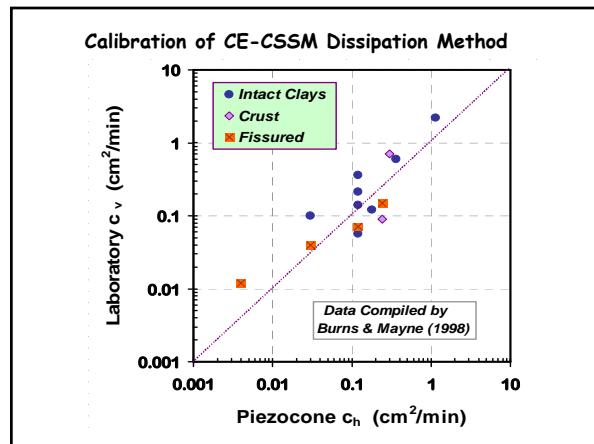




Calibration: Coefficient of Consolidation from CPTu

Site	Field Value (mm^2/s)	Lab Value (mm^2/s)
Bothkennar	0.2	0.3
Drammen	0.2	0.5-1.5
McDonald Farm	1.9	1.8-5.5
Onsøy	0.05	0.1-0.2
Porto Tolle*	0.2	0.3-0.5
St. Alban	0.6	0.3
Amherst	0.4	0.1
Brent Cross	0.0005	0.008-0.03
Canon's Park**	0.25	0.01-0.03
Cowden	0.2	0.05-0.2
Maddingley***	0.05	0.03-0.08
Raquette River	0.5	0.04-0.7
St Lawrence	0.3	0.25-0.8
Strong Pit	0.2	0.06-0.3
Taranto	0.4	0.1-0.25

* 15 mm piezoprobe
**Pile



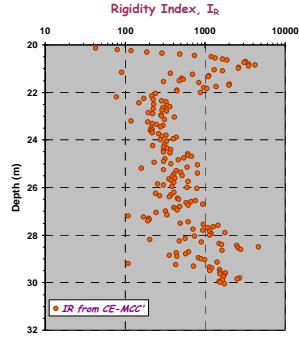
Rigidity Index (I_R) Evaluated from CPTu Data using CE-CSSM

$$I_R = \exp \left[\left(\frac{1.5}{M} + 2.925 \right) \left(\frac{q_t - \sigma_{vo}}{q_t - u_2} \right) - 2.925 \right]$$

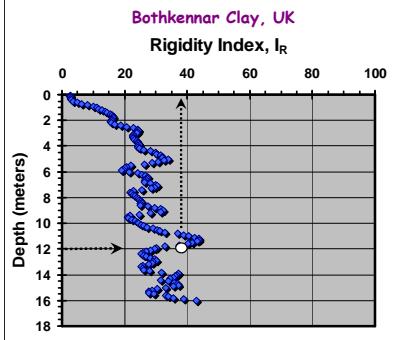
where $M = 6 \sin\phi' / (3 - \sin\phi')$

Very Sensitive to Values Used

CPTu Evaluation of I_R for Cooper Marl, Charleston, SC (Sounding CHBDH2)



CPTu Evaluation of I_R for Bothkennar Clay, UK (Data from Powell, Quarterman & Lunne, 1988)



Approximate Monotonic & Dilatory Piezo-Dissipations (Mayne, In-Situ 2001)

Define $T^* = (c_{vh} \cdot t) / (\alpha^2 \cdot I_R^{0.75})$

$\Delta u_{meas} = \Delta u_{oct} + \Delta u_{shear}$

Dissipation:

$$\Delta u_{meas} \approx \frac{\Delta u_{oct}}{1 + 50T^*} + \frac{\Delta u_{shear}}{1 + 5000T^*}$$

Octahedral Component:

$$\Delta u_{oct} = \frac{2M}{3} \left(\frac{OCR}{2} \right)^{\Delta} \cdot \ln(I_R) \cdot \sigma_{vo}'$$

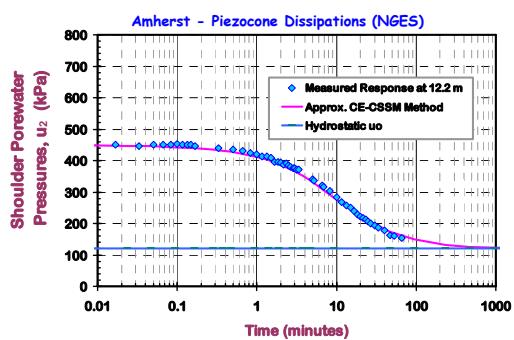
Shear Component:

$$\Delta u_{shear} = \left[1 - \left(\frac{OCR}{2} \right)^{\Delta} \right] \cdot \sigma_{vo}'$$

Approximate Piezodissipation Method

Calculated Dissipations from Cavity Expansion-Critical State Theory										
Time (sec)	Mod. Time Factor T	OCTAHEDRAL POREWATER PRESSURE			SHEAR-INDUCED POREWATER PRESSURE			u_p	u_s	time (min)
		Δu_{oct}	Δu_{vo}	Δu (kPa)	Δu_{oct}	Δu_{vo}	Δu (kPa)			
0.001	0.000001	0.9950	1.71	433	0.9756	0.314	78.4	2.05	716	200.0
0.002	0.000002	0.9932	1.73	433	0.9504	0.306	76.5	2.04	709	200.0
0.006	0.000006	0.9977	1.72	431	0.8990	0.286	71.4	2.01	703	200.0
0.011	0.000011	0.9958	1.72	430	0.8002	0.257	64.3	1.98	694	200.0
0.022	0.000022	0.9907	1.71	427	0.6670	0.214	53.8	1.92	681	200.0
0.044	0.000044	0.9840	1.70	424	0.4940	0.148	35.1	1.85	650	200.0
0.088	0.000088	0.9756	1.67	418	0.2860	0.092	23.0	1.76	641	200.0
0.112	0.000112	0.9596	1.67	418	0.1669	0.054	13.4	1.70	625	200.0
0.225	0.000225	0.9450	1.65	412	0.1669	0.054	13.4	1.62	605	200.0
0.448	0.000448	0.9162	1.60	398	0.0742	0.024	6.0	1.58	580	200.0
0.896	0.000896	0.8800	1.58	388	0.0380	0.012	3.1	1.58	559	200.0
1.124	0.001124	0.8440	1.56	378	0.0196	0.006	1.6	1.47	568	200.0
2.248	0.002248	0.8406	1.46	369	0.0196	0.006	1.6	1.47	568	200.0
5.619	0.005619	0.7559	1.32	329	0.0079	0.003	0.6	1.32	530	200.0
11.238	0.011238	0.6646	1.06	260	0.0040	0.001	0.3	1.06	480	200.0
22.475	0.022475	0.5955	0.95	238	0.0020	0.001	0.2	0.95	452	200.0
33.7	0.0337	0.4957	0.81	203	0.0013	0.000	0.1	0.81	403	200.0
56.2	0.0562	0.3570	0.62	156	0.0008	0.000	0.1	0.62	356	200.0
108.4	0.1084	0.2080	0.46	111	0.0003	0.000	0.0	0.45	293	200.0
112	0.100000	0.2149	0.37	94	0.0004	0.000	0	0.37	294	200.0
225	0.200000	0.1957	0.18	46	0.0002	0.000	0	0.18	246	200.0
448	0.400000	0.1593	0.07	39	0.0001	0.000	0	0.07	246	200.0
1124	1.000000	0.0996	0.02	4	0.0000	0.000	0	0.02	204	200.0
2248	2.000000	0.0028	0.00	1	0.0000	0.000	0	0.00	201	200.0
5619	5.000000	0.0005	0.00	0	0.0000	0.000	0	0.00	200	200.0
11238	10.000000	0.0001	0.00	0	0.0000	0.000	0	0.00	200	200.0

Monotonic Porewater Dissipation: NC Lacustrine Clay



LTRC (1999)

Approximate Monotonic & Dilatory Piezo-Dissipations (Mayne, In-Situ Bali 2001)

Define $T^* = (c_{vh} \cdot t) / (a^2 \cdot I_R^{0.75})$

$\Delta u_{meas} = \Delta u_{oct} + \Delta u_{shear}$

Dissipation:

$$\Delta u_{meas} \approx \frac{\Delta u_{oct}}{1+50T^*} + \frac{\Delta u_{shear}}{1+5000T^*}$$

Octahedral Component:

$$\Delta u_{oct} = \frac{2M}{3} \left(\frac{OCR}{2} \right)^{\lambda} \cdot \ln(I_R) \cdot \sigma_{vo}'$$

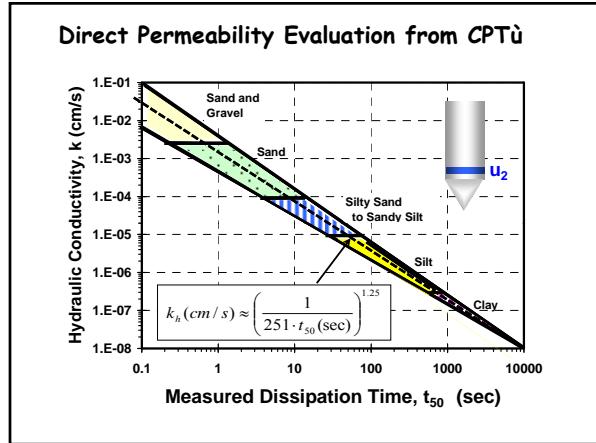
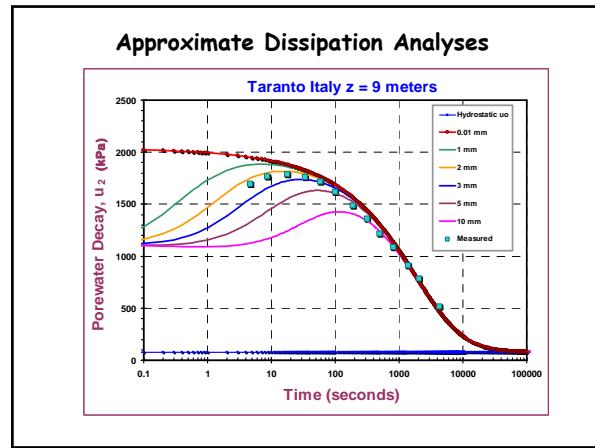
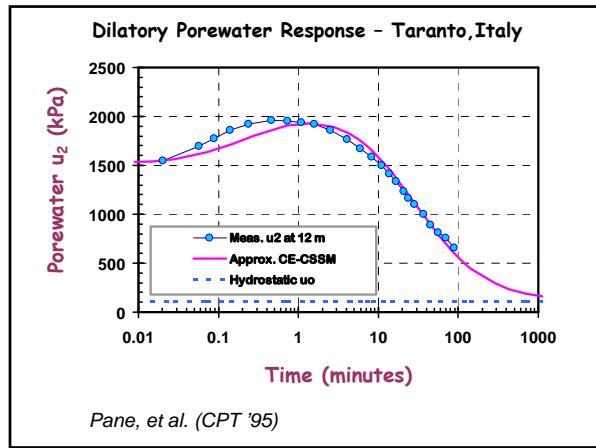
Shear Component:

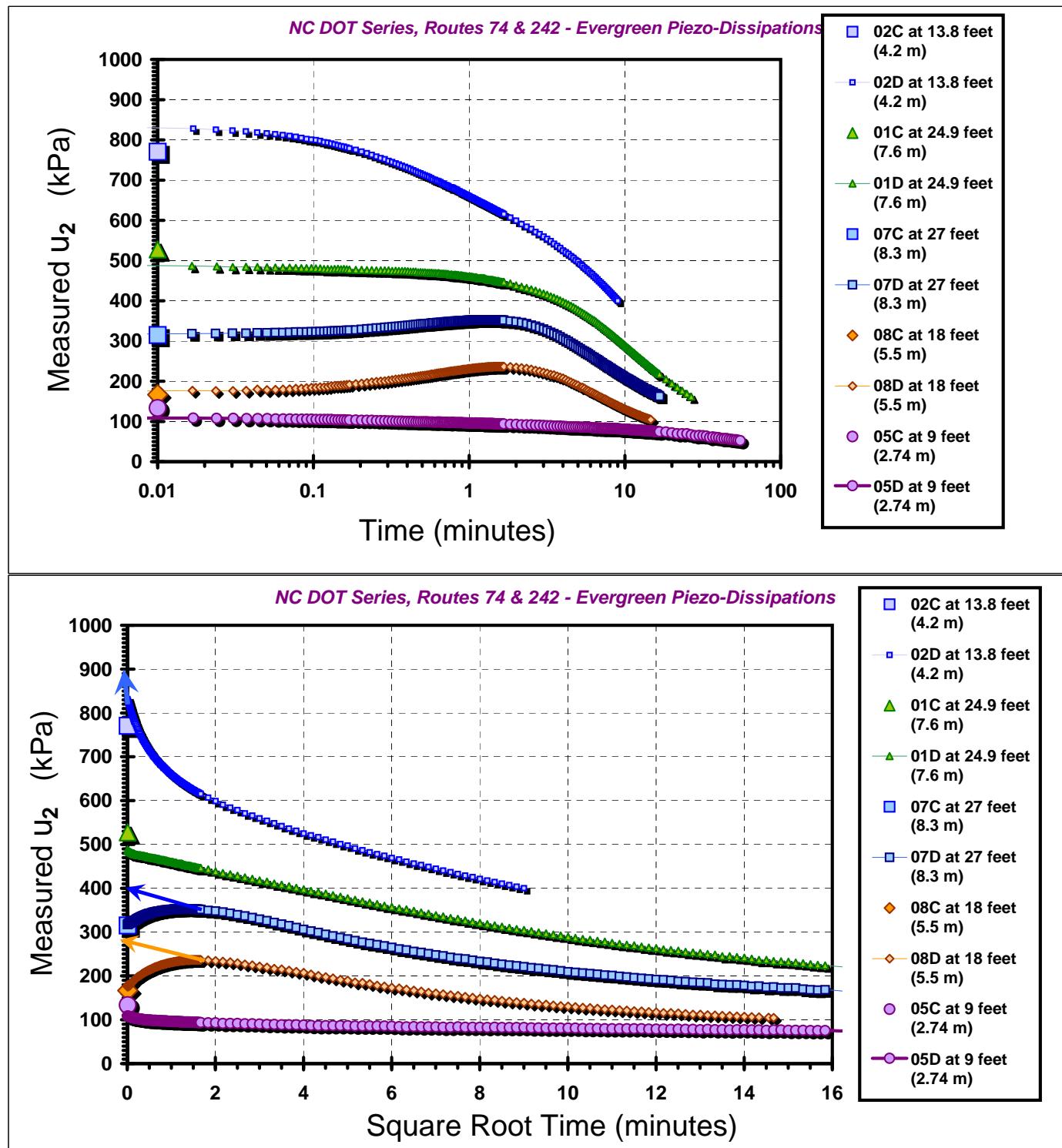
$$\Delta u_{shear} = \left[1 - \left(\frac{OCR}{2} \right)^{\lambda} \right] \cdot \sigma_{vo}'$$

Approximate Piezodissipation Method

Calculated Dissipations from Cavity Expansion-Critical State Theory

Time (sec)	Mod. Time Factor T*	thickness x (shear) = 2 mm				Total				time (min)	Norm $\Delta u/\Delta u_i$
		$\Delta u/\Delta u_i$	$\Delta u/\sigma_{vo}'$	Δu (kPa)	$d\Delta u/dt$	$\Delta u/\sigma_{vo}'$	Δu (kPa)	$\Delta u_2/\sigma_{vo}'$	Δu_2 (kPa)		
0.00			1.74	436	0.321	80.3	20.6	716	200.0	0.00	1.00
0.001	0.000001	0.9950	1.73	433	0.9756	0.314	78.4	2.05	712	200.0	0.00
0.002	0.000002	0.9932	1.73	433	0.9524	0.306	76.5	2.04	709	200.0	0.00
0.006	0.000005	0.9897	1.72	431	0.8890	0.286	71.4	2.01	703	200.0	0.00
0.011	0.000010	0.9859	1.72	430	0.8002	0.257	64.3	1.98	694	200.0	0.00
0.022	0.000020	0.9807	1.71	427	0.6670	0.214	53.6	1.92	681	200.0	0.00
0.056	0.000050	0.9707	1.69	423	0.4448	0.143	35.7	1.83	659	200.0	0.00
0.112	0.000100	0.9599	1.67	418	0.2860	0.092	23.0	1.76	641	200.0	0.00
0.225	0.000200	0.9450	1.65	412	0.1669	0.054	13.4	1.70	625	200.0	0.00
0.562	0.000500	0.9162	1.60	399	0.0742	0.024	6.0	1.62	605	200.0	0.01
1.124	0.001000	0.8945	1.54	385	0.0385	0.012	3.1	1.55	588	200.0	0.02
2.248	0.002000	0.8406	1.46	366	0.0196	0.006	1.6	1.47	568	200.0	0.04
5.619	0.005000	0.7559	1.32	329	0.0079	0.003	0.6	1.32	530	200.0	0.09
11.2	0.010000	0.6648	1.16	290	0.0040	0.001	0.3	1.16	490	200.0	0.19
22.5	0.020000	0.5485	0.95	238	0.0020	0.001	0.2	0.95	438	200.0	0.37
33.7	0.030000	0.4657	0.81	203	0.0013	0.000	0.1	0.81	403	200.0	0.56
56.2	0.050000	0.3570	0.62	156	0.0008	0.000	0.1	0.62	356	200.0	0.94
89.9	0.080000	0.2583	0.45	113	0.0005	0.000	0.0	0.45	313	200.0	1.50
112	0.100000	0.2149	0.37	94	0.0004	0.000	0.0	0.37	294	200.0	1.87
225	0.200000	0.1057	0.18	46	0.0002	0.000	0.0	0.18	246	200.0	3.75
562	0.500000	0.0302	0.05	13	0.0001	0.000	0.0	0.05	213	200.0	9.37
1124	1.000000	0.0096	0.02	4	0.0000	0.000	0.0	0.02	204	200.0	18.73
2248	2.000000	0.0028	0.00	1	0.0000	0.000	0.0	0.00	201	200.0	37.46
5619	5.000000	0.0005	0.00	0	0.0000	0.000	0.0	0.00	200	200.0	93.65
11238	10.000000	0.0001	0.00	0	0.0000	0.000	0.0	0.00	187.31	0.00	

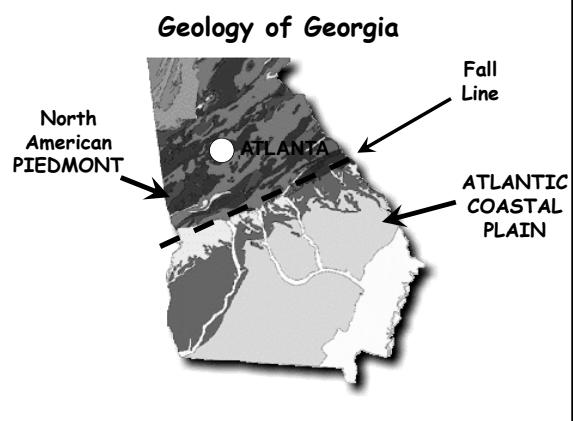




Geotechnical Site Characterization of Piedmont Residuum in North America

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Geosystems Program
Civil & Environmental Engineering
Georgia Institute of Technology
Atlanta, Georgia

Atlantic Piedmont Geologic Province in North America



Geologic Time Scale

PIEDMONT ROCKS
Chew (1993)

Era	Period	Epoch	Time Boundaries (Years Ago)
	Quaternary	Holocene - Recent	10,000
	Pleistocene		2 million
	Pliocene		5 million
	Miocene		26 million
Cenozoic	Oligocene		38 million
	Eocene		54 million
	Paleocene		65 million
	Cretaceous		130 million
Mesozoic	Jurassic		185 million
	Triassic		230 million
	Permian		265 million
	Carboniferous	Pennsylvanian	310 million
		Mississippian	355 million
Paleozoic	Devonian		413 million
	Silurian		425 million
	Ordovician		475 million
	Cambrian		570 million
	Precambrian		3.9 billion
	Earth Beginning		4.7 billion

Piedmont Rock Formations

Metamorphic & Igneous Rocks:

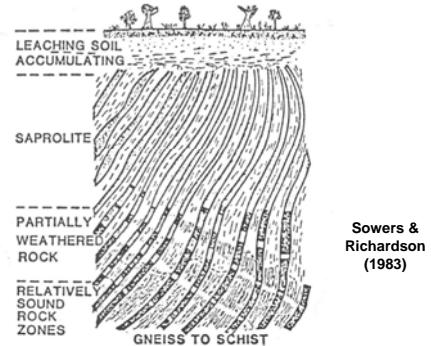
Gneiss, Schist, and Granite

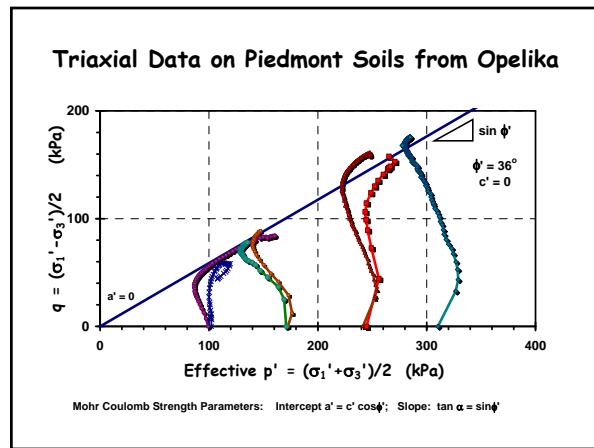
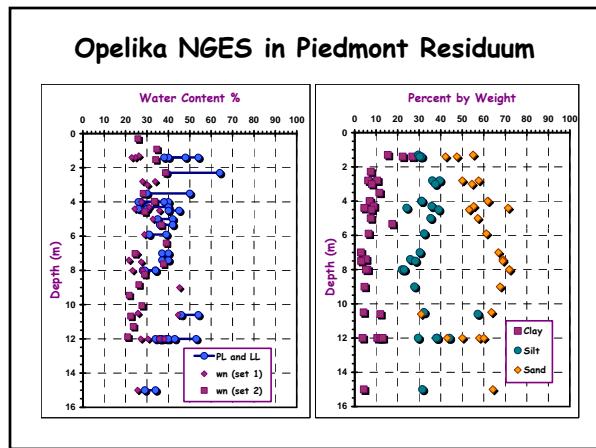
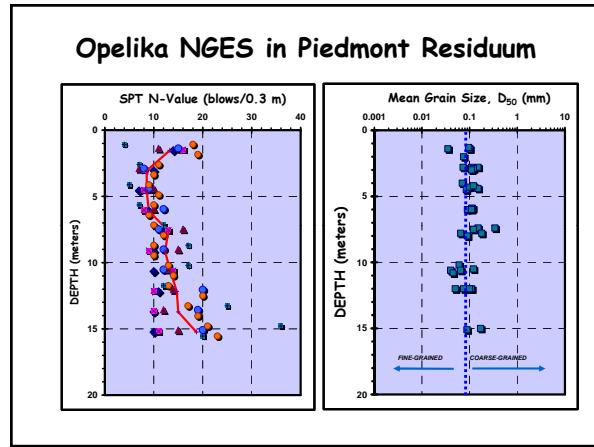
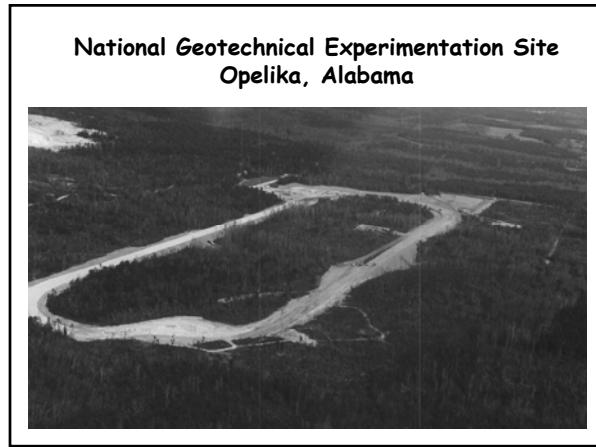
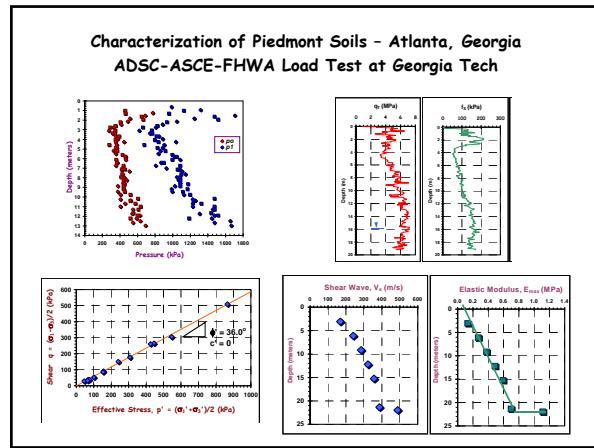
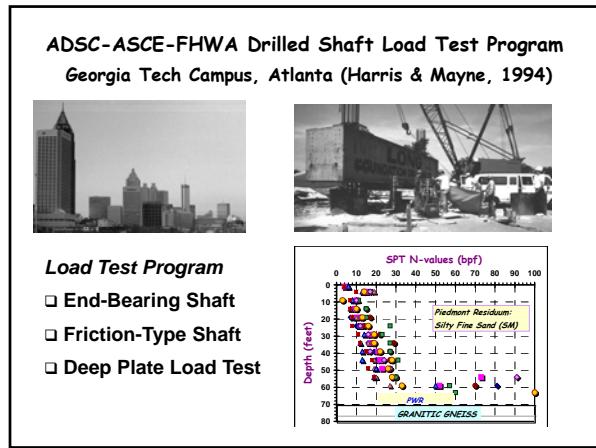
(localized: Greenstone, Soapstone, Quartzite, Diabase)

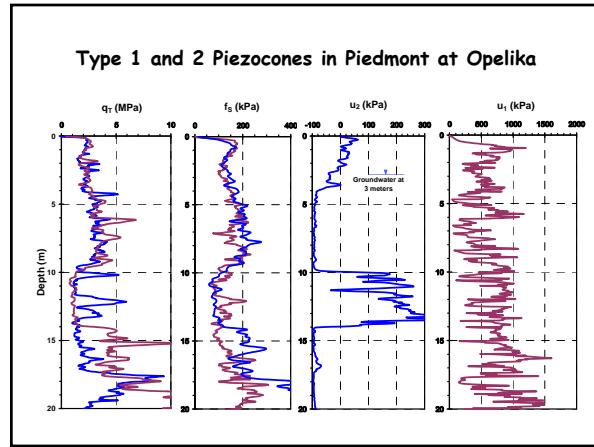
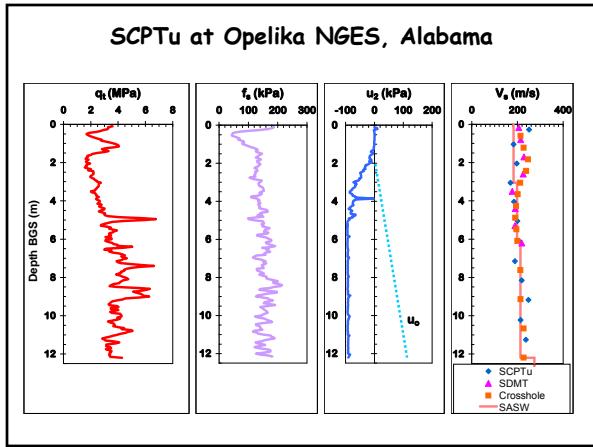
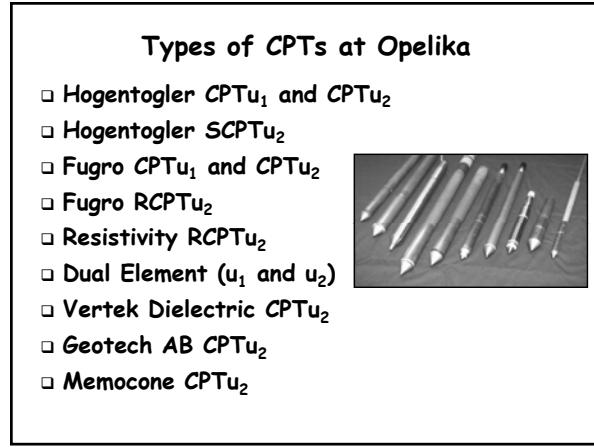
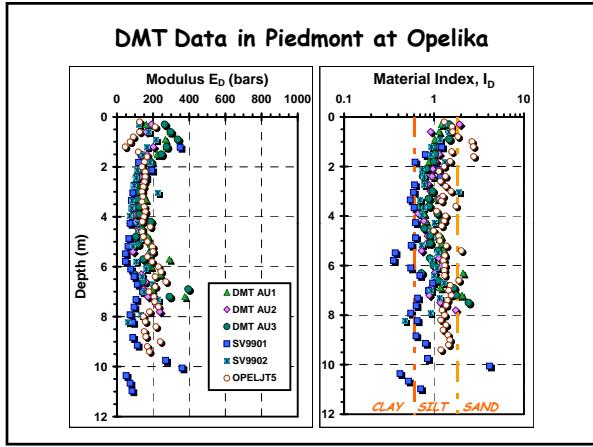
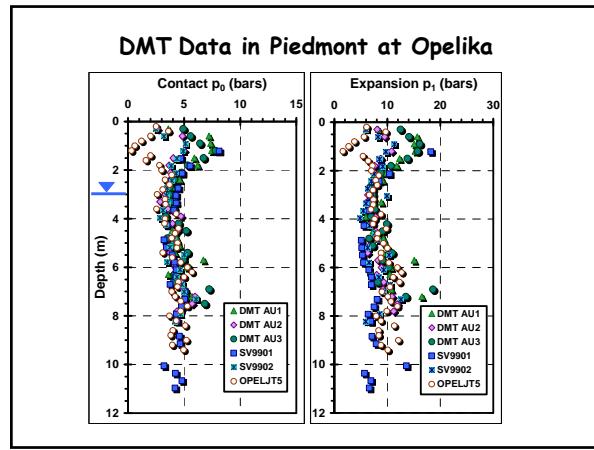
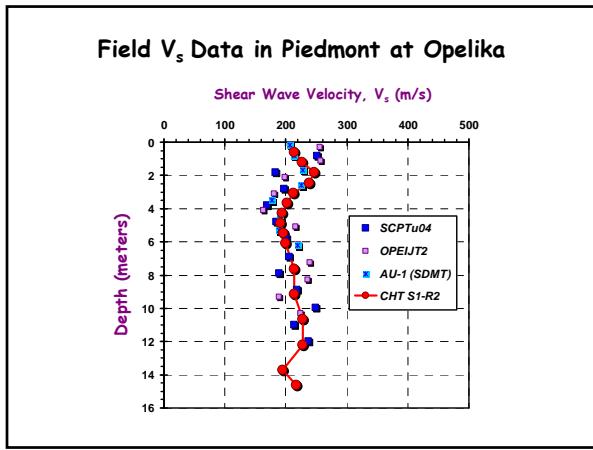


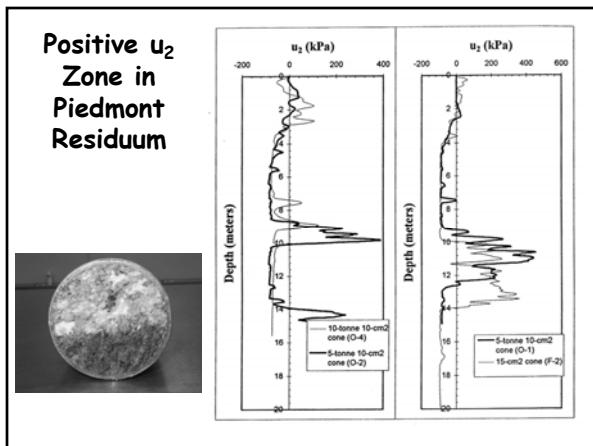
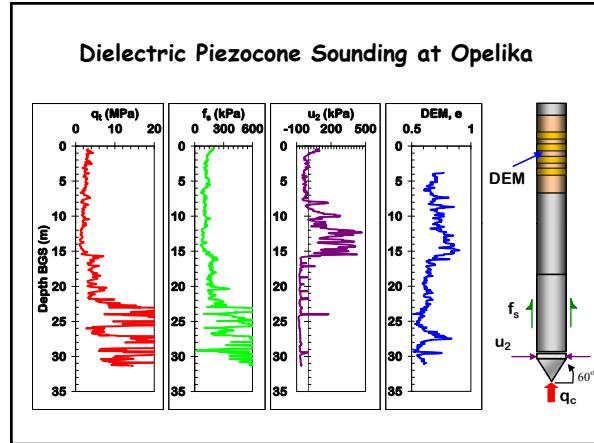
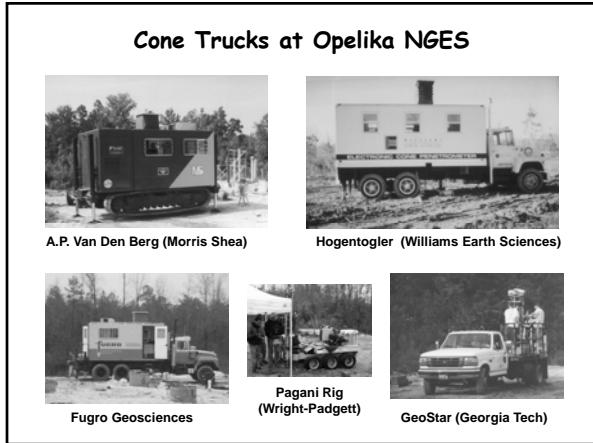
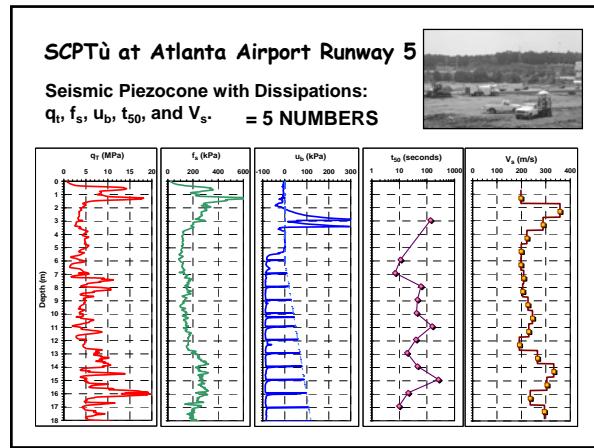
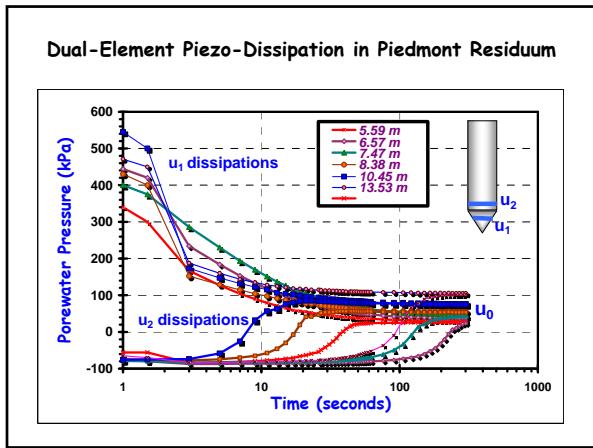
Stone
Mountain
Atlanta
Georgia

Weathered Profile in Piedmont





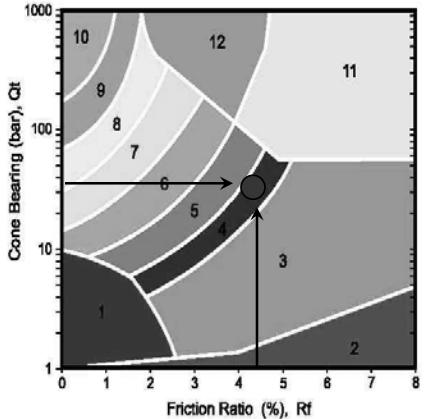




- Other In-Situ Tests in Piedmont at Opelika**
- Full-displacement pressuremeter
 - DMT dissipations
 - Prebored (Menard) pressuremeter
 - (Iowa) Borehole shear test
 - Spectral analysis of surface waves
 - Variable CPT \bar{u} penetration rates
 - Push-in permeameter probe (GeoProbe)
 - Seismic flat dilatometer tests
 - Surface resistivity surveys
 - Foundation load tests (axial & lateral drilled shafts, deWaal piles, pipe piles, lateral groups, statnamic)

Piedmont CPTs at Opelika NGES: 1986 SBT Charts and SPT-CPT interrelationship

(After Robertson, et al., 1986)



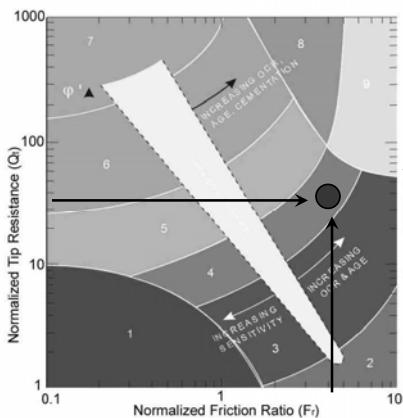
ZONE	SBT
1	Sensitive, fine grained
2	Organic materials
3	Clay
4	Silty clay to clay
5	Clayey silt to silty clay
6	Sandy silt to clayey silt
7	Silty sand to sandy silt
8	Sand to silty sand
9	Sand
10	Gravelly sand to sand
11	Very stiff fine grained*
12	Sand to clayey sand*

*over consolidated or cemented

MEANS: Cone Resistance $q_t = 35$ bars; Friction Ratio FR = 4.34 %

Piedmont CPTs at Opelika NGES: 1990/1991 SBT Charts and SPT-CPT interrelationship

(After Robertson, 1990)

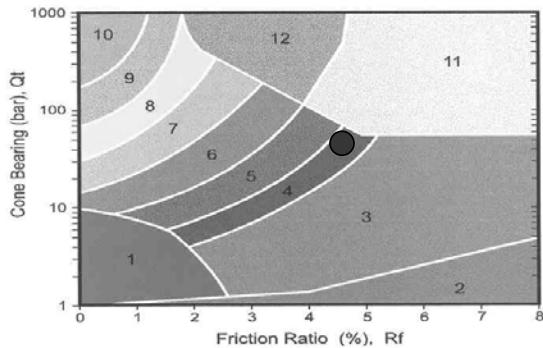


Zone	Normalized Soil Behavior Type
1	sensitive fine grained
2	organic material
3	clay to silty clay
4	clayey silt to silty clay
5	silty sand to sandy silt
6	clean sands to silty sands
7	gravelly sand to sand
8	very stiff sand to clayey sand
9	very stiff fine grained

MEAN VALUES: Normalized Tip Q = 34.5; Normalized Friction F = 4.5%

Piedmont at Opelika NGES:

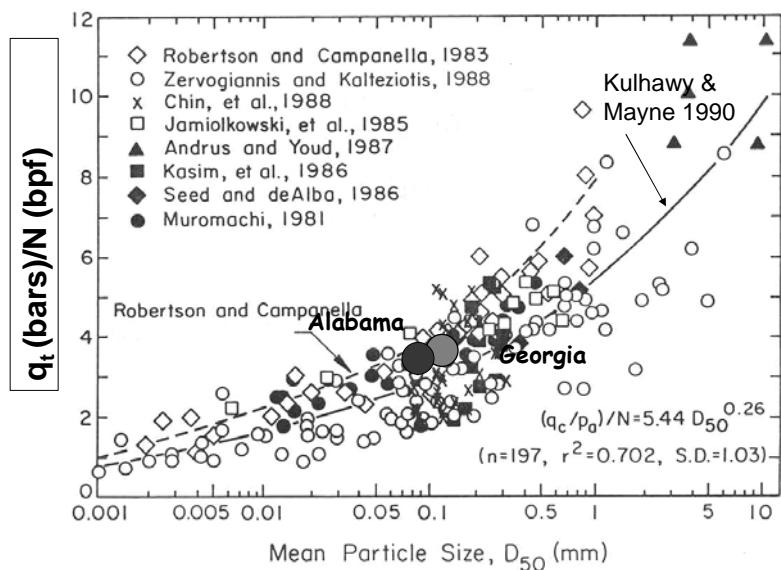
SPT-CPT
relations from
SBT Charts

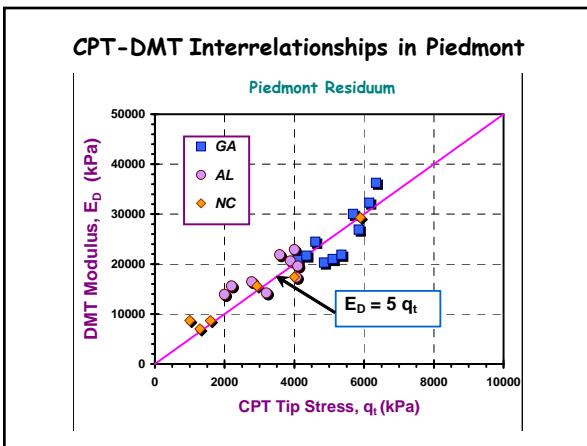
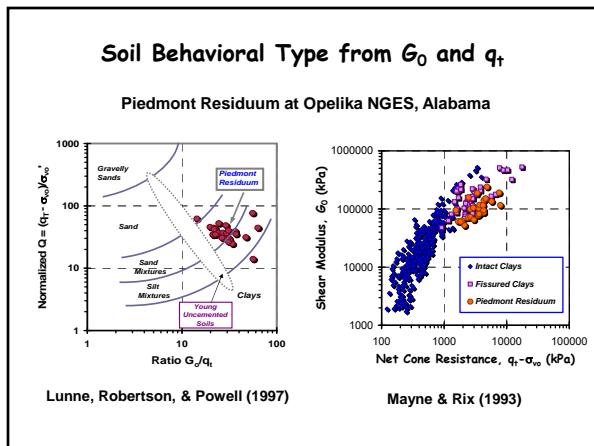
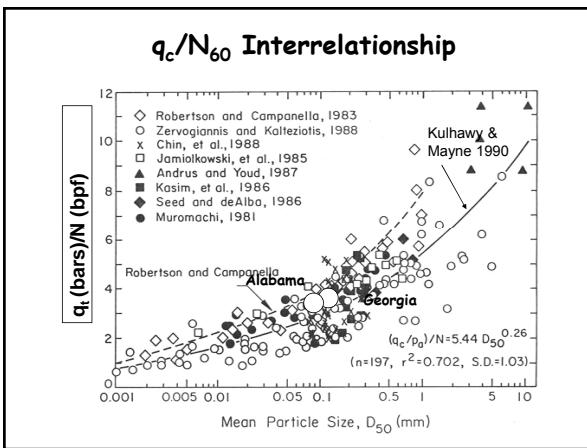
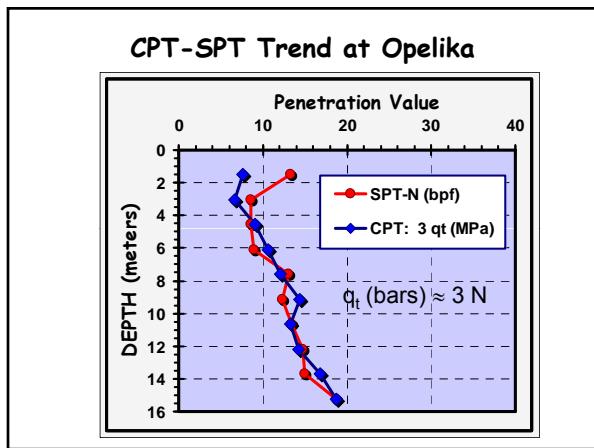
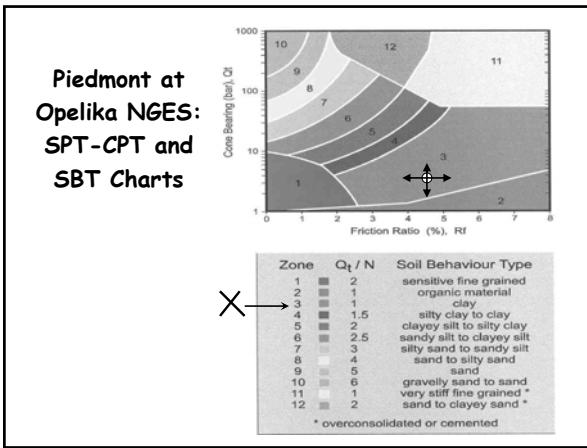
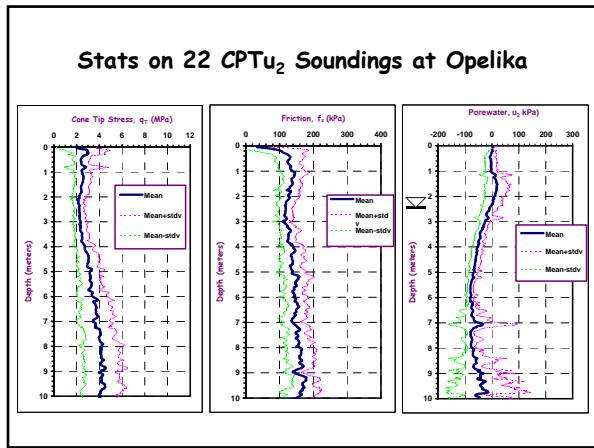


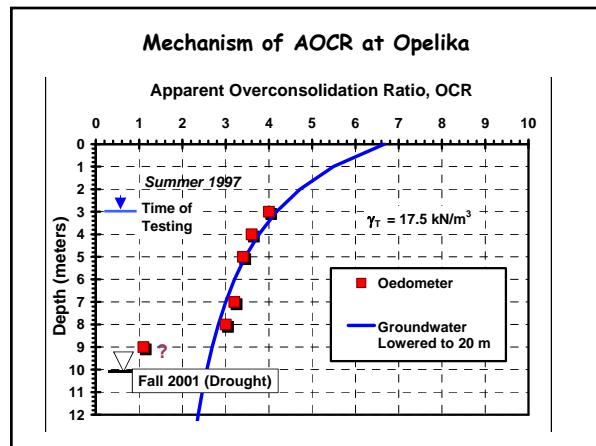
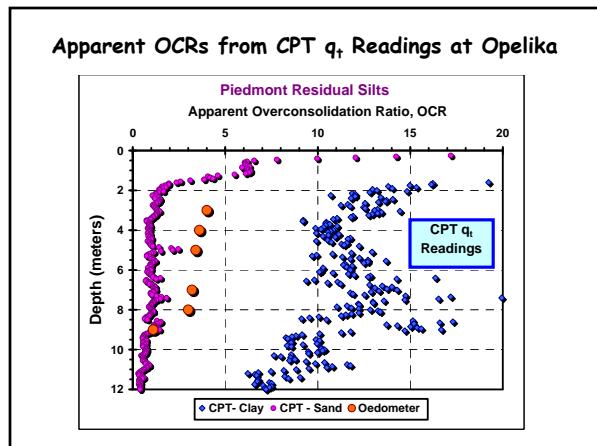
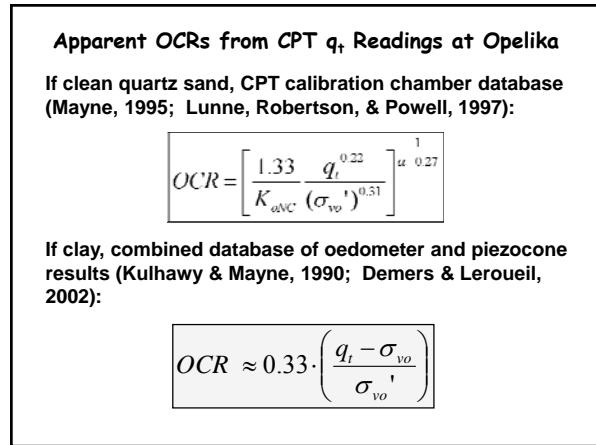
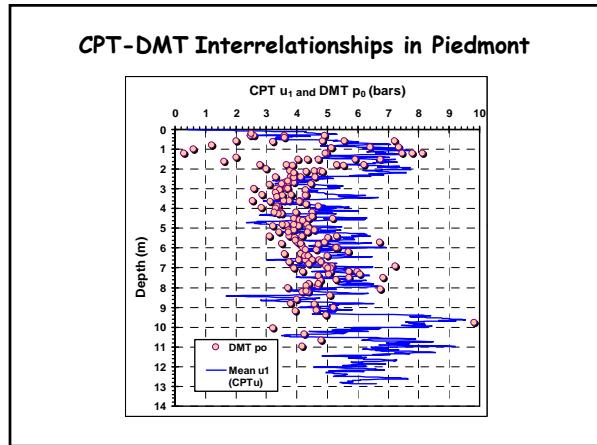
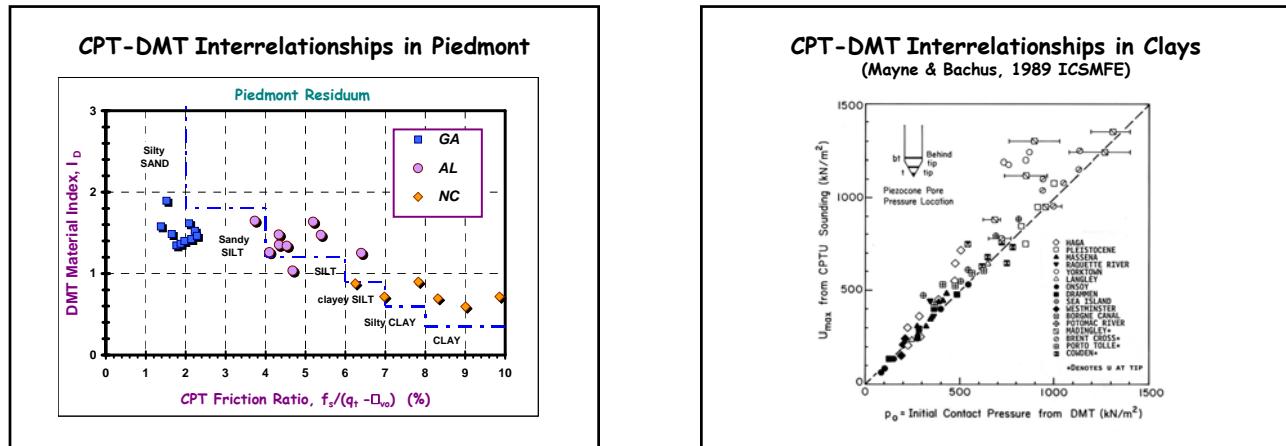
Zone	Q_t / N	Soil Behaviour Type
1	2	sensitive fine grained
2	1	organic material
3	1	clay
4	1.5	silty clay to clay
5	2	clayey silt to silty clay
6	2.5	sandy silt to clayey silt
7	3	silty sand to sandy silt
8	4	sand to silty sand
9	5	sand
10	6	gravelly sand to sand
11	1	very stiff fine grained *
12	2	sand to clayey sand *

* overconsolidated or cemented

q_t/N_{60} Interrelationships (CPT-SPT)



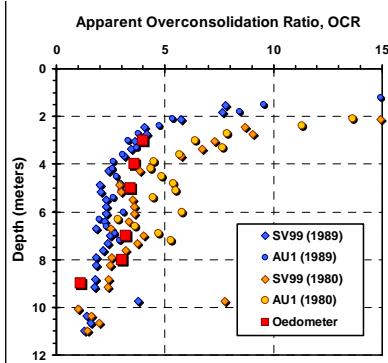




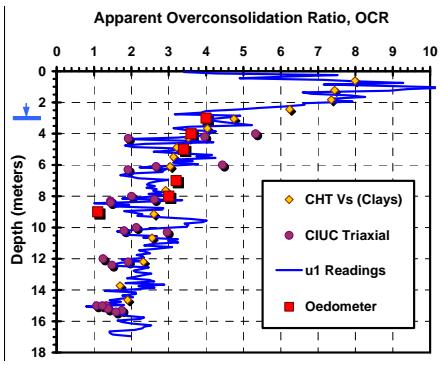
Apparent OCR from In-Situ Tests

- ❑ Flat Dilatometer
 - 1980: $OCR = (0.5 K_D)^{1.56}$
 - 1989: $\sigma_p' = 0.509 (p_0 - u_0)$
- ❑ Type 1 Piezocone: $\sigma_p' = 0.47 (u_1 - u_0)$
- ❑ Shear Wave: $\sigma_p' = 0.107 V_s^{1.47}$

Apparent OCR From DMTs at Opelika



Apparent OCR Profiles at Opelika



Piedmont Residuum of North America

- ❑ "Georgia Red Clay" classifies as ML - SM
- ❑ Contradictions/Agreement in interpreted soil properties (Test-Specific: "loose sand" and/or "stiff fissured clay")
- ❑ Rate-dependent behavior, thus undrained and/or drained conditions apply
- ❑ Piedmont requires site-specific correlations for interpretation

View from Stone Mountain, Georgia



**Georgia State University
Dorm B Settlements on Piedmont
Soils, Atlanta, Georgia**

**Paul W. Mayne
Georgia Institute of Technology**

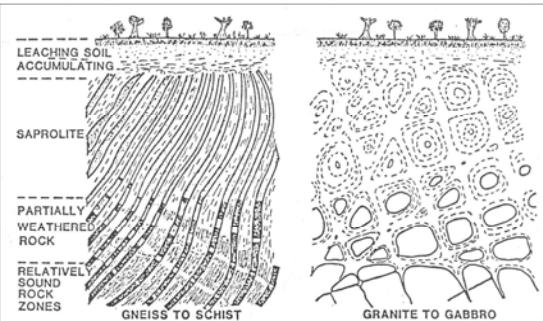
Georgia Tech

Geology of Georgia



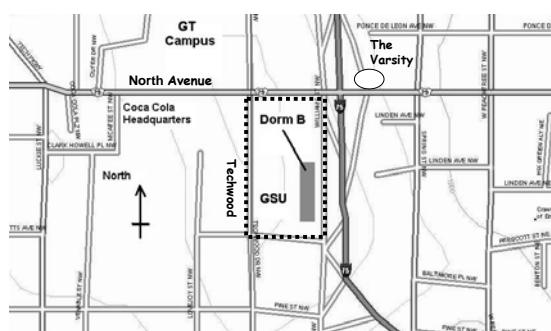
Georgia Tech

Piedmont Geology



Georgia Tech

Site of Georgia State Dormitories, North Ave.



Georgia Tech

Georgia State Dormitory B



Georgia Tech

Georgia State Dormitory B



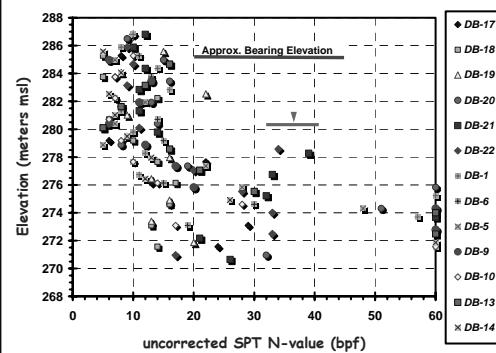
Georgia Tech

GSU Dorm B, Atlanta

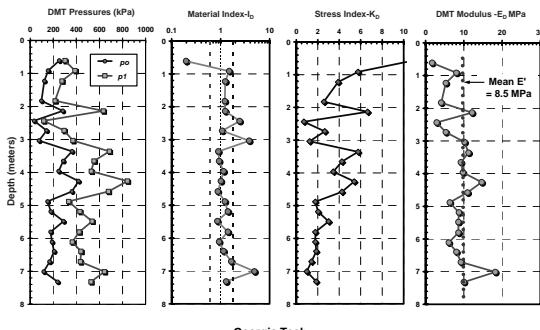
- ❑ Constructed just before 1996 Olympics
- ❑ 4 Dorm Buildings situated on Piedmont Residuum over PWR/Bedrock
- ❑ Deep Foundations, Footings, & Mat
- ❑ Dorm B Mat: 340 feet long, 60 feet wide, and 3.5 feet thick. Applied $q = 1.5 \text{ tsf}$
- ❑ Design firm predicted 1.5" max settlement.
- ❑ Measured 9.75" by end of construction.

Georgia Tech

Summary of SPT N-values at GSU Complex



Results of DMT at GSU Complex



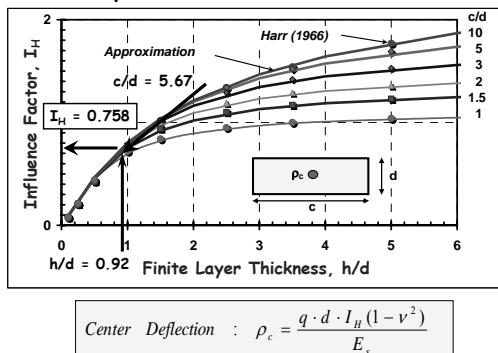
Georgia Tech

Dorm B Mat Settlements

- ❑ Mat breadth, $c = 340 \text{ feet}$
- ❑ Mat width, $d = 60 \text{ feet}$
- ❑ Rectangle Ratio $c/d = 5.67$
- ❑ Approx. depth to rock/PWR = 55 feet
- ❑ Depth ratio, $h/d = 55/60 = 0.92$
- ❑ Determine values of displacement influence factor (I) from elastic theory

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Displacement Influence Factors

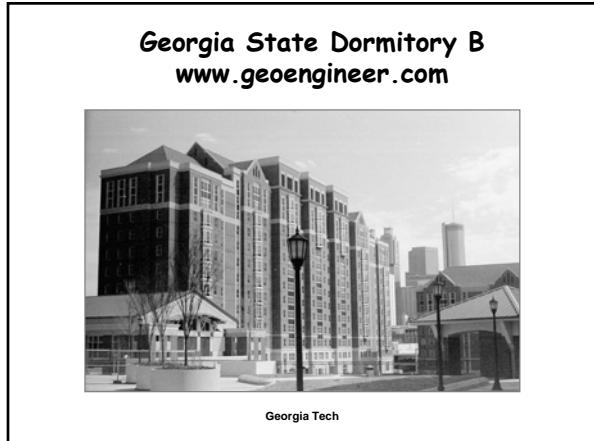
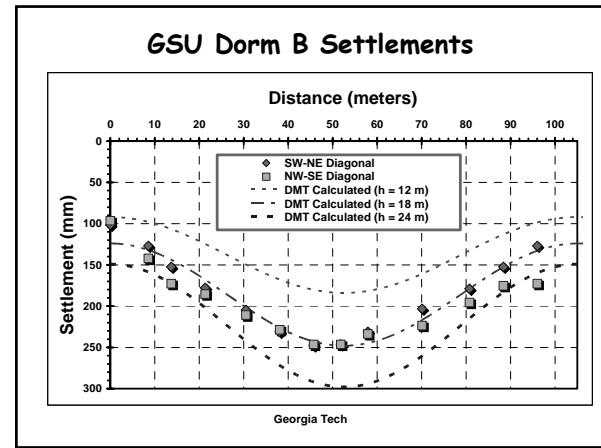
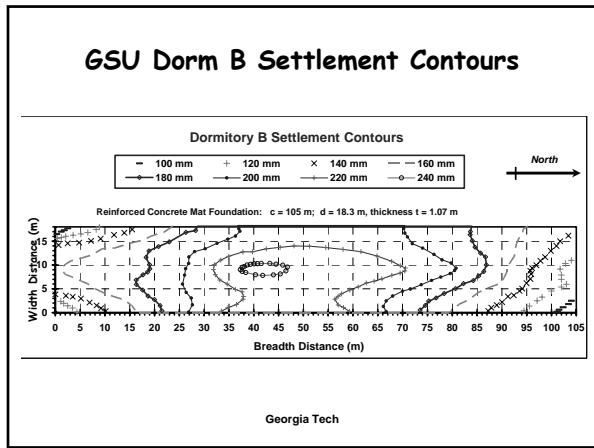


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Dorm B Settlement Calculation

- ❑ $I_H = 0.758$ from elastic solution
 - ❑ $E_s = 8.5 \text{ MPa} = 88.5 \text{ tsf}$ from DMT
 - ❑ Assume drained $\nu' = 0.2$
 - ❑ Centerpoint Settlement:
- $$\rho = \frac{q \cdot d \cdot I_H \cdot (1 - \nu'^2)}{E_s} = \frac{1.5 \text{ tsf}(60 \text{ ft})(0.758)(0.96)}{88.5 \text{ tsf}}$$
- $\rho_{center} = 0.74 \text{ feet} = 8.88 \text{ inches} = 225 \text{ mm}$
- Flexible Fdn: $\rho_{edge} = \frac{1}{2}\rho_{center} = 4.44 \text{ inches} = 113 \text{ mm}$

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Evaluation of Soil Stiffness from Cone Penetration Tests

Paul W. Mayne

Geosystems Group
Georgia Institute of Technology

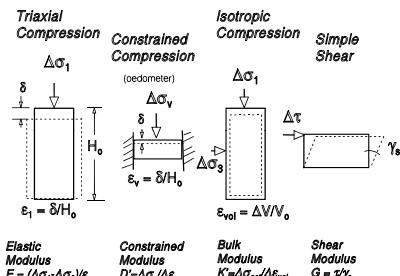
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Representation of Soil Stiffness

- Compressibility Parameters (C_c , C_r , C_s) or alternative $D' = 1/m_v = \Delta\sigma_v/\Delta\sigma_e$
- Subgrade Modulus: $k_s = q/\delta$
- Spring Coefficients: $k_v = Q/\delta$
- Equivalent Moduli from Elastic Theory
 - Undrained: E_u , G_u
 - Drained: E' , G' , K' , D'

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Equivalent Modulus for Monotonic Loading Response



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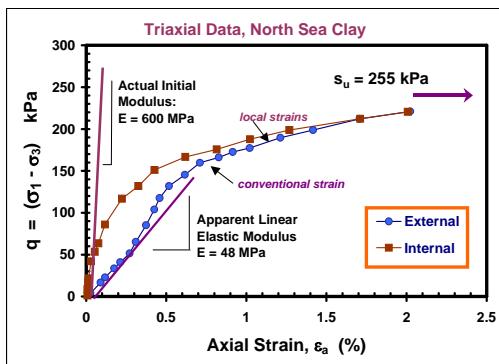
Evaluation of Soil Stiffness

Direct measurements:

- Plate Load Tests (PLT)
- Screw plate load test (SPLT) = field compressiometer (Stroud, NTNU, 1998)
- Pressuremeter Test (PMT)
- Flat Plate Dilatometer (DMT)
- Backcalculate from Full-Scale Load Test or instrumented foundation performance
- Lab tests (TX, PS, SS)
- Small-strain measurements (i.e., V_s)

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Local Strain Measurements in Lab Tests (Jardine, et al. 1984; Jamiolkowski, 1999)

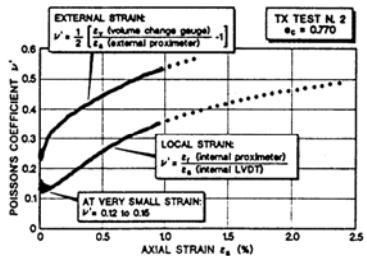


Internal Local Strain Measurements

- Burland (1989, Canadian Geot. Journal): "Small is beautiful" → Majority of soil elements engaged in deformations are at small strains < 0.1%
- Tatsuoka, et al. (1997, 14th ICSMGE, Hamburg) - SOA Report
- International Symposia: Deformation Characteristics of Geomaterials (Japan, UK, Italy, France, and USA)

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Local Strain Measurements of Poisson's Ratio, ν
Jamiolkowski, et al. (1994)



Drained:
 $\nu' = 0.2$
Undrained
 $\nu_u = 0.5$

Poisson's coefficient of Toyoura sand.

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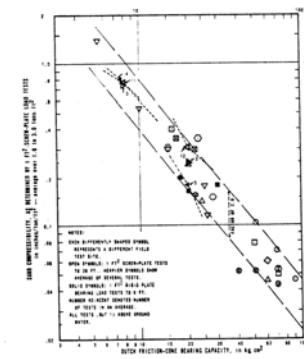
Sand Stiffness From CPTs



Schmertmann, (ASCE JSMFD, 1970)

'Screw-Plate Tests of some fine sands in North Florida'

$$E' = 2 q_c$$



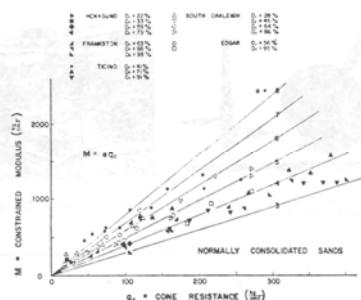
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NC Sand Stiffness from CPT Chamber Tests

NC quartz sands in flexible-walled Calibration Chambers Tests:

$$D' = \alpha q_c$$

$$3 < \alpha < 8$$



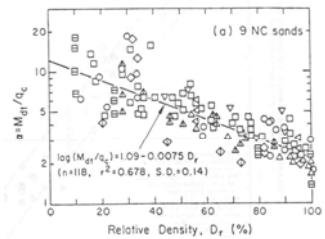
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NC Sand Stiffness from CPT Chamber Tests

NC quartz sands in flexible-walled Calibration Chambers Tests:

$$D' = \alpha q_c$$

$$\alpha = \text{fctn} (D_R)$$



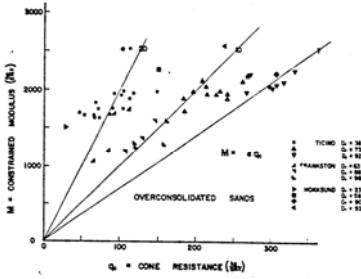
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OC Sand Stiffness from CPT Chamber Tests

OC quartz sands in flexible-walled Calibration Chambers Tests:

$$D' = \alpha q_c$$

$$7 < \alpha < 20+$$



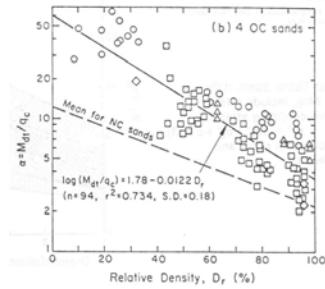
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OC Sand Stiffness from CPT Chamber

OC quartz sands in flexible-walled Calibration Chambers Tests:

$$D' = \alpha q_c$$

$$\alpha = \text{fctn} (D_R)$$



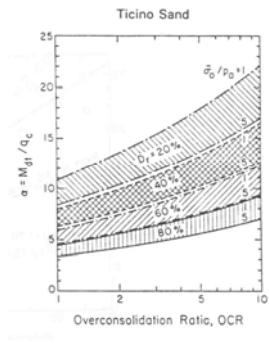
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NC and OC Sand Stiffness from CPT

NC and OC Ticino sand in Calibration Chambers Tests:

$$D' = \alpha q_c$$

$$\alpha = fctn(D_R, OCR, \sigma_{oct}')$$



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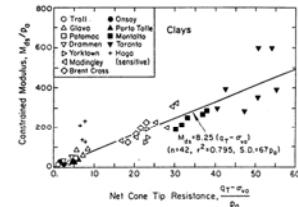
Drained Clay Modulus from CPT

- Difficulty to obtain drained parameter, D' = oedometric modulus from undrained CPT

- First-order Estimate only: $D' = 8 (q_t - \sigma_{vo})$

- Singapore (2007): $D' \approx 5 (q_t - \sigma_{vo})$

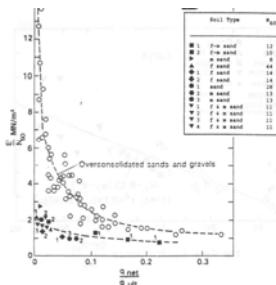
- Similar to Sanglerat form: $D' = \alpha q_c$



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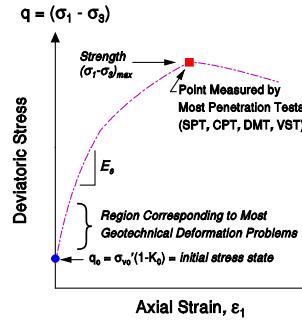
Soil Modulus from Penetration Results

- Modulus E backcalculated from foundation performance (Stroud, 1988)
- E decreases as mobilized stress increases
- $FS = q_{ult}/q_{applied}$



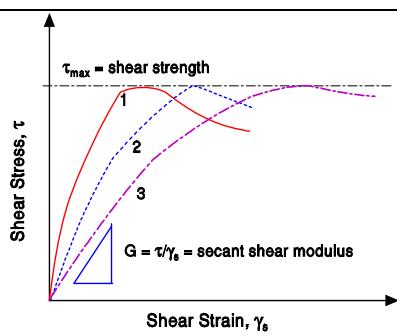
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Equivalent Modulus for Monotonic Static Response



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Equivalent Modulus for Foundation Response



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Small-Strain Stiffness

- Initial Stiffness from Shear Wave Velocity (strain levels are 10^{-6} or smaller)

- Small-Strain Shear Modulus: $G_{max} = \rho_T V_s^2$

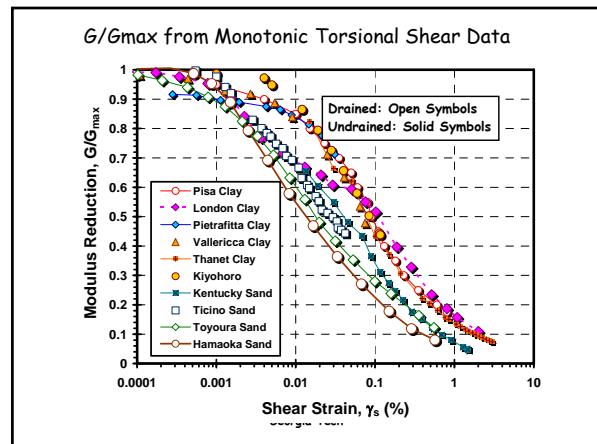
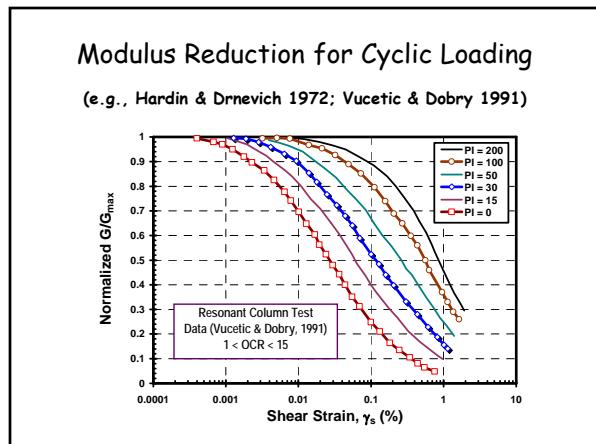
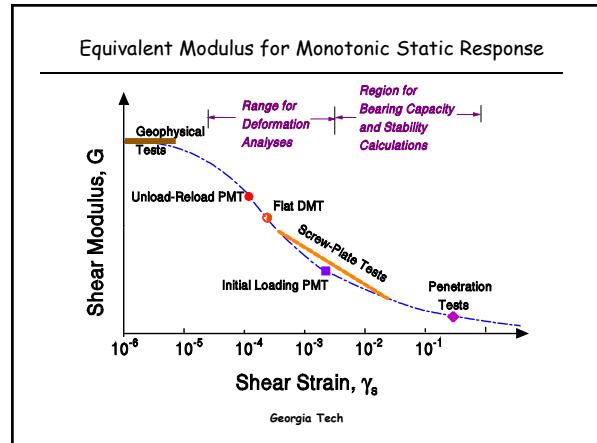
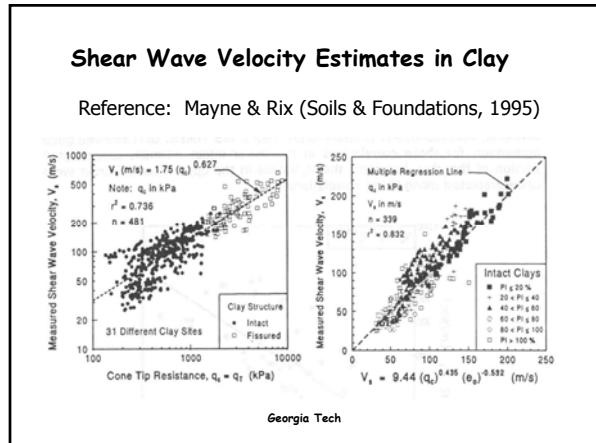
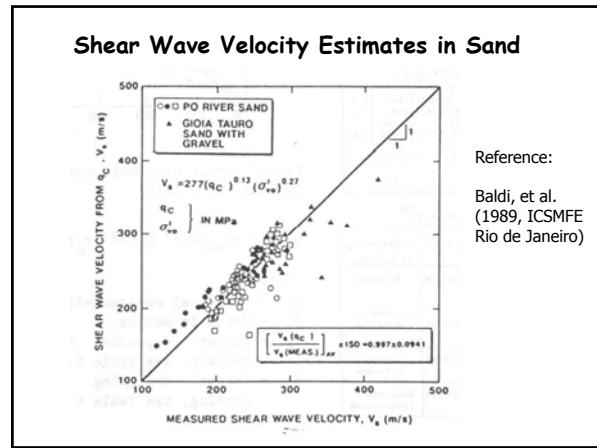
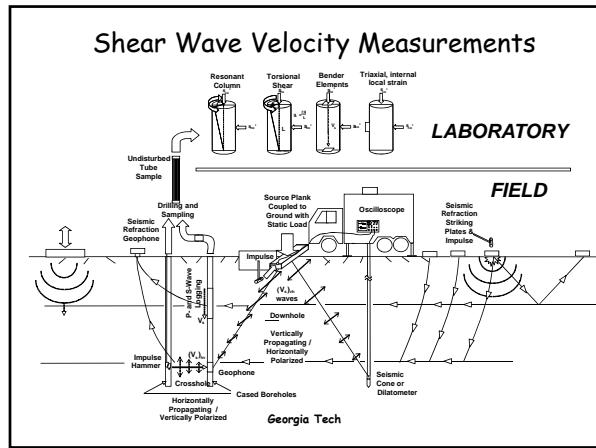
- $\rho_T = \gamma/g$ = total mass density

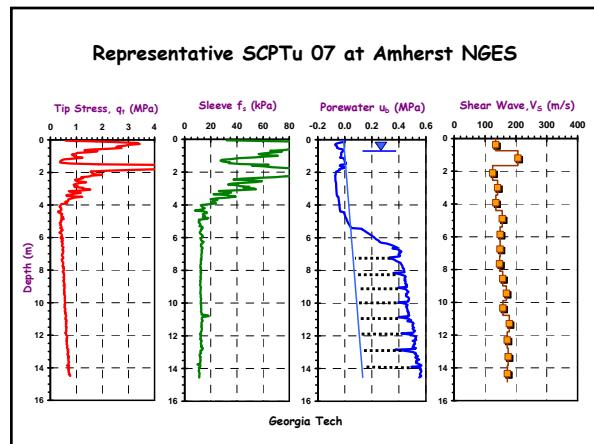
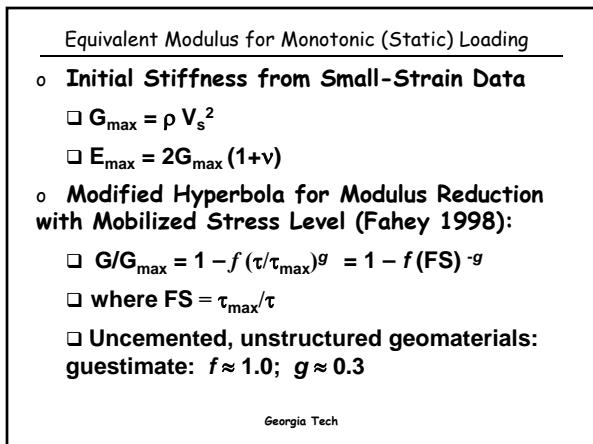
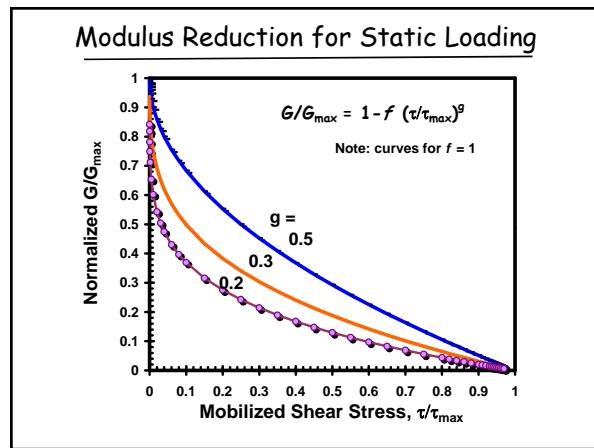
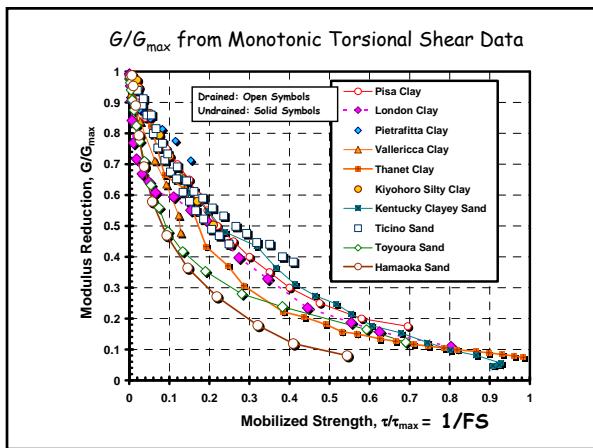
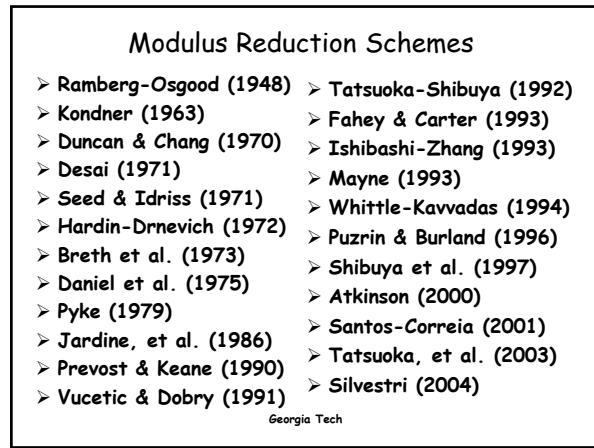
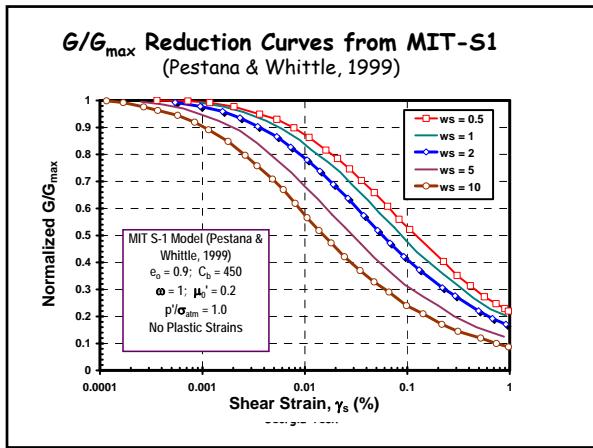
- $E_{max} = 2 G_{max} (1+v)$

- Poisson's ratio: $v = 0.15$ to 0.20 at small strains

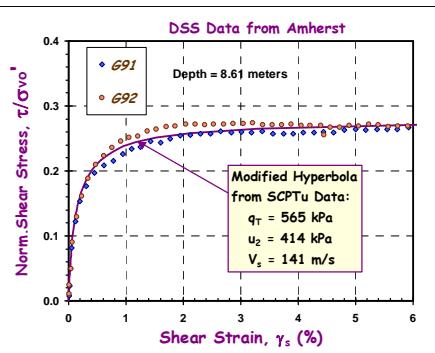
- Modulus too stiff for direct use in deformation analysis. Must be reduced.

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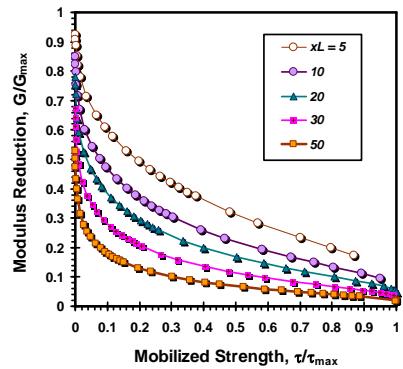




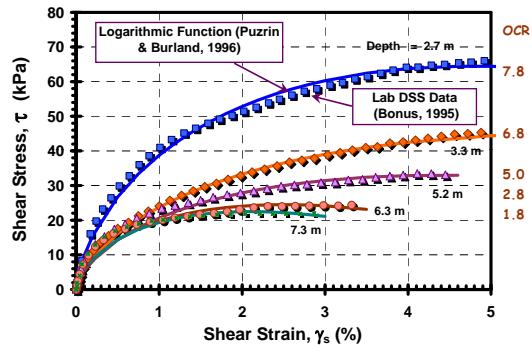
Stress-Strain-Strength From SCPTu



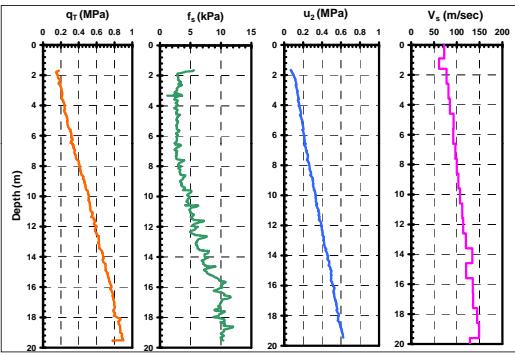
Logarithmic Function (Puzrin & Burland, 1996)



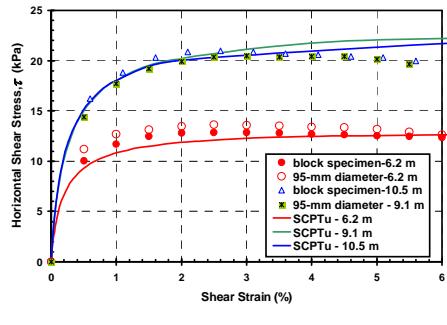
Stress-Strain-Strength Curves - Amherst NGES



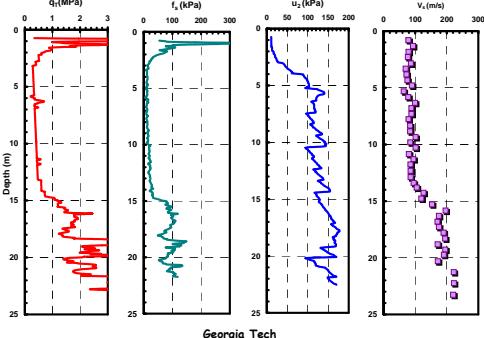
Representative SCPTu from Onsøy, Norway (Gillespie, et al. 1985, NGI Report)

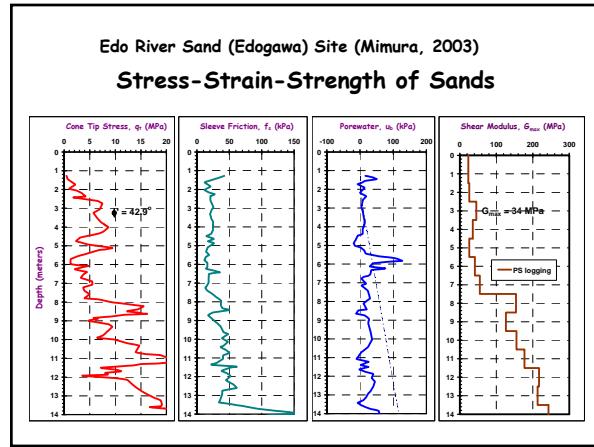
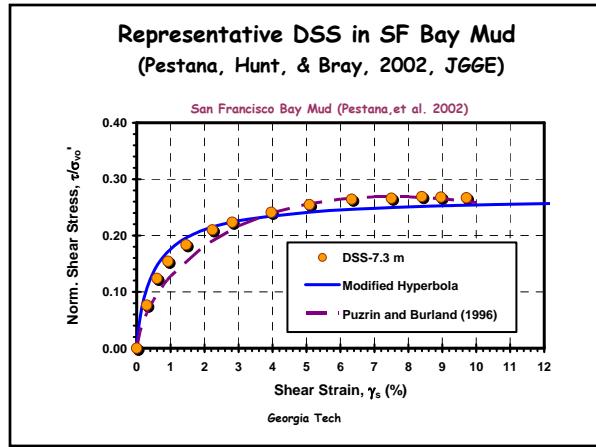
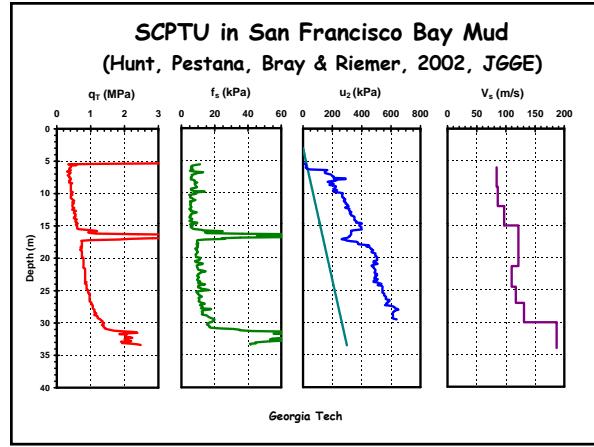
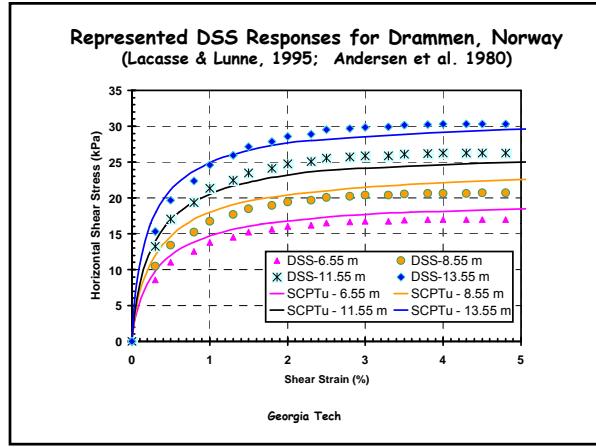
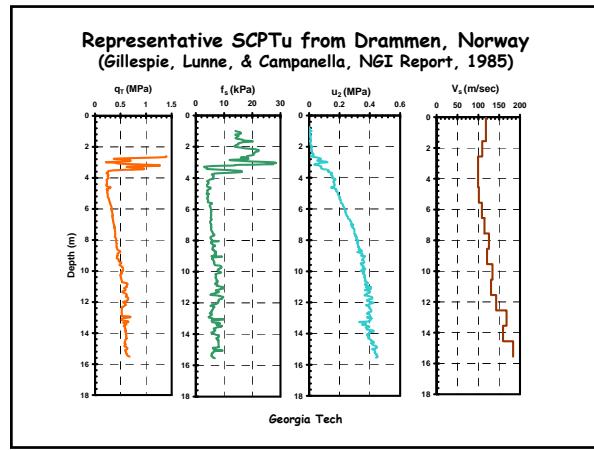
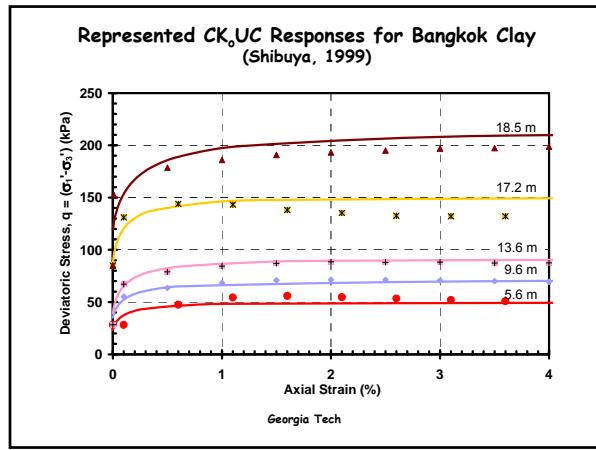


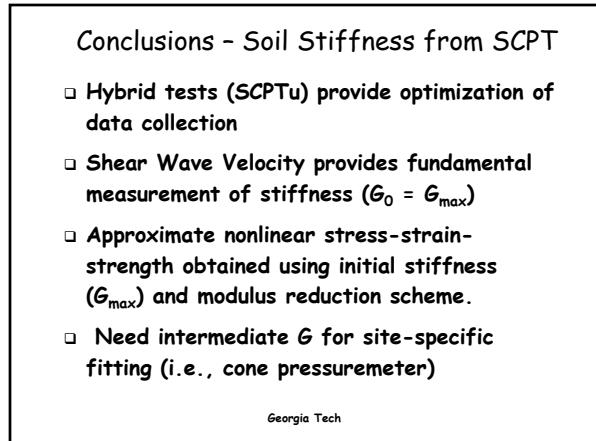
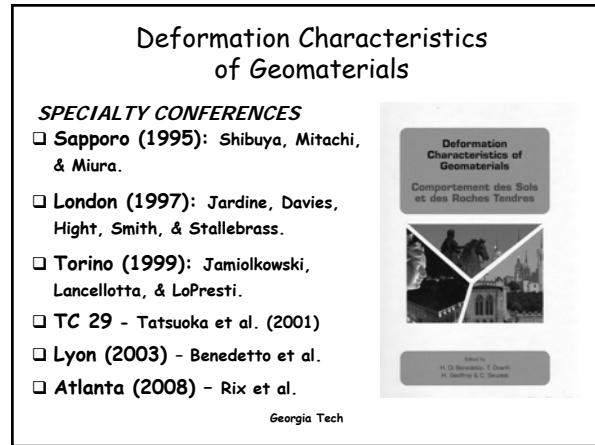
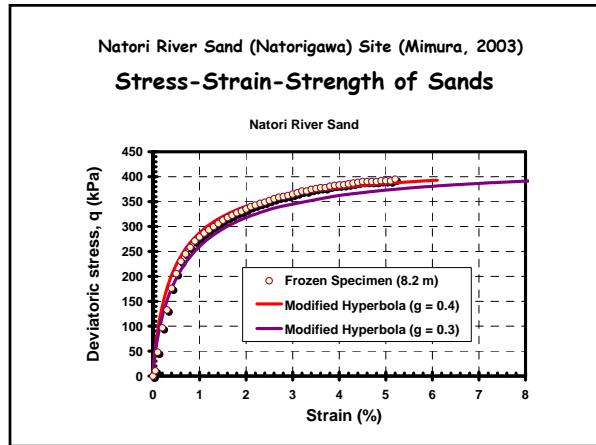
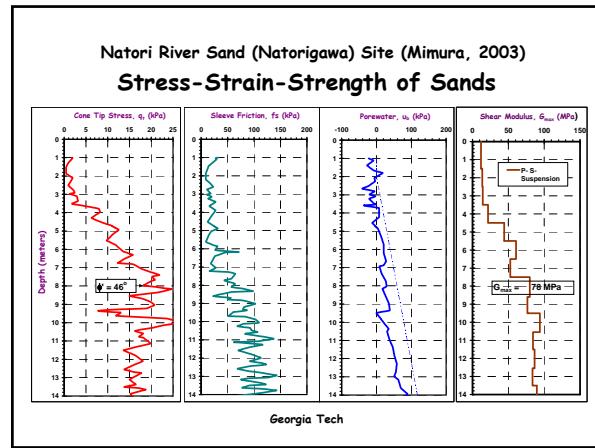
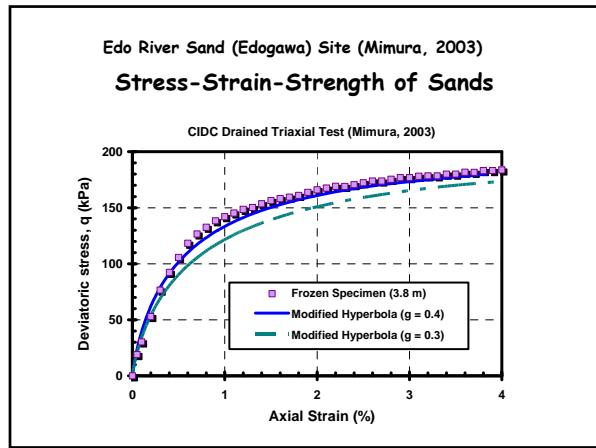
Represented DSS Responses for Onsøy, Norway (Lacasse, Berre, & Lefebvre, ICSMFE, 1995)

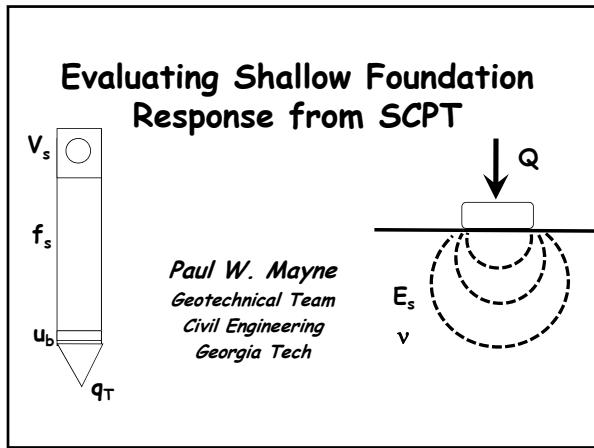


Representative SCPTu from Bangkok, Thailand (Shibuya, 1998, ISC)





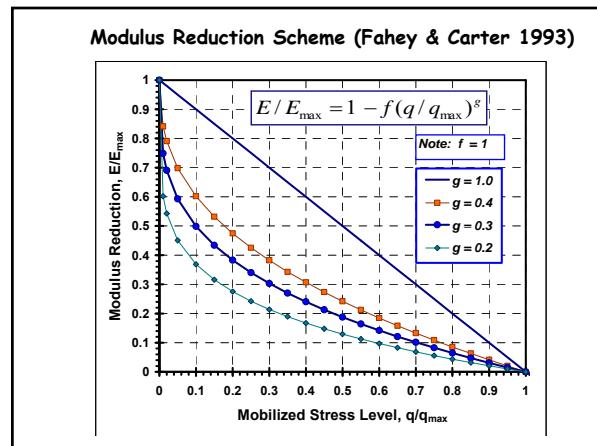
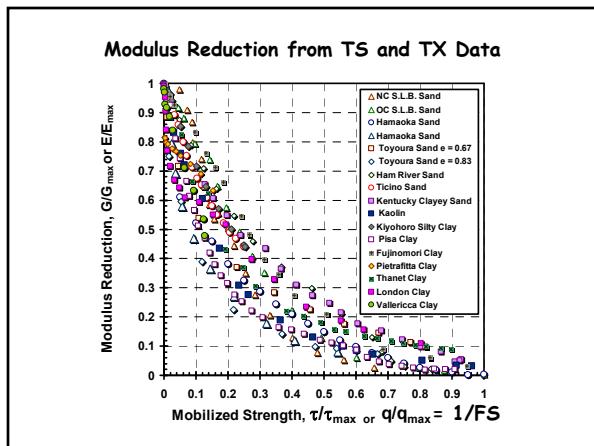
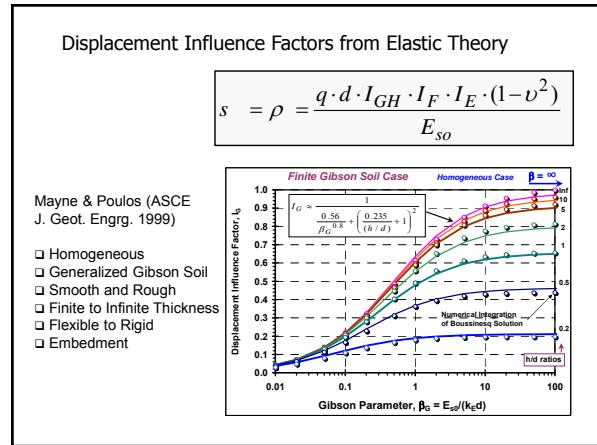
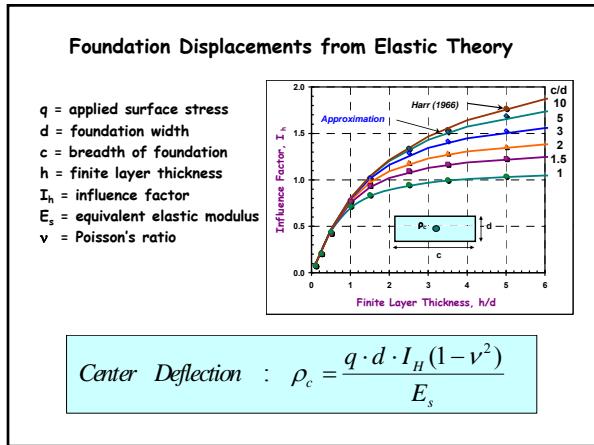




Displacements for Uniaxial Case

Total Deflection: $\rho_{\text{total}} = \sum \rho_i$

Δz_1	$\frac{\Delta \sigma_{z1}}{E_1} = \varepsilon_1$	$\rho_1 = \varepsilon_1 \Delta z_1$
Δz_2	$\frac{\Delta \sigma_{z2}}{E_2} = \varepsilon_2$	$\rho_2 = \varepsilon_2 \Delta z_2$
Δz_3	$\frac{\Delta \sigma_{z3}}{E_3} = \varepsilon_3$	$\rho_3 = \varepsilon_3 \Delta z_3$



Equivalent Modulus for Foundation Response

- o Initial Stiffness from Small-Strain Shear Modulus
 - $G_{\max} = \rho V_s^2$
 - $E_{\max} = 2G_{\max}(1+\nu)$
- o Modified Hyperbola for Reducing Modulus with Load Level:
 - $E/E_{\max} = 1 - f(q/q_{ult})^g = 1 - (FS)^{-g}$
 - where $FS = q_{ult}/q$
 - "Well-behaved" soils: $f = 1$ and $g \approx 0.3$
(not for cemented, highly-structured geomaterials)

Limit Plasticity Solution for Bearing Capacity

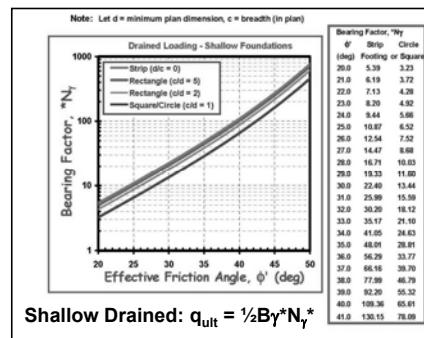
$$q_{ult} = c N_c + \frac{1}{2} B \gamma N_\gamma + \sigma_{vo}' N_q$$

↑ ↑ ↑ ↑
 undrained undrained drained drained
 loading of loading of loading of loading of
 shallow fdns deep fdns shallow fdns deep fdns

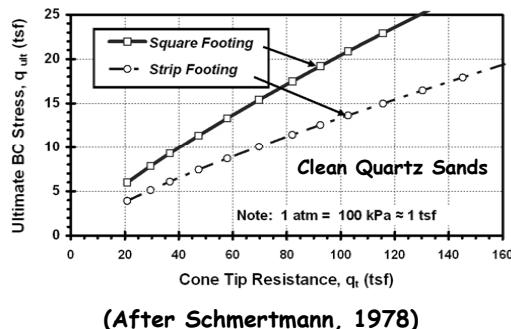
Bearing Capacity of Spread Footings

- UNDRAINED LOADING ($\Delta V/V_o = 0$)
 - $q_{ult} = *N_c s_u$ > Strip Footing: $*N_c = 5.14$
 > Square/Circle: $*N_c = 6.14$
 - (a) $s_u = \frac{1}{2} \sin \phi' \sigma_{vo}' OCR^{(1-C_s/C_c)}$
 - (b) $s_u = 0.23 OCR^{0.80}$ (default values)
 - (c) $s_u = 0.2 \sigma_p'$ in soft clays with $OCR < 2$
- DRAINED LOADING ($\Delta u = 0$)
 - $q_{ult} = \frac{1}{2} B * \gamma * N_\gamma$ $* \gamma = \gamma_T$ (no GWT)
 $* \gamma = \gamma'$ (high GWT)

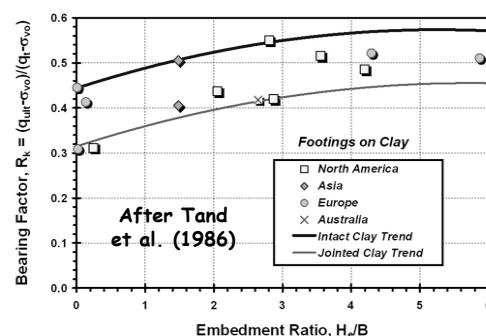
Bearing Capacity

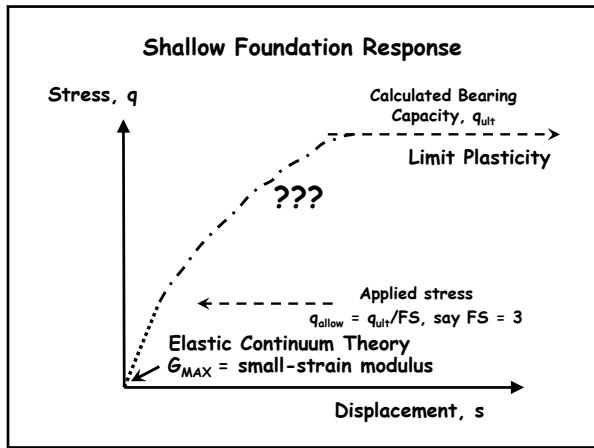


Direct CPT Methods for Bearing Capacity



Direct CPT Methods for Bearing Capacity



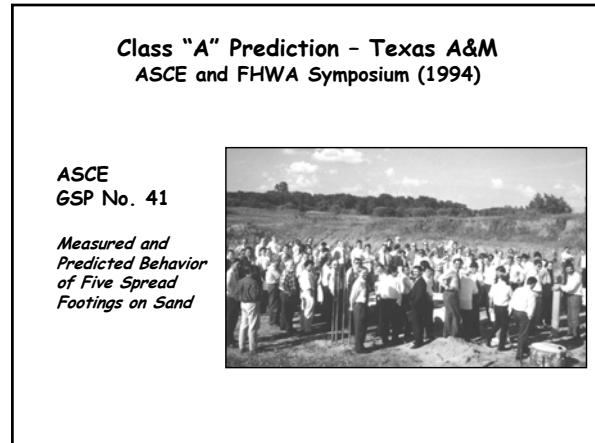
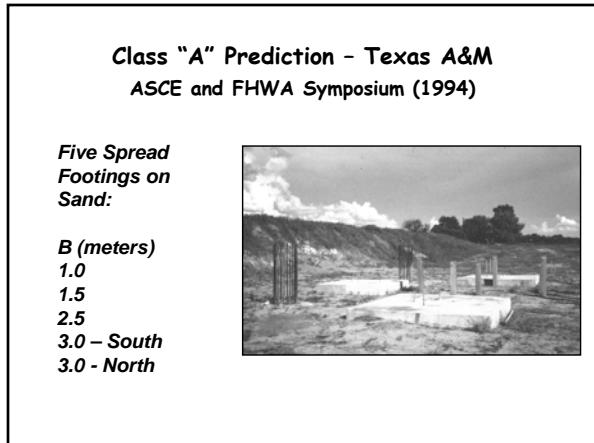
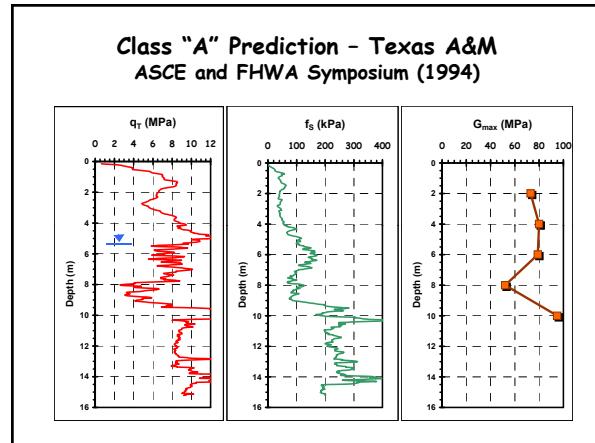
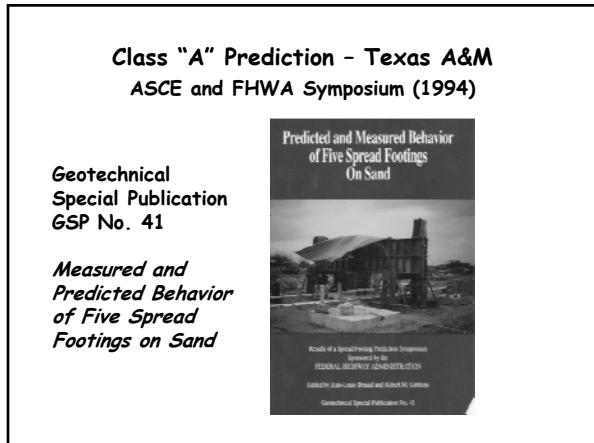


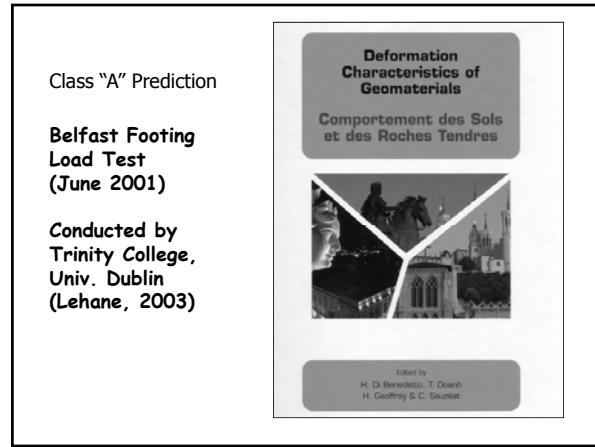
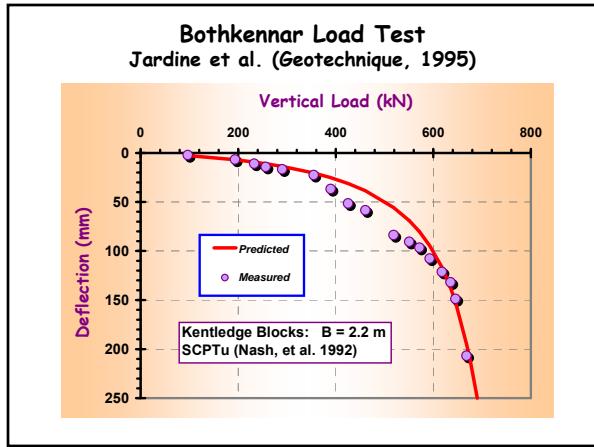
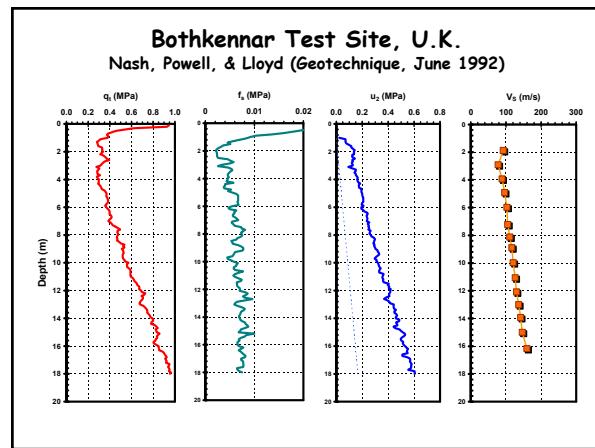
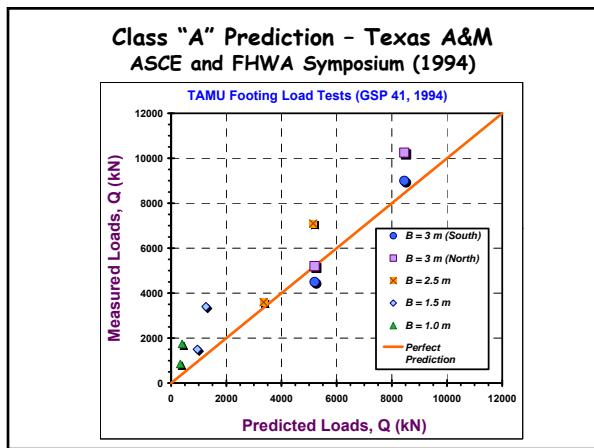
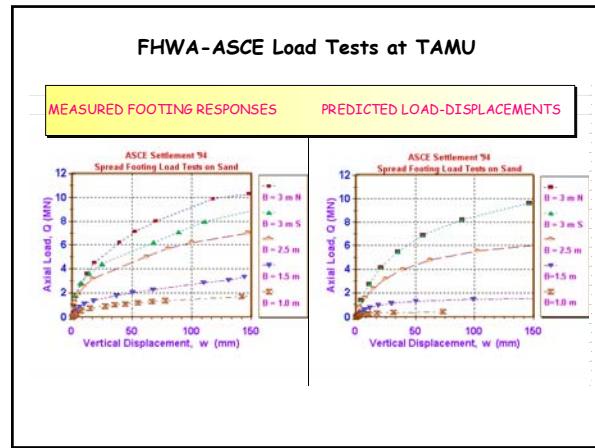
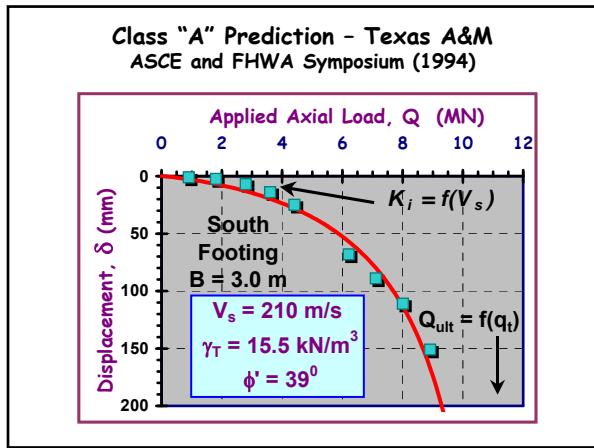
Nonlinear Foundation Deflection Analyses

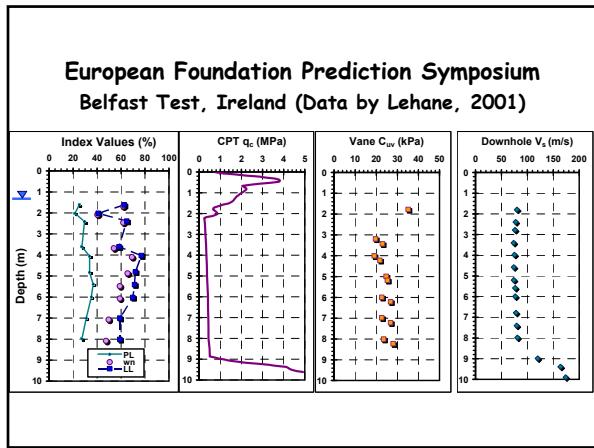
$$S_{center} = \frac{q \cdot d \cdot I_G \cdot I_F \cdot I_E (1 - v^2)}{E_{MAX} [1 - (q / q_{ult})^{0.3}]}$$

where

- q = applied surface stress;
- q_{ult} = ultimate bearing stress;
- d = equivalent footing diameter
- I_G , I_F , I_E = elastic factors for modulus variation, rigidity, and embedment, respectively.
- v = Poisson's ratio
- E_{max} = initial elastic modulus







Geotechnical News (December 2001)

Professor Mayne a Top Predictor of Ultimate Bearing Capacity

JOHN INSTITUTE NEWS

<http://www.johninst.ac.uk/~sfcl/geotech-news.html>

http://www.johninst.ac.uk/~sfcl/geotech-news.html

Professor O'Donohue's 20th Bernstein lecture at Columbia University on November 13, 2001, was entitled "What is New in Deep Soil Foundations".

Joe O'Donohue has degrees from Columbia, Harvard, and Cornell Universities. After graduation he worked during summers for foundation contractors, his first full-time employment was with the U.S. Army Corps of Engineers in the Waterways Experiment Station, Mississippi River Project. After leaving WES, he entered the academic world at Northeastern University, he made the Chair of Geotechnical Engineering.

He founded and built the soils laboratory, and developed the O'Donohue Hydromechanical Tester. Since his retirement he has continued consulting and developing the O'Donohue Load Testing System, which includes small shafts and driven piles. He holds 10 patents.

O'Donohue is a former Chairman of the Soil Mechanics and Foundation Engineering Committee of ASCE, and a member of the Geotechnical Society of America. He delivered the 1990 GSP Lecture in 1985, and later received the Terzaghi

Remember Lecture
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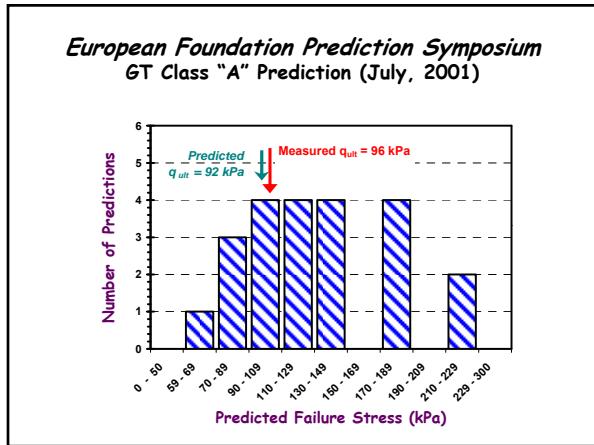
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Professor Mayne

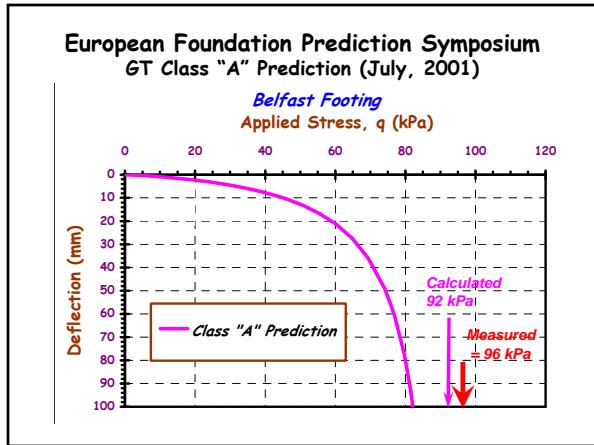
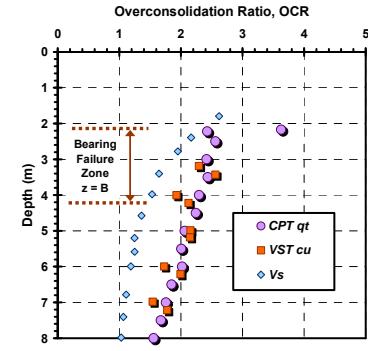
Mayne geotechnical experts recently accepted an invitation to present a paper on the prediction of the ultimate capacity of a pile footing in sand.

The following three were the top predictions:

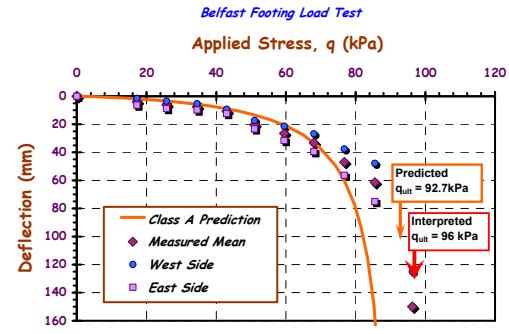
- o **Mayne - USA**
- o **Orr - Ireland**
- o **Murry - NZ**



European Foundation Prediction Symposium GT Class "A" Prediction (July, 2001)



European Foundation Prediction Symposium GT Class "A" Prediction (July, 2001)



Belfast Footing Load Test GT Class "C" Prediction

1. Preferred Undrained Strength Equation (CSSM):

$$s_u/\sigma_{vo}'_{DSS} = \frac{1}{2}\sin\phi' OCR^\Lambda \quad \text{where } \Lambda = 1 - C_s/C_c$$

2. Empirical (MIT - SHANSEP): Default:

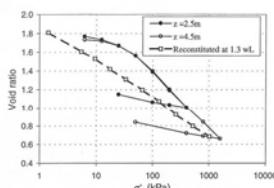
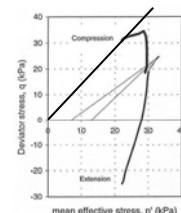
$$s_u/\sigma_{vo}'_{DSS} = 0.22 OCR^{0.80}$$

3. Empirical at Low OCRs < 2 (VST, SHANSEP):

$$s_u = 0.22 \sigma_p'$$

Belfast Footing Load Test

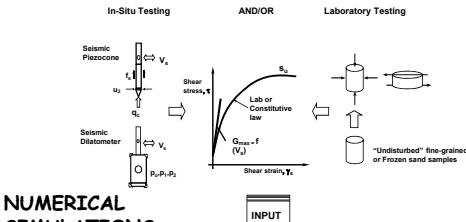
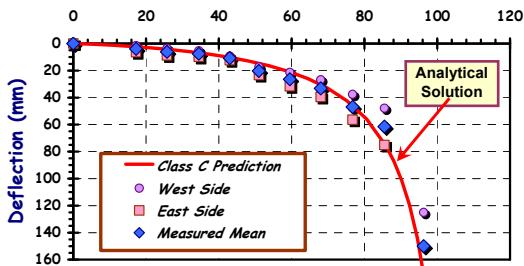
Lehane (2003) Paper in Geotechnical Engineering



- 15 CAUC Triaxials: $c'=0$ and $\phi' = 33.5^\circ \pm 1.5^\circ$
- Consols: $C_c = 0.65$; $C_s = 0.13$; $\Lambda = 0.80$

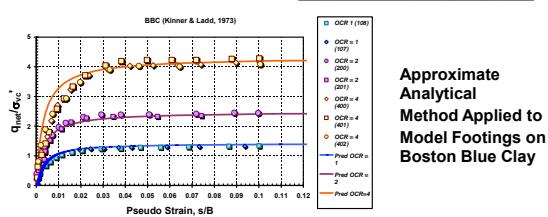
Belfast Footing Load Test GT Class "C" Prediction

Belfast Footing Load Test Applied Stress, q (kPa)



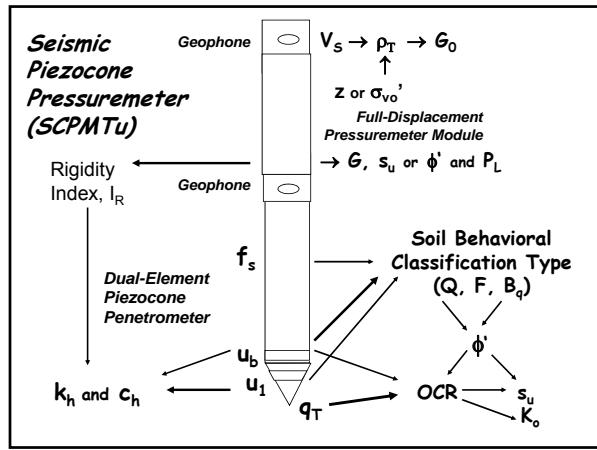
NUMERICAL SIMULATIONS For Footing Response

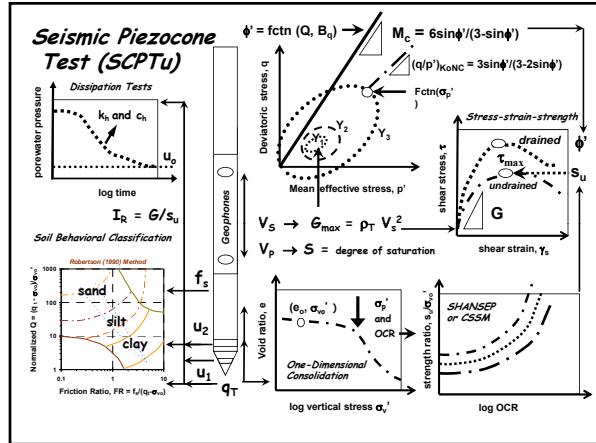
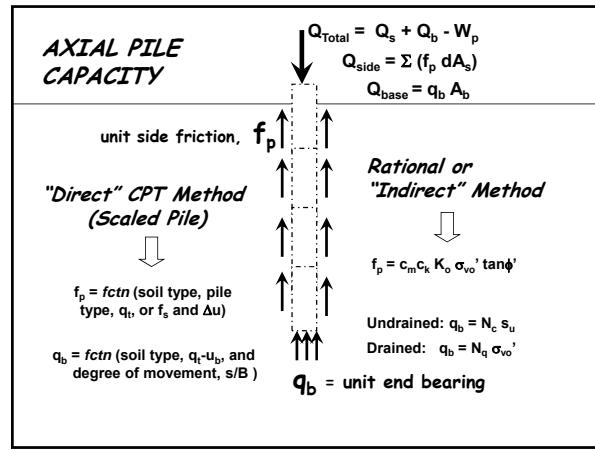
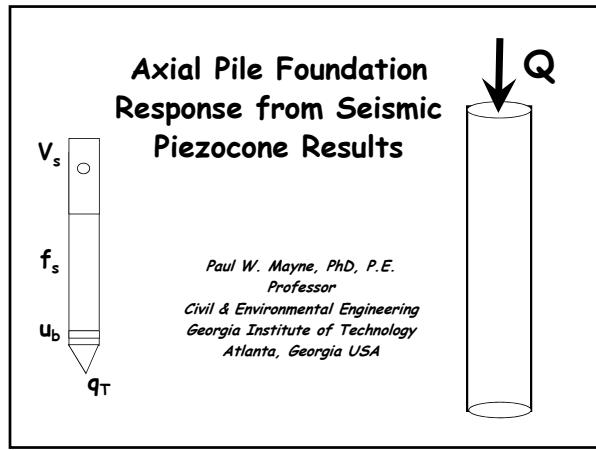
- FLAC Mesh for modeling surface footings
- Calibration with 23 Full-Scale Load Tests
- Approx. Analytical Method



SUMMARY: Footing Response from SCPT

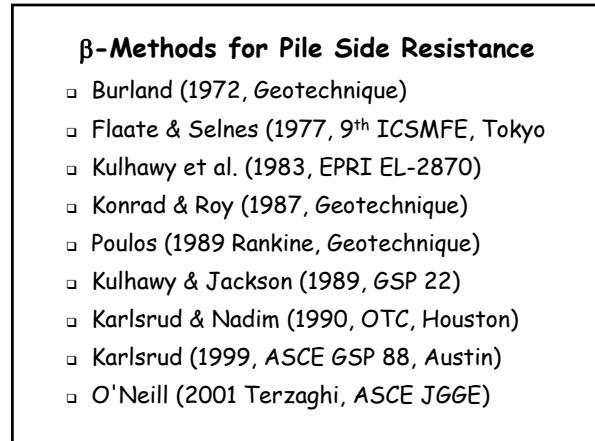
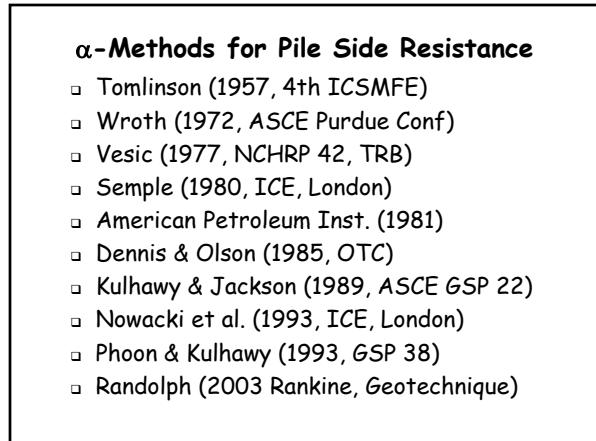
- Hybrid tests (SCPTu and SDMT) with dissipation phases provide optimization of data collection: q_t , f_s , u_b , V_s , and $\Delta u(t)$
- Shear Wave Velocity provides fundamental measurement of soil stiffness: G_{MAX}
- Modulus Reduction Schemes (Hardin-Drnevich, Ramberg-Osgood, Fahey-Carter, Puzrin-Burland)
- Approximate nonlinear Q- δ - Q_{ult} using elastic continuum solutions for shallow foundations





Rational (Indirect) CPT Methods

- End Bearing: q_b = pile tip/base resistance from limit plasticity, cavity expansion, limit equilibrium
- Side Resistance: f_p = pile side friction
 - α method: $f_p = \alpha s_u$ and $\alpha = f_{\text{ctn}}(s_u)$
 - β method: $f_p = \beta \sigma_{vo}'$ and $\beta \approx K_0 \tan \phi'$
 - λ method (offshore)
 - effective stress methods
 - numerical simulations



Direct CPT Methods (using q_c and/or f_s)

- Aoki & de Alencar (1975, 5th PanAm, Buenos Aires)
- Schmertmann (1978, FHWA TS-78-209)
- de Ruiter & Beringer (1979, Marine Geotechnology)
- Bustamante and Ganeselli (1982, LCPC)
- Zhou et al. (1982, ESOPT, Amsterdam)
- Tumay & Fakhroo (1982, ASCE CPT Experience)
- Price & Wardle (1982, ESOPT, Amsterdam)
- Van Impe (1986, 4th Intl. Geot. Seminar, Singapore)
- Robertson et al. (1988, ISOPT-1, Orlando)
- Alsamman (1995, PhD, Univ. Illinois-Urbana)
- Fioravante et al. (1995, 10th Asian Reg. Conf)
- Abu-Farsakh & Titi (2004, ASCE JGGE)

Direct CPTU Methods (q_t , f_s , and/or u_b)

- Almeida et al. (1996, Canadian Geot. J.)
- Eslami and Fellenius (1997, Can. Geot. J.)
- Jardine & Chow (1996, Marine Tech Director)
- Takesue et al. (1998, ISC-1, Atlanta)
- Lee & Salgado (1999, ASCE JGGE)
- Powell, et al. (2001, 15th ICSMGE, Istanbul)
- Karlstrud et al. (NGI 2005)
- Kolk et al. (Fugro 2005)
- Lehane & Schneider (UWA 2005)

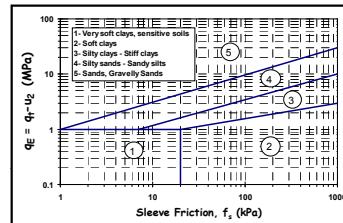
Georgia Tech

Direct CPT Method for Driven Piles in Clays (Almeida et al., 1996; Powell et al. 2001)

- Define net cone resistance: $q_{t\text{net}} = q_t - \sigma_{vo}$
- Unit Pile Side Friction ($L/d < 60$): $f_p = q_{t\text{net}}/k_1$
where $k_1 = 10.5 + 13.3 \log(q_{t\text{net}}/\sigma_{vo}')$
- Unit Pile End Bearing: $q_b = q_{t\text{net}}/k_2$
where $k_2 \approx 2$
- Value of k_2 depends on installation and clay consistency (driven piles: $k_2 = 2.7$; jacked piles: $k_2 = 1.5$ for soft clay and 3.4 for stiff clay)

Georgia Tech

Eslami and Fellenius (1997, Canadian Geot. J.)



UNICONE METHOD:

- A. Define effective cone resistance:
 $q_E = q_t - u_2$

- B. Plot q_E vs f_s for soil type

- C. Pile Side:
 $f_p = C_s q_E$

- D. Toe: $q_b \approx q_E$

Georgia Tech

MTD Method (Jardine and Chow, 1996) Driven piles in sands

Side Resistance

$$\tau_f = \sigma'_{rf} \tan(\delta_f)$$

$$\sigma'_{rf} = \sigma'_{rc} + \Delta \sigma'_{rd}$$

$$\sigma'_{rc} = 0.029 q_c \left(\frac{\sigma'_{vo}}{P_a} \right)^{0.13} \left(\frac{h}{R} \right)^{-0.38}$$

$$\Delta \sigma'_{rd} = 2 G \frac{\delta_h}{R}$$

$$q_b = q_c \left[1 - 0.5 \log \left(\frac{D}{D_{CPT}} \right) \right]$$

YSR	yield stress ratio = OCR
σ'_{rc}	radial effective stress on side after equalization
σ'_{rf}	radial effective stress at maximum shear stress
IVR	relative void index (see Burland, 1990, Geot.)
h	height above the pile tip
G	sand shear modulus
δ_f	pile-soil interface angle at max. shear stress
δ_h	$2 \cdot R_{CLA} \approx 2.0 \times 10^{-5} \text{ m}$ for typical offshore piles

b Pile diameter

D_{CPT} Cone diameter

$K_c = \sigma'_c / \sigma'_{vo}$

S_i clay sensitivity

q_c total cone stress

q_b tip resistance

R pile radius

$\tau_f = f_p = \text{pile side friction}$

MTD Method (Jardine and Chow, 1996) Driven piles in clay

Side Resistance

$$\tau_f = \sigma'_{rf} \tan(\delta_f)$$

$$\sigma'_{rf} = 0.8 \sigma'_{rc}$$

$$\sigma'_{rc} = K_c \sigma'_{vo}$$

$$\Delta I_{vv} = \log(S_i)$$

Tip Resistance

Drained loading

$$q_b = 0.8 q_c$$

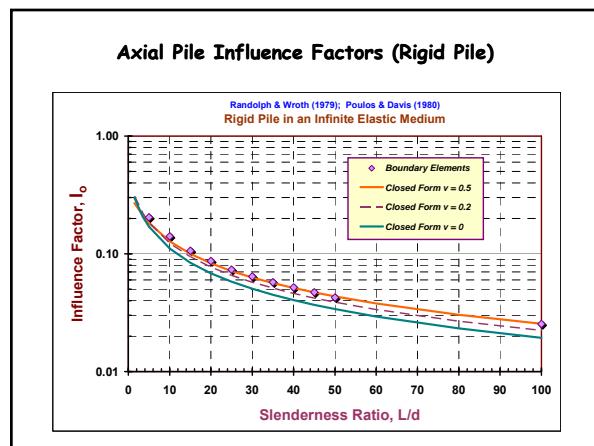
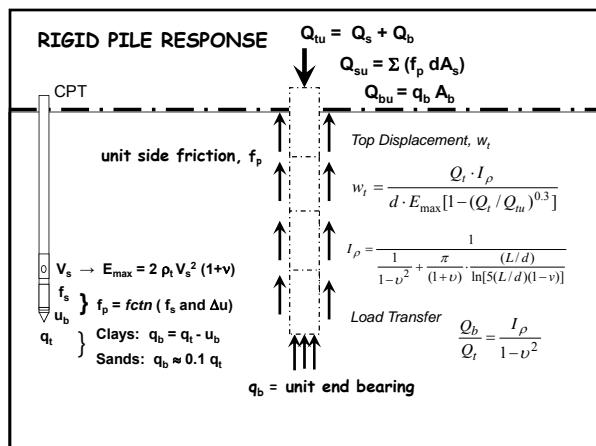
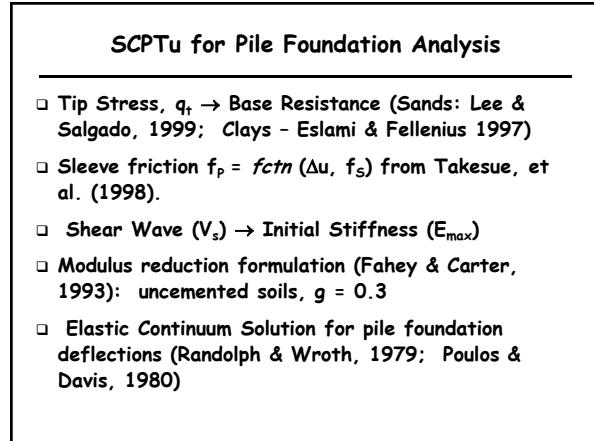
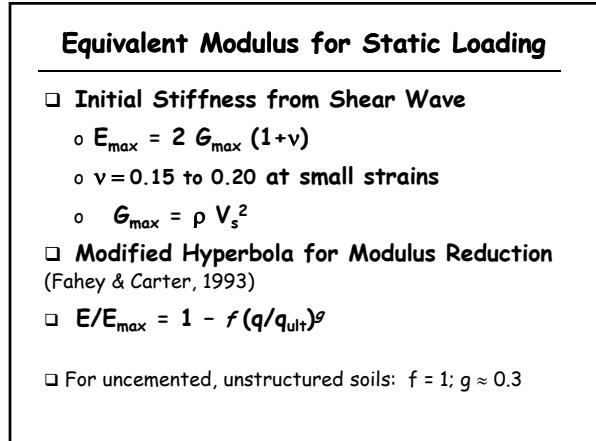
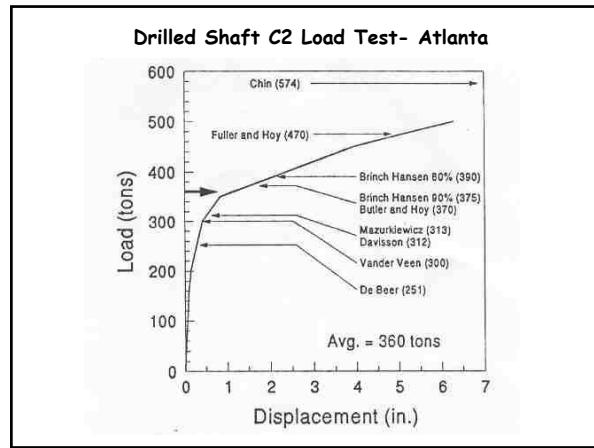
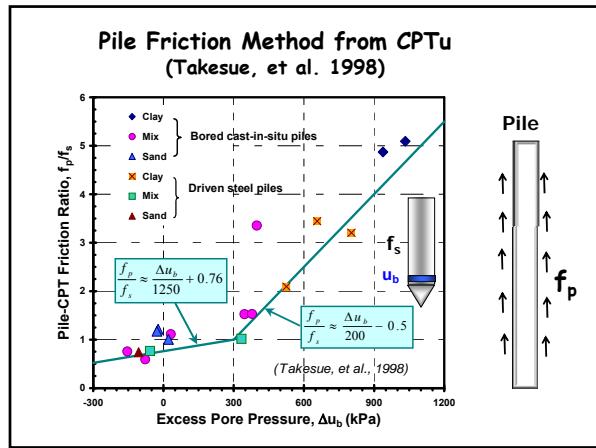
Undrained loading

$$q_b = 1.3 q_c$$

$$K_c = \left(2 - 0.625 I_{vv} \right) YSR^{0.42} \left(\frac{h}{R} \right)^{-0.20}$$

$$\text{or } K_c = \left(2.2 + 0.016 YSR - 0.870 \Delta I_{vv} \right) YSR^{0.42} \left(\frac{h}{R} \right)^{-0.20}$$

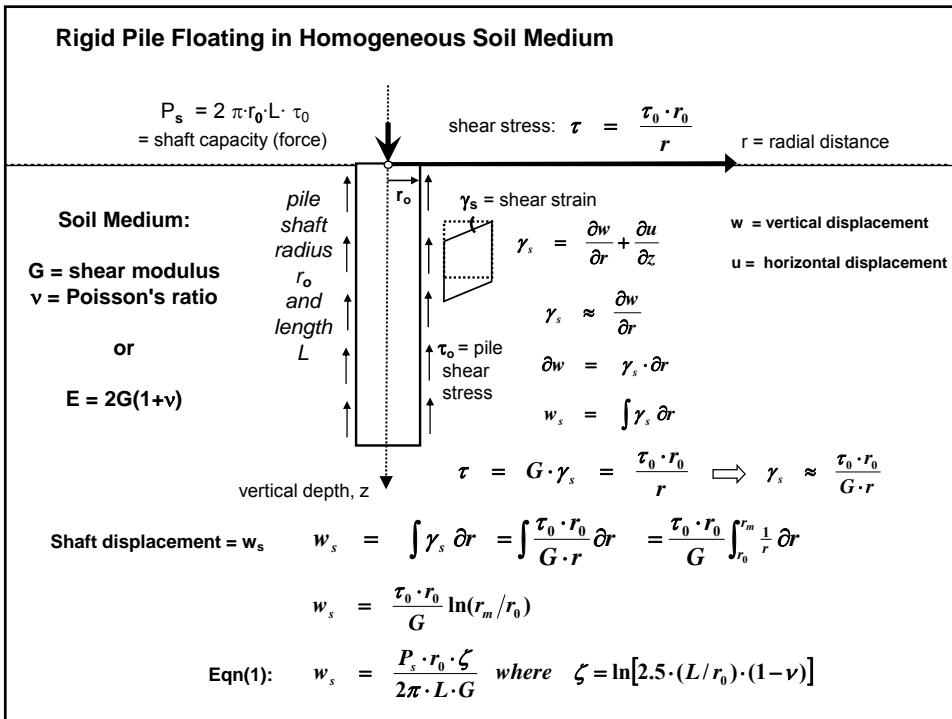
$$(\text{Lehane et al., 2000}): K_c = 0.27 \left(2 - 0.625 I_{vv} \right) \left(\frac{q_t}{\sigma'_{vo}} \right)^{0.7} \left(\frac{h}{R} \right)^{-0.20}$$

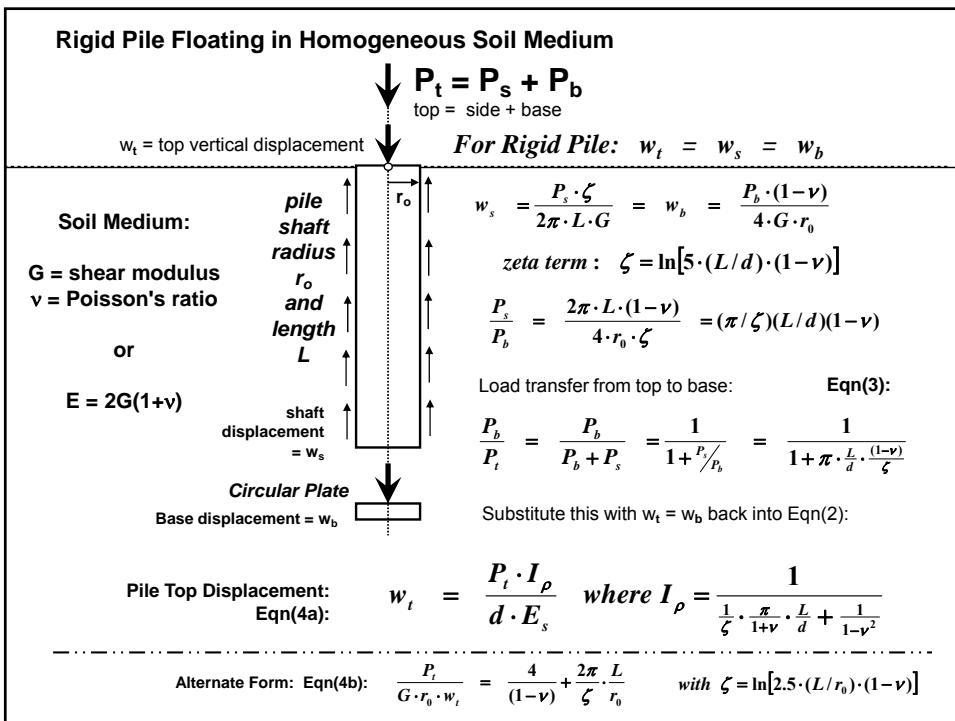
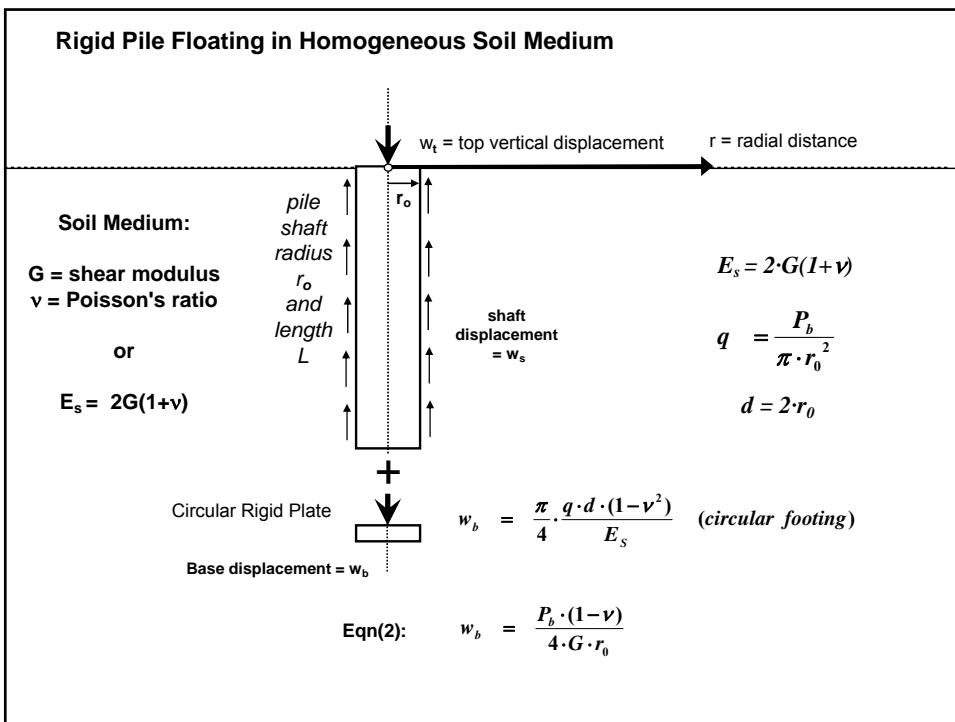


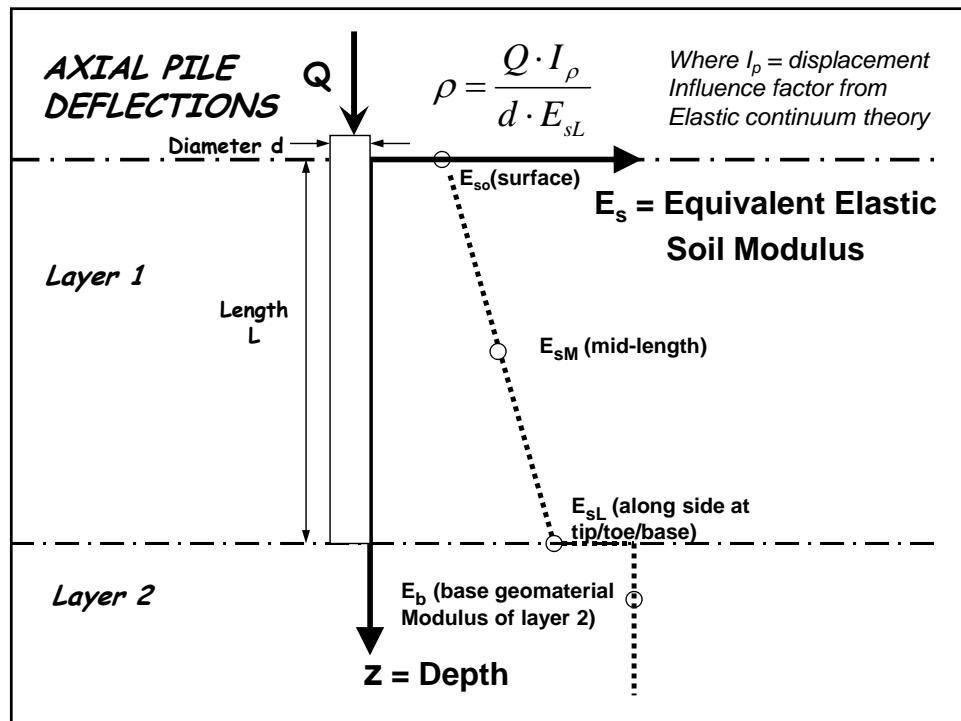
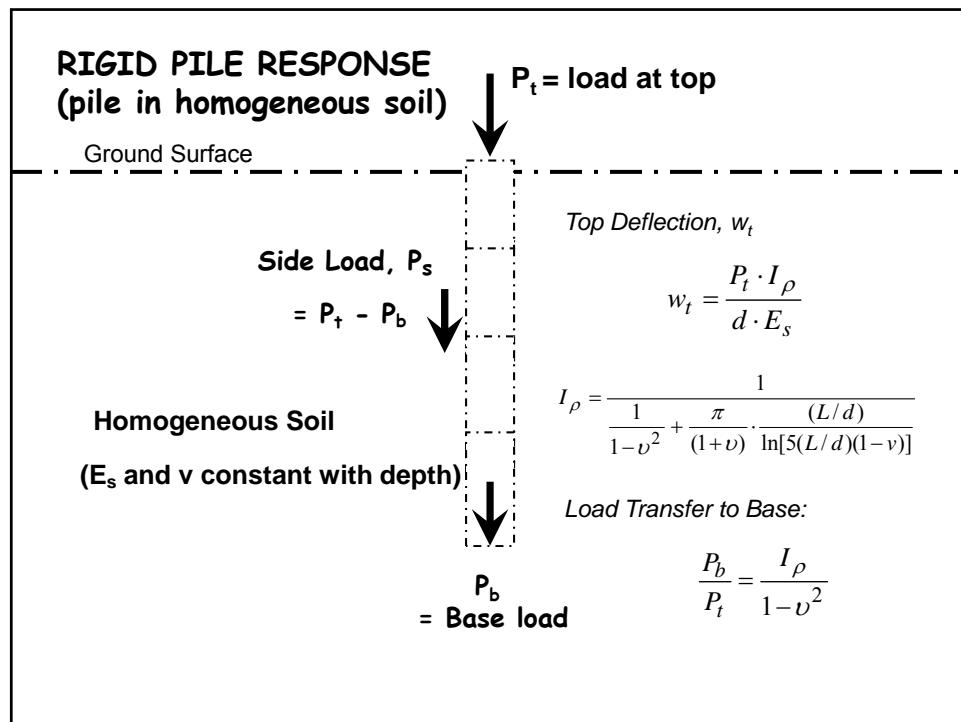
Elastic Continuum Solutions for Axial Pile Response

Based on Mark Randolph Solutions

(Randolph & Wroth, 1978 ASCE JGE, 1979 ASTM STP 777; Fleming et al., 1992: *Piling Foundations*)







RIGID PILE RESPONSE

(two layer soil system)

Ground Surface

Generalized Gibson Soil:

$$E_s = E_{so} + k_E \cdot z$$

Alternately represented by:

Rate parameter $\rho_E = E_{sm}/E_{sL}$

$$\rho_E = 1 - \frac{1}{\frac{2 \cdot E_{so}}{k_E \cdot L} + 2}$$

Layer 1 (side geomaterial)

Layer 2 (base geomaterial)

Load Transfer:

P_t = top load

Top Deflection, w_t

$$w_t = \frac{P_t \cdot I_\rho}{d \cdot E_s}$$

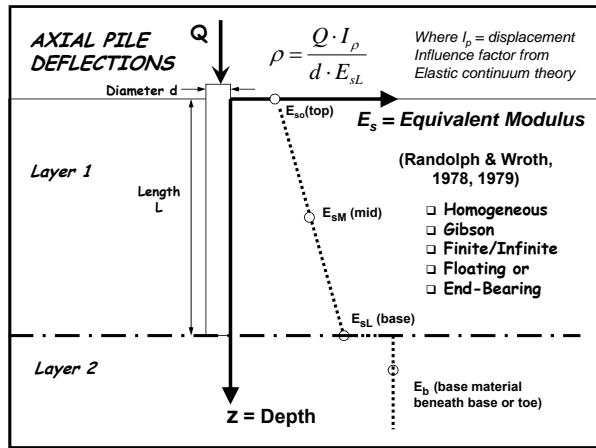
$$I_\rho = \frac{1}{\frac{E_b}{E_{sL}} \frac{1}{1-\nu^2} + \frac{\pi \cdot \rho_E}{(1+\nu)} \cdot \frac{(L/d)}{\zeta_p}}$$

$$\zeta_p = \ln \left(5 \cdot (L/d) \left[\frac{E_{sL}}{E_b} \{ \rho_E (1-\nu) - 0.1 \} + 0.1 \right] \right)$$

side load, $P_s = P_t - P_b$

P_b = base load

$$\frac{P_b}{P_t} = \frac{1}{1 + \frac{\pi \cdot \rho_E}{\zeta_p} \cdot \frac{E_{sL}}{E_b} (L/d)(1-\nu)}$$



RIGID PILE RESPONSE
(two layer soil system)

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side load, P_s = P_t - P_b

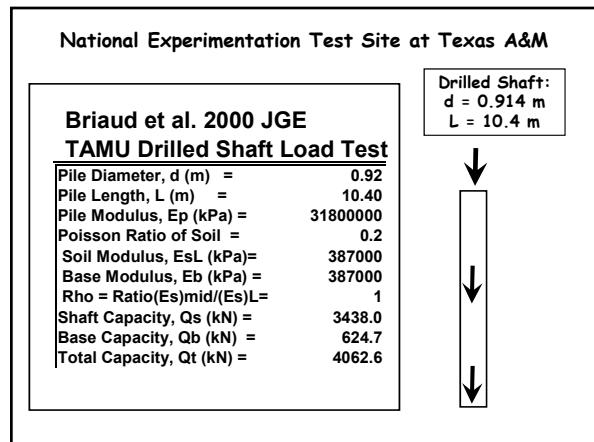
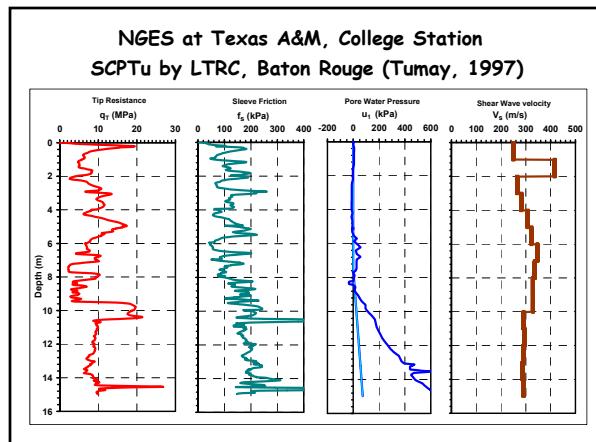
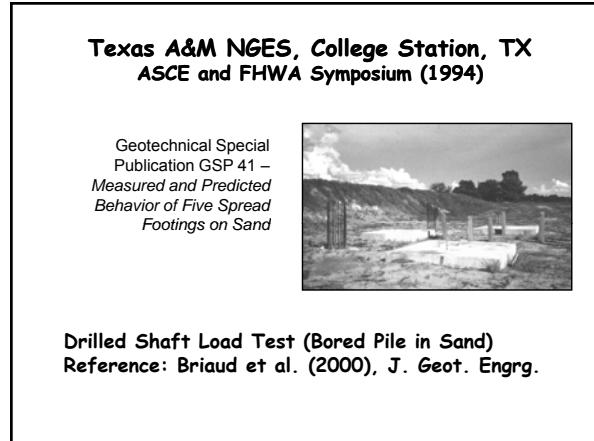
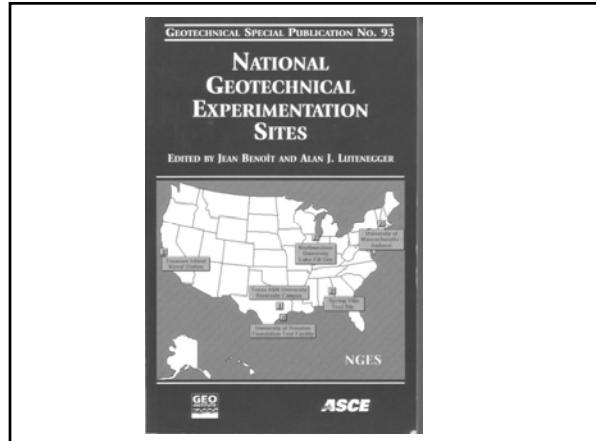
P_b = base load

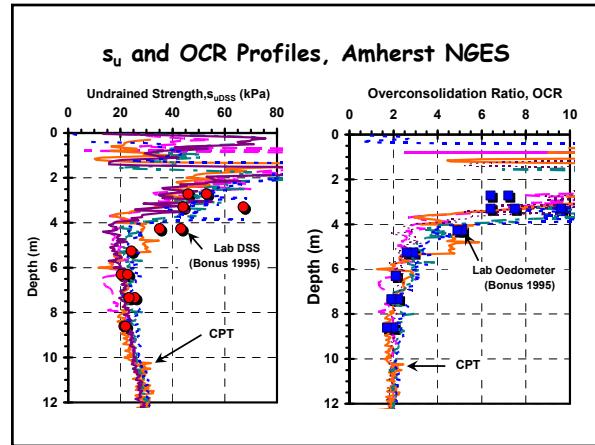
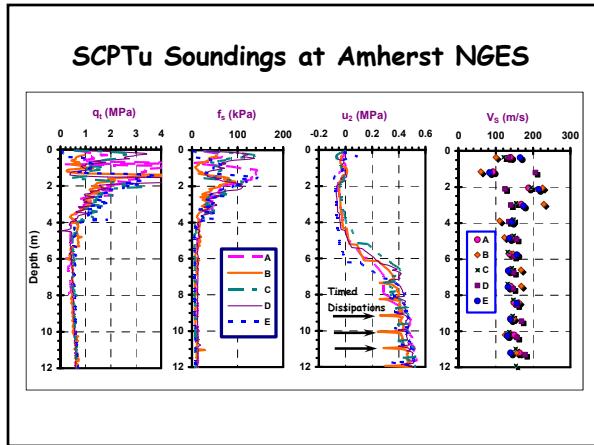
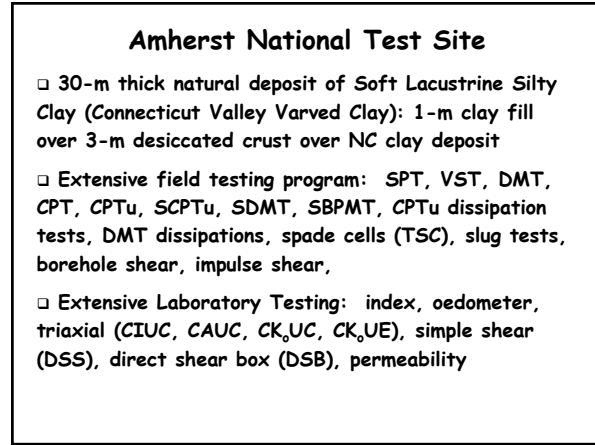
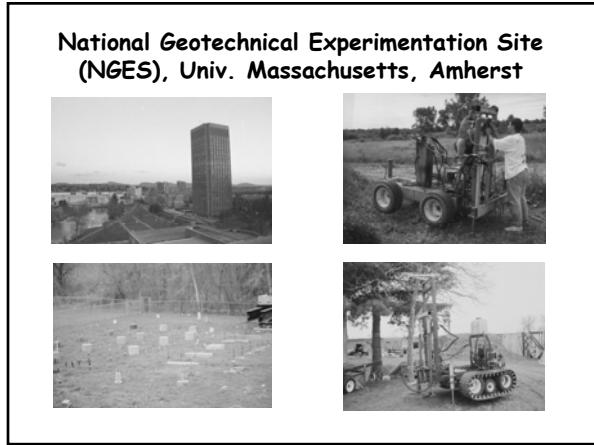
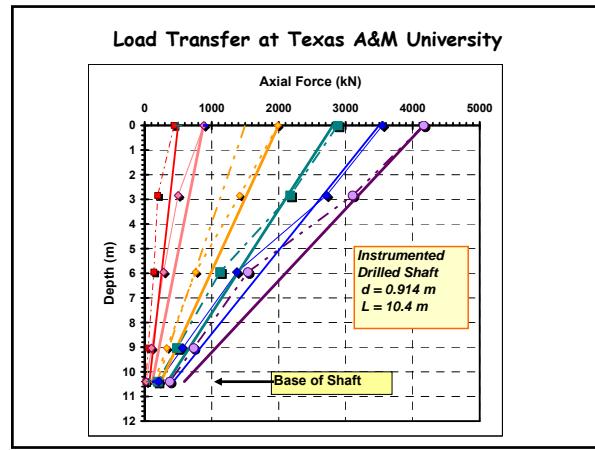
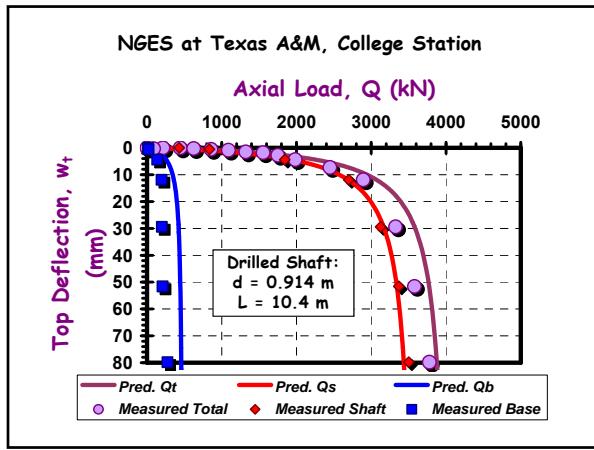
Layer 1 (side geomaterial)

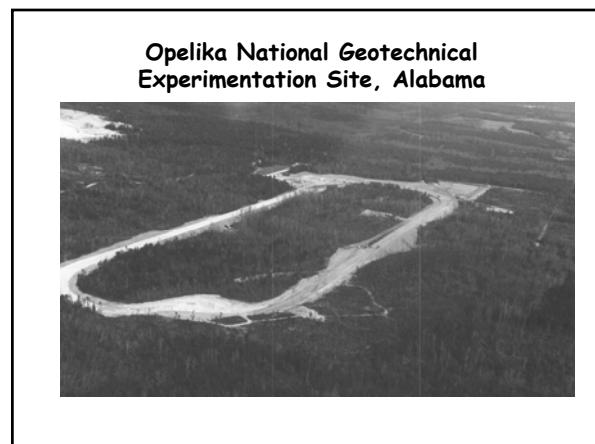
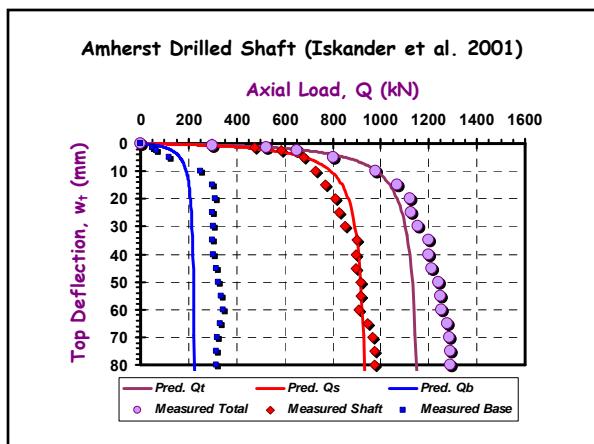
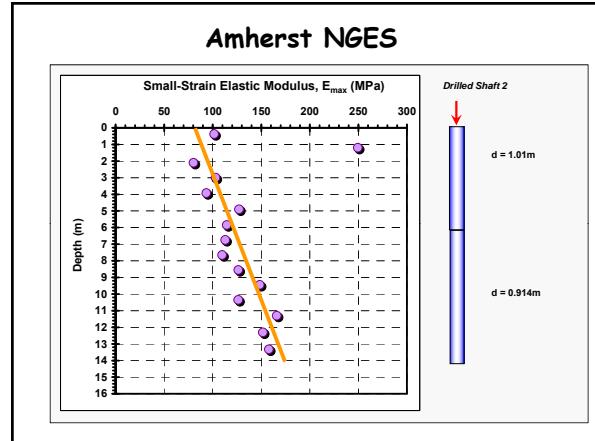
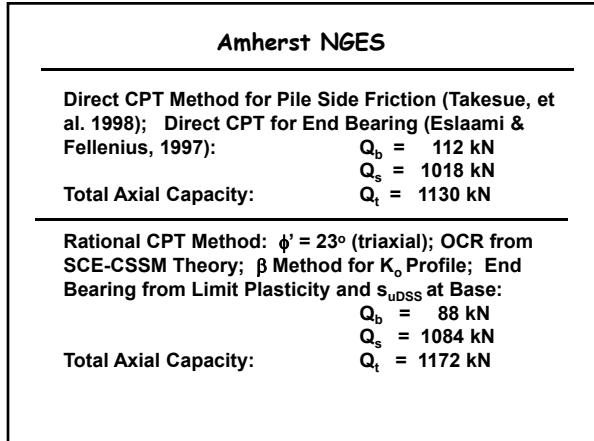
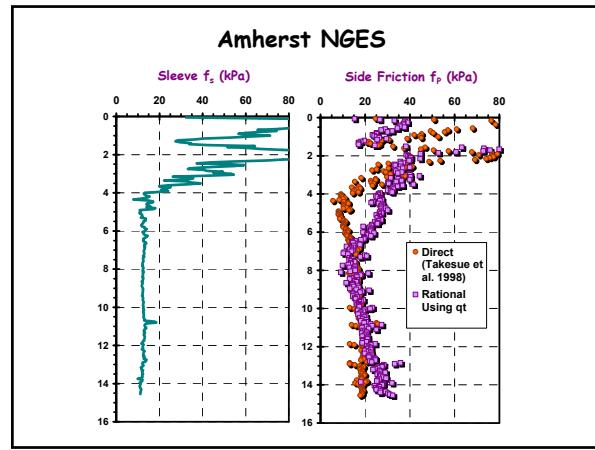
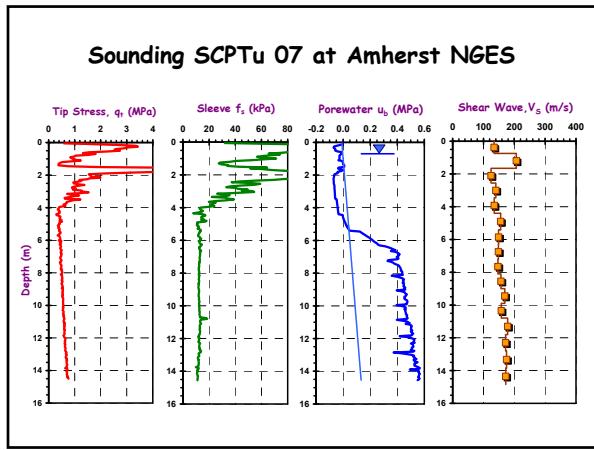
Layer 2 (base geomaterial)

Load Transfer:

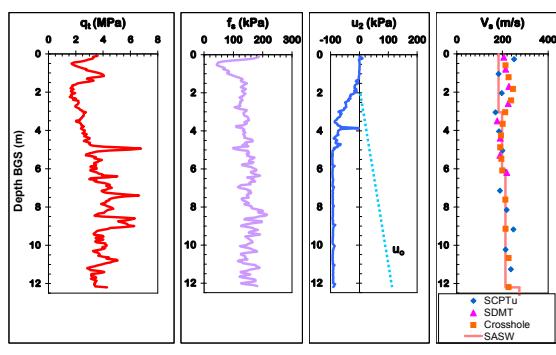
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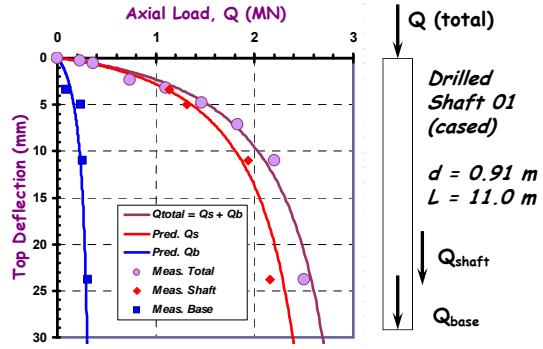




SCPTu at Opelika NGES, Alabama



Axial Bored Pile Load Test at Opelika, Alabama

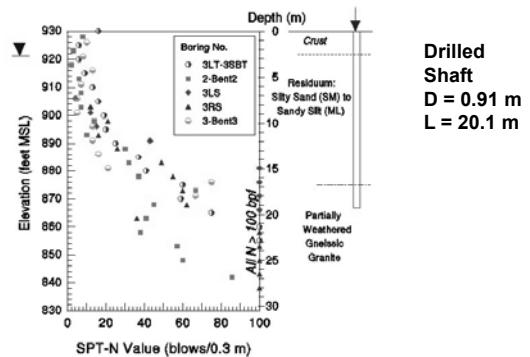


I-85 Bridge, Coweta County, Georgia

Drilled Shaft: d = 0.914 m; L = 19.m



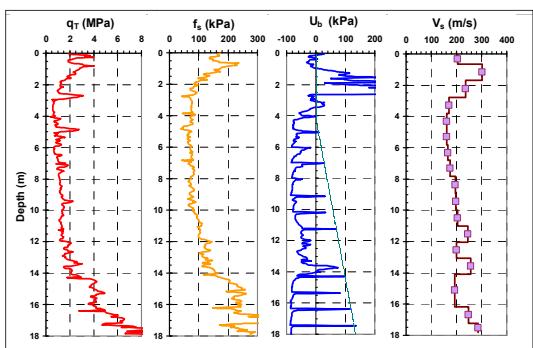
Soil Profile at I-85 Bridge, Coweta County, GA

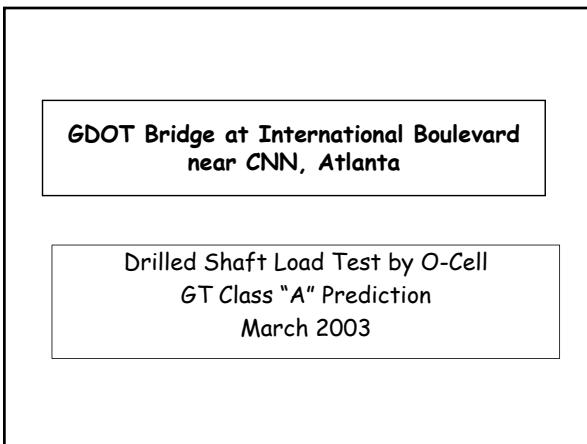
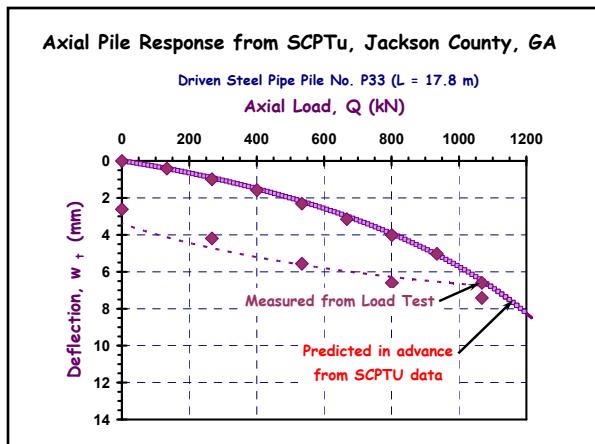
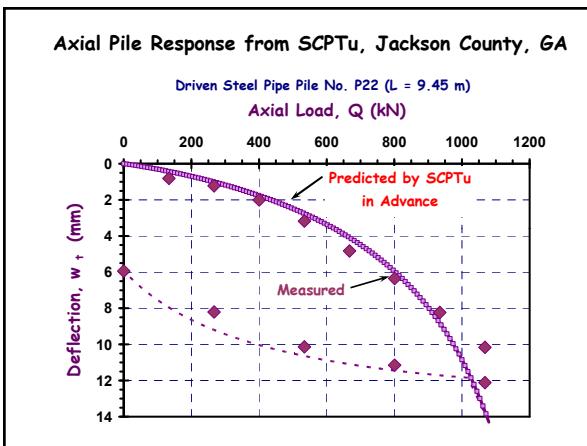
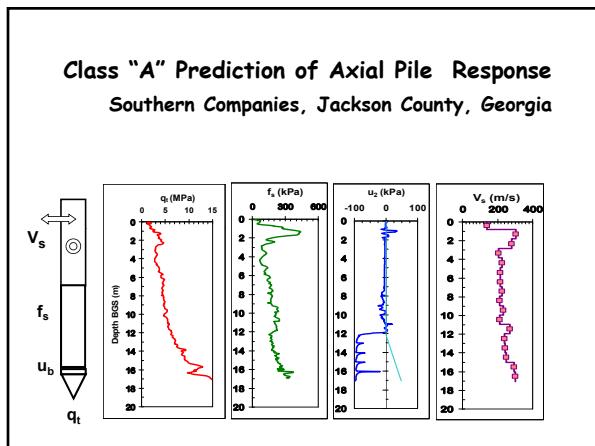
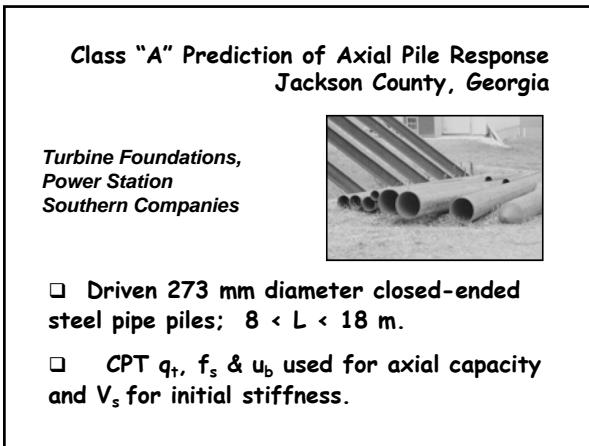
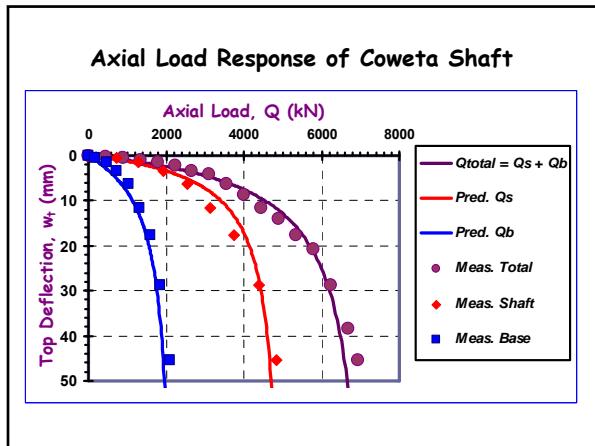


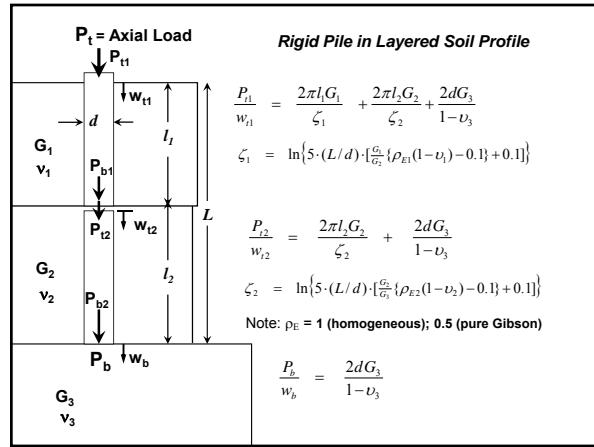
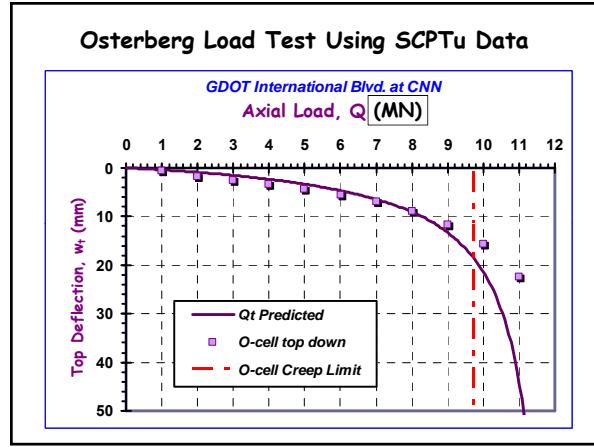
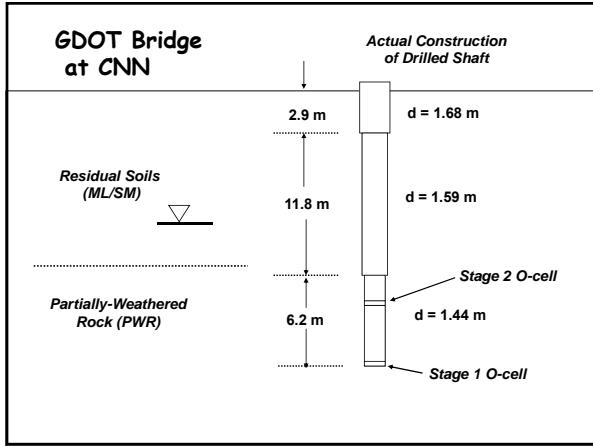
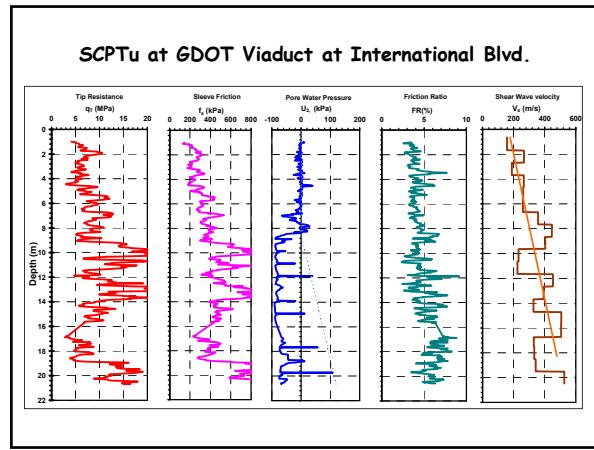
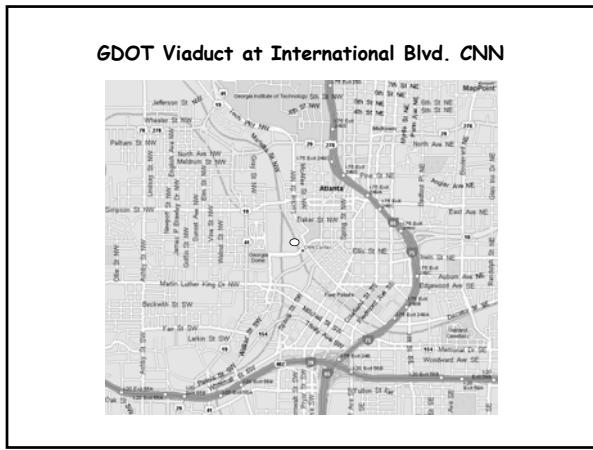
Cone Rig at I-85 Bridge, Newnan, Georgia



SCPTu at I-85 Bridge, Coweta County, GA







Conclusions - Axial Pile Response by SCPT

- Hybrid tests (SCPT_ü and SDMT_a) for optimal soil parameter determinations
- Fundamental soil stiffness: $G_0 = \rho V_s^2$
- Modulus Reduction Schemes (mod hyperb)
- Used within elastic continuum framework
- Needs more numerical simulations
- Add intermediate stiffness measurement (SDMT, dilatometer, cone pressuremeter)

INTERNATIONAL SITE CHARACTERIZATION

ISC-3, Taipei, Taiwan 01-04 April 2008

□ www.geoforum.com/tc16



GeoMusic



*ISC 2004 Porto
GeoMusic*

Jim Mitchell - sax
Jean Nuyens - piano
Martin Fahey - mandolin
Nino Cruz - guitar
Paul Mayne - bass

Evaluation of Calgary Drilled Shaft by SCPTu



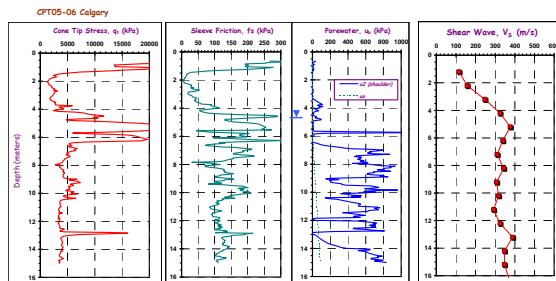
Paul W. Mayne
Class "A" Prediction
16 March 2007

Revised Class "C" Prediction –
May 2008

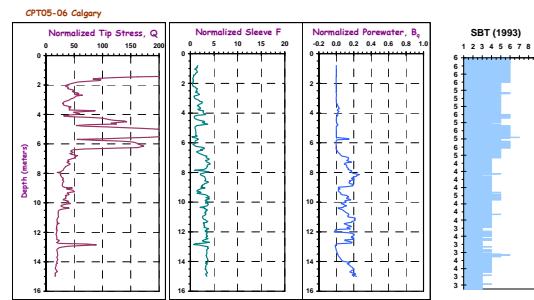
Evaluation of Calgary Drilled Shaft Response

- Foothills Medical Center
- Geotechnical Engineers: Golder Associates
- Piezocone and Seismic Cone Tests by ConeTec Investigations
- Drilled shaft constructed using dry cased method ($d = 0.9 \text{ m}$ and $L = 15 \text{ m}$).
- Fill over sand/silt over hard Clay Till
- Provided soil boring with SPTs
- Static O-cell results by LoadTest

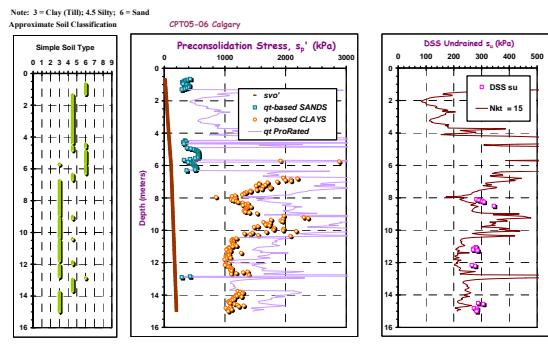
Evaluation of Calgary Drilled Shaft Response by Seismic Piezocone Tests



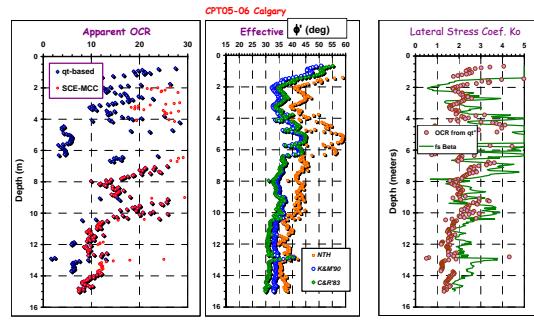
Evaluation of Calgary Drilled Shaft Response by Seismic Piezocone Tests

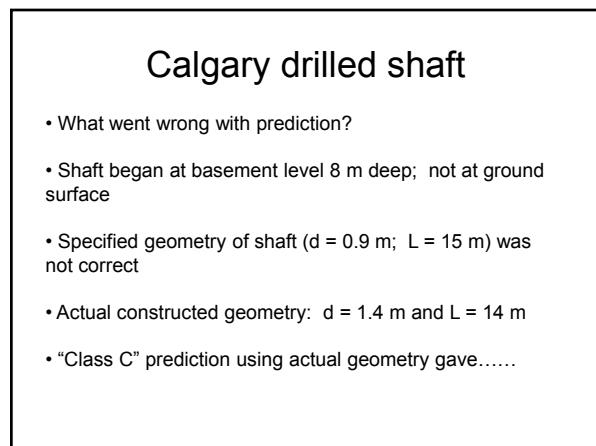
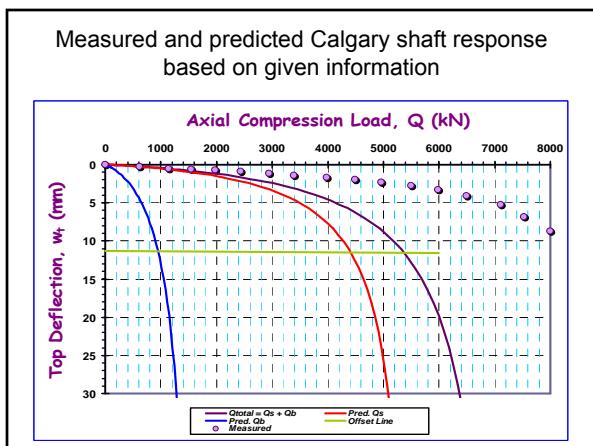
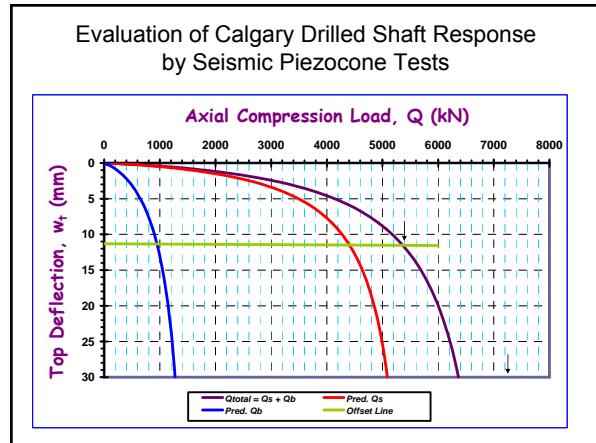
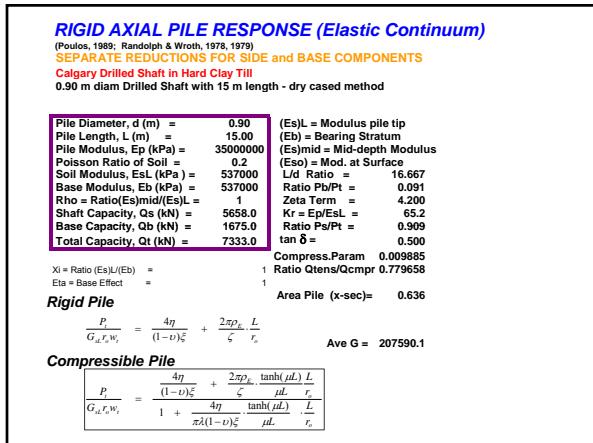
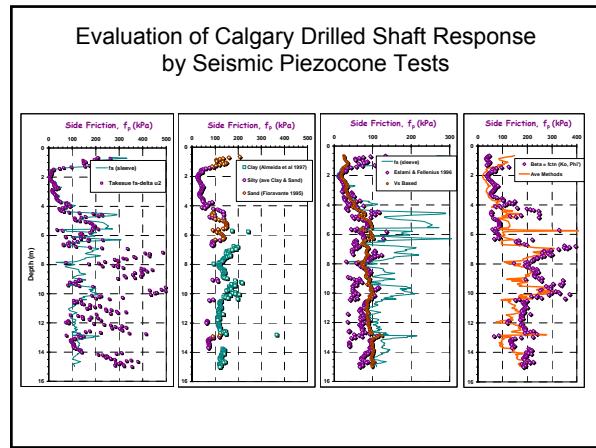
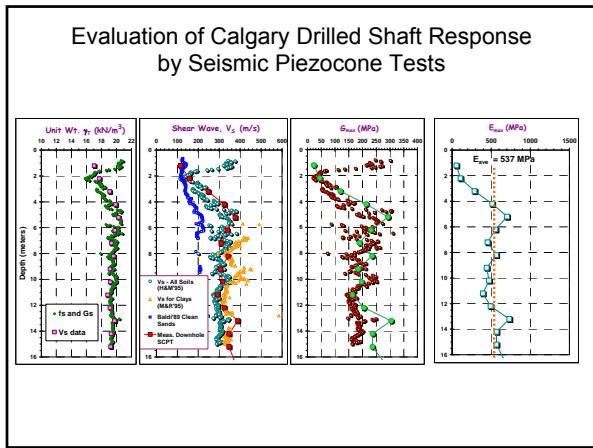


Evaluation of Calgary Drilled Shaft Response by Seismic Piezocone Tests



Evaluation of Calgary Drilled Shaft Response by Seismic Piezocone Tests





RIGID AXIAL PILE RESPONSE (Elastic Continuum)

(Poulos, 1989; Randolph & Wroth, 1978, 1979)

SEPARATE REDUCTIONS FOR SIDE and BASE COMPONENTS

Calgary Drilled Shaft in Hard Clay Till

0.90 m diam Drilled Shaft with 15 m length - dry cased method

Pile Diameter, d (m) =	0.90
Pile Length, L (m) =	15.00
Pile Modulus, E_p (kPa) =	35000000
Poisson Ratio of Soil =	0.2
Soil Modulus, E_{sL} (kPa) =	537000
Base Modulus, E_b (kPa) =	537000
Rho = Ratio(E_s)mid/(E_s)L =	1
Shaft Capacity, Q_s (kN) =	5658.0
Base Capacity, Q_b (kN) =	1675.0
Total Capacity, Q_t (kN) =	7333.0

(E_s)L = Modulus pile tip
(E_b) = Bearing Stratum
(E_s)mid = Mid-depth Modulus
(E_s)o = Mod. at Surface
L/d Ratio = 16.667
Ratio P_b/P_t = 0.091
Zeta Term = 4.200
$K_r = E_p/E_{sL}$ = 65.2
Ratio P_s/P_t = 0.909
$\tan \delta = 0.500$

$$\begin{aligned} X_i = \text{Ratio } (E_s)L/(E_b) &= 1 \\ \text{Eta} = \text{Base Effect} &= 1 \end{aligned}$$

$$\text{Area Pile (x-sec)} = 0.636$$

Rigid Pile

$$\frac{P_t}{G_{sL} r_o w_t} = \frac{4\eta}{(1-\nu)\xi} + \frac{2\pi\rho_E \cdot L}{\zeta \cdot r_o} \quad \text{Ave G} = 207590.1$$

Compressible Pile

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

Evaluation of Calgary Drilled Shaft Response by Seismic Piezocone Tests

NONLINEAR AXIAL PILE RESPONSE

(Poulos, 1989; Randolph & Wroth, 1978, 1979)

Axial Drilled Shaft Calgary, Alberta

Using SCPTU data and Elastic Continuum Soln

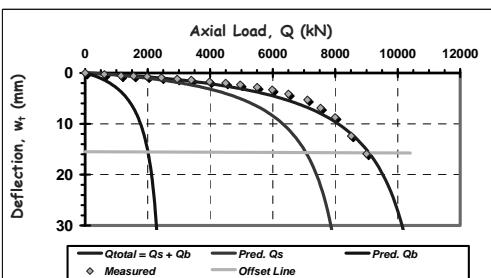
Pile Diameter, d (m) =	14.00	(E_s)L = Modulus pile tip
Pile Length, L (m) =	14.00	(E_b) = Bearing Stratum
Pile Modulus, E_p (kPa) =	3.5E+07	(E_s)mid = Mid-depth Mod.
Poisson Ratio of Soil =	0.2	(E_s)o = Mod. at Surface
Soil Modulus, E_{sL} (kPa)s =	537000	L/d Ratio = 10
Base Modulus, E_b (kPa)s =	1360000	Ratio P_b/P_t = 0.105
Rho = Ratio(E_s)mid/(E_s)L =	1	Zeta Term = 2.935
Shaft Capacity, Q_s (kN) =	8888.0	$K_r = E_p/E_{sL}$ = 65
Base Capacity, Q_b (kN) =	3236.0	Ratio P_s/P_t = 0.895
Total Capacity, Q_t (kN) =	12124.0	

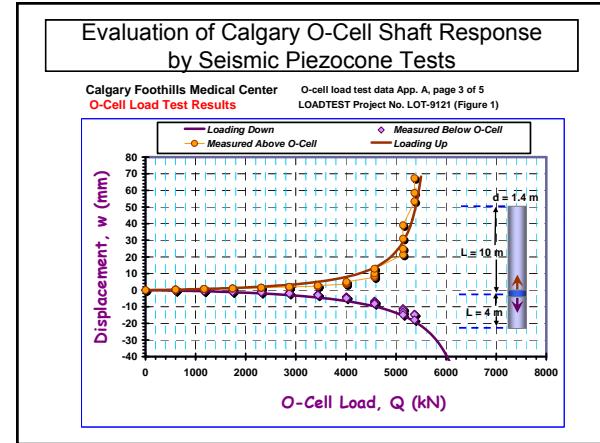
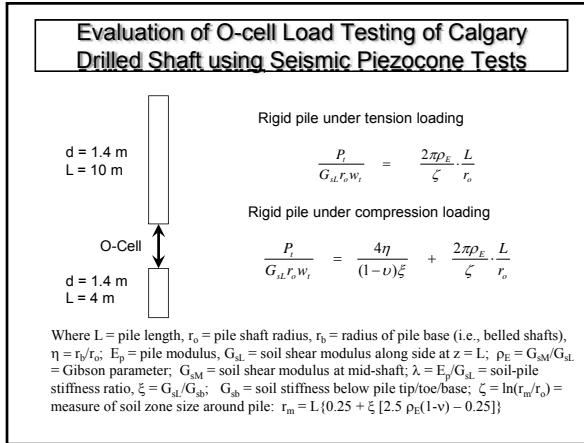
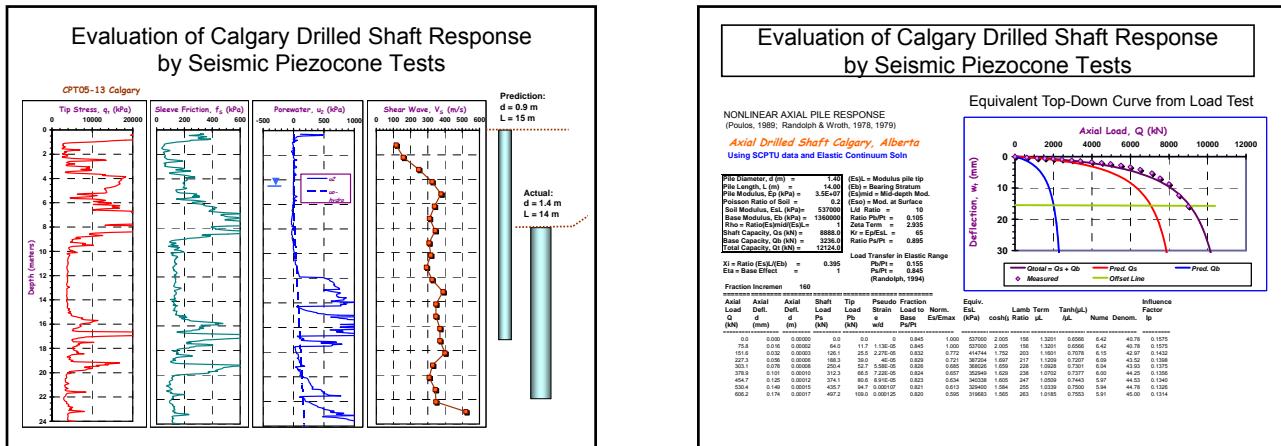
Load Transfer in Elastic Range
 $X_i = \text{Ratio } (E_s)L/(E_b) = 0.395$
 $\text{Eta} = \text{Base Effect} = 1$
 $P_b/P_t = 0.155$
 $P_s/P_t = 0.845$
(Randolph, 1994)

Fraction Increment 160

Axial Load (kN)	Axial Defl. (mm)	Axial Defl. (m)	Shaft Load (kN)	Tip Load (kN)	Pseudo Strain (w/L)	Fraction Load to Norm. (Ps/Pt)	Equiv. EsL (kPa)	Lamb Term cosh(μL)	Tanh(μL)	Influence Factor Ip					
0.0	0.0	0.0	0	0	0	0.000	307000	2.05	198	0.377					
75.8	0.016	0.0002	64.0	11.7	1.13E-05	0.845	1.000	537000	2.005	156	1.3201	0.6566	6.42	40.78	0.1575
151.6	0.032	0.0003	126.1	25.5	2.27E-05	0.832	0.772	414744	1.752	203	1.1601	0.7078	6.15	42.97	0.1432
227.3	0.058	0.0006	188.3	39.0	4E-05	0.829	0.721	387200	1.697	217	1.1209	0.7207	6.09	43.52	0.1398
303.1	0.078	0.0010	250.4	52.7	5.58E-05	0.826	0.685	368026	1.659	228	1.0928	0.7301	6.04	43.93	0.1375
378.9	0.098	0.0015	312.5	66.4	7.13E-05	0.823	0.656	349812	1.621	240	1.0648	0.7401	5.99	44.33	0.1353
454.7	0.125	0.0021	374.1	80.8	8.91E-05	0.823	0.634	340338	1.605	247	1.0359	0.7443	5.97	44.53	0.1340
530.4	0.149	0.0025	435.7	94.7	9.0001E-07	0.821	0.613	329400	1.584	255	1.0330	0.7500	5.94	44.78	0.1326
606.2	0.174	0.0017	497.2	109.0	0.000125	0.820	0.595	319683	1.565	263	1.0185	0.7553	5.91	45.00	0.1314

Equivalent Top-Down Curve from Load Test





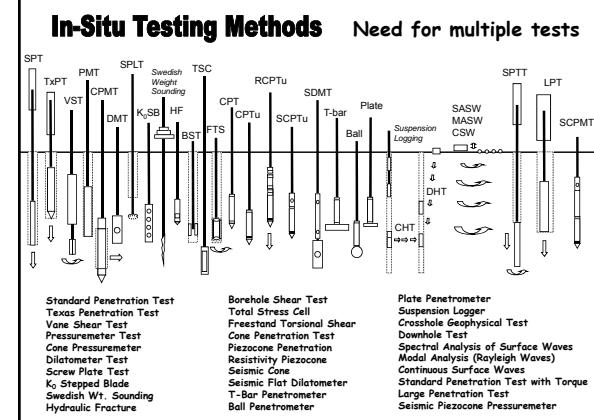
2006 Singapore

Characterization & Engineering Properties of Natural Soils

In-Situ Test Calibrations for Evaluating Soil Parameters

Paul W. Mayne

Georgia Tech

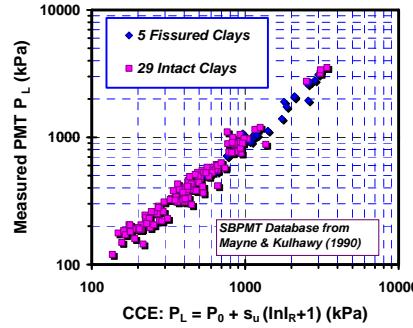


Overview on In-Situ Test Calibrations

- Expand the analytical model for clay (Cavity Expansion + CSSM)
- Forward evaluations of CPTu (3 readings) and DMT (2 readings) in clays
- Include piezo-dissipations
- Global trends in field test data

Georgia Tech

Self-Boring Pressuremeter Data in Clays Cylindrical Cavity Expansion Theory



Cavity Expansion-Critical State Model for CPTu-OCR

Soil Properties:

ϕ' = effective friction angle

C_c = compression index

C_s = swelling index

G = shear modulus

Soil Parameters

$\Lambda = 1 - C_s/C_c$

$M = 6 \sin\phi'/(3-\sin\phi')$

s_u = undrained shear strength

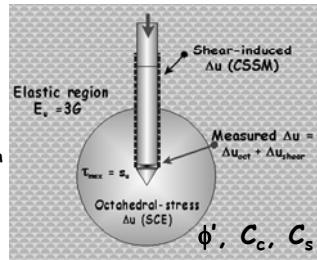
$I_R = G/s_u$ = rigidity index

State:

σ_{vo}' = effective overburden stress

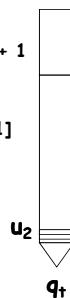
e_0 = initial void ratio

$OCR = \sigma_p'/\sigma_{vo}'$ = overconsolidation ratio



Hybrid SCE-CSSM Piezocene Model for Clays

- Cone Tip Resistance: $q_t - \sigma_{vo} = N_{kt} s_u$
- Vesic (1977) SCE: $N_{kt} = (4/3) [\ln(I_R) + 1] + \pi/2 + 1$
- Wroth (1984) CSSM: $s_u/\sigma_{vo}' = (M/2)(OCR/2)^{\Lambda}$
- $q_t - \sigma_{vo} = (M/2) \sigma_{vo}' (OCR/2)^{\Lambda} [(4/3) (\ln I_R + 1) + \pi/2 + 1]$
- $\Delta u_{meas} = \Delta u_{oct} + \Delta u_{shear}$
- SCE: $\Delta u_{oct} = (4/3) s_u \ln(I_R)$
- CSSM: $\Delta u_{shear} = \sigma_{vo}' [1 - (OCR/2)^{\Lambda}]$
- $\Delta u_2 = (2/3M) \sigma_{vo}' (OCR/2)^{\Lambda} \ln(I_R) + \sigma_{vo}' [1 - (OCR/2)^{\Lambda}]$



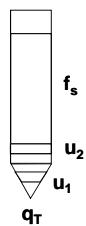
Cavity Expansion-Critical State Piezocone Model

- Intact Clays: $\Delta u_1 = \Delta u_2 + (1/s^*) \cdot M \cdot (OCR/2)^\Lambda \cdot \sigma_{vo}'$
- Stress Path: $s^* = \Delta q / \Delta p' = 3/4$ (midface filter element). Note: see Chen & Mayne (1994) for additional details.
- $f_s = K_0 \cdot \tan \delta \cdot \sigma_{vo}'$
- Interface: $\tan \delta / \tan \phi' = 0.4$
- $K_0 = (1 - \sin \phi')$ $OCR^{\sin \phi'}$

Take representative values:

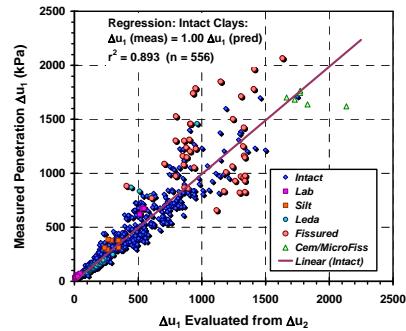
$$M = 0.92$$

$$\Lambda = 0.80$$



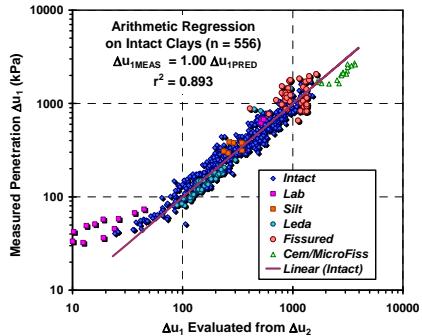
Cavity Expansion-Critical State Piezocone Model

$$\Delta u_1 = \Delta u_2 + (4/3) \cdot M \cdot (OCR/2)^\Lambda \cdot \sigma_{vo}'$$



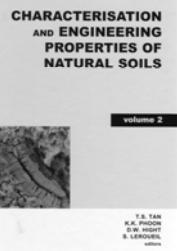
Cavity Expansion-Critical State Piezocone Model

$$\Delta u_1 = \Delta u_2 + (4/3) \cdot M \cdot (OCR/2)^\Lambda \cdot \sigma_{vo}'$$

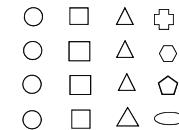


Database Calibrations

□ Connect the Dots



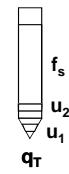
□ Collect the Dots



Sarapuí Soft Clay, Rio de Janeiro, Brasil (Almeida and Marques, 2003)

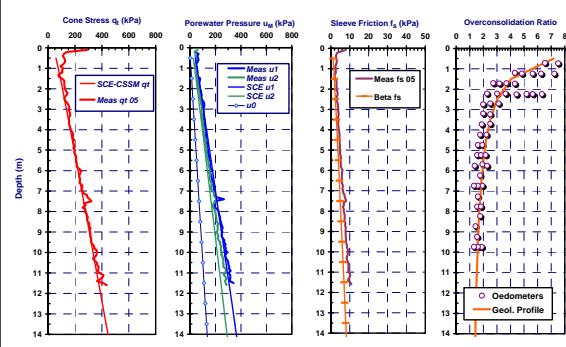
INPUT DATA

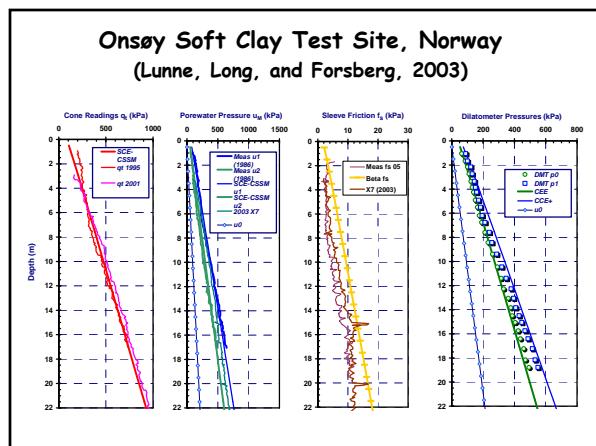
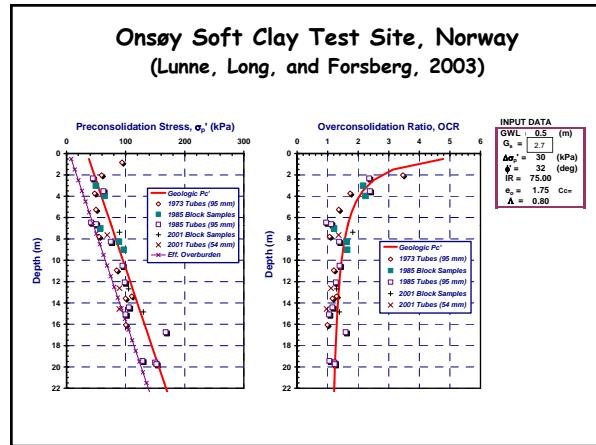
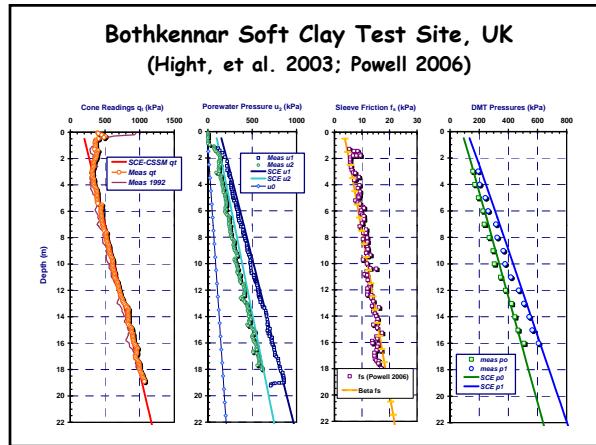
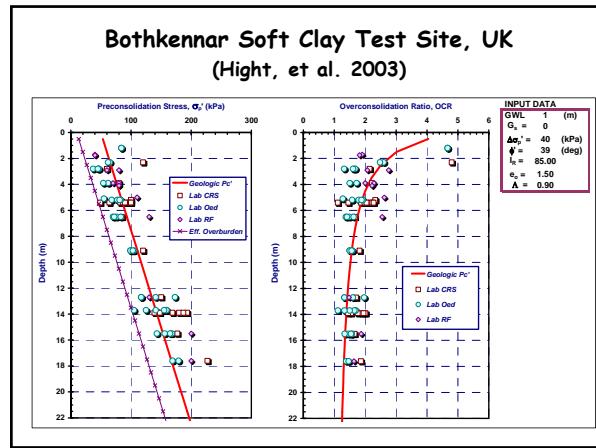
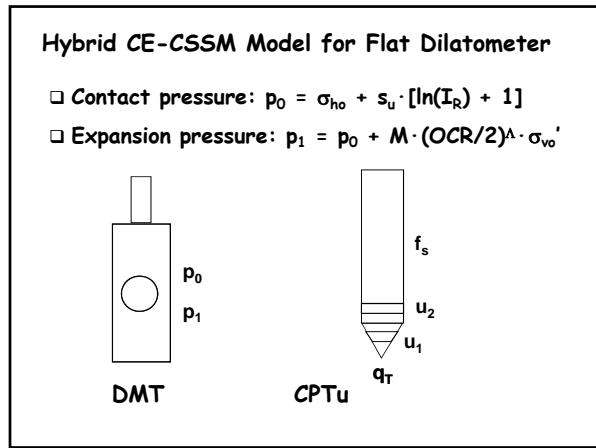
GWL :	0.2 (m)
G_s =	2.7
$\Delta \sigma_v'$ =	25 (kPa)
ϕ' =	29 (deg)
I_R =	50
ϵ_0 =	3.00
Λ =	0.70

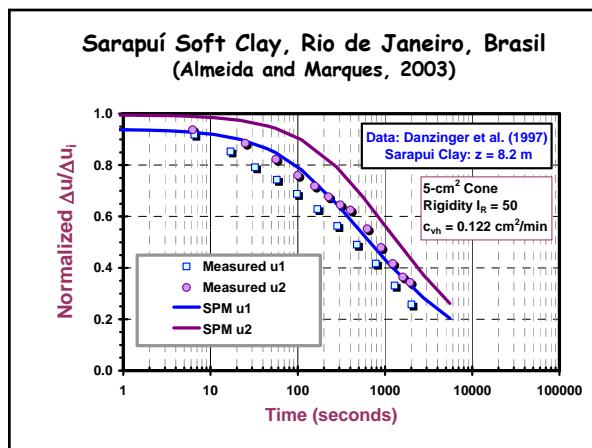
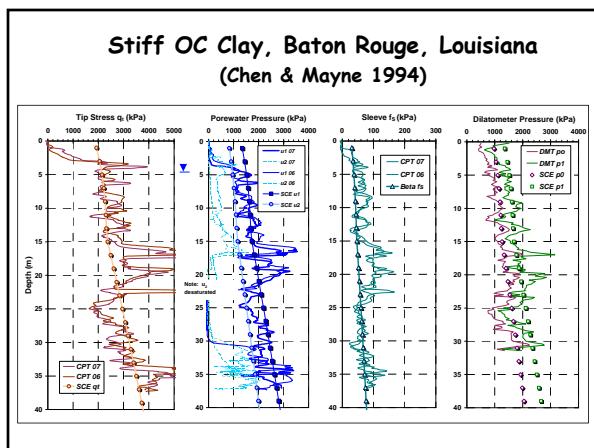
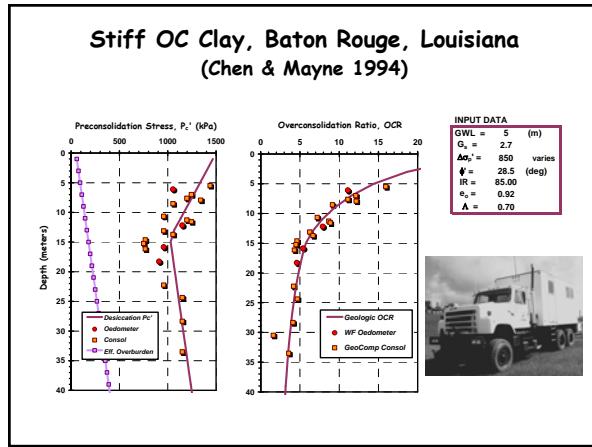
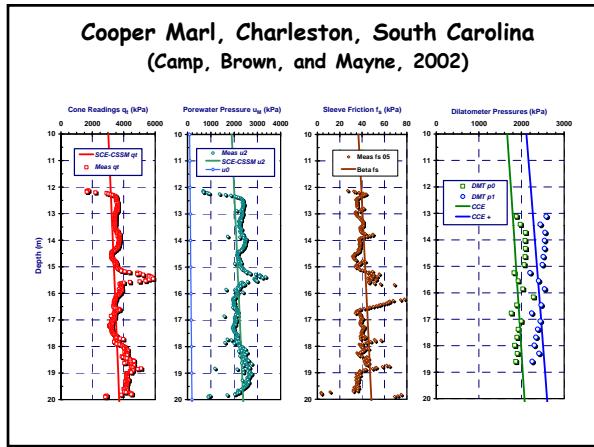
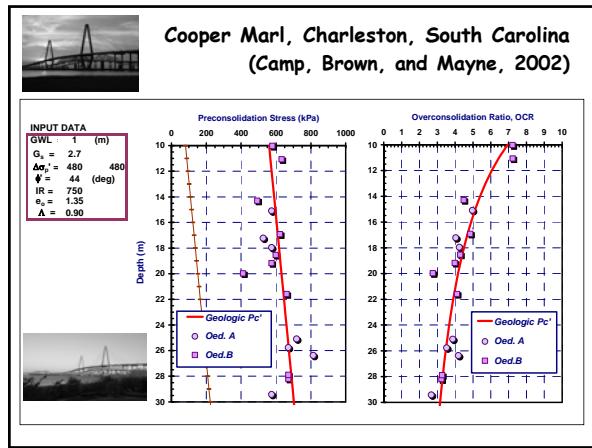
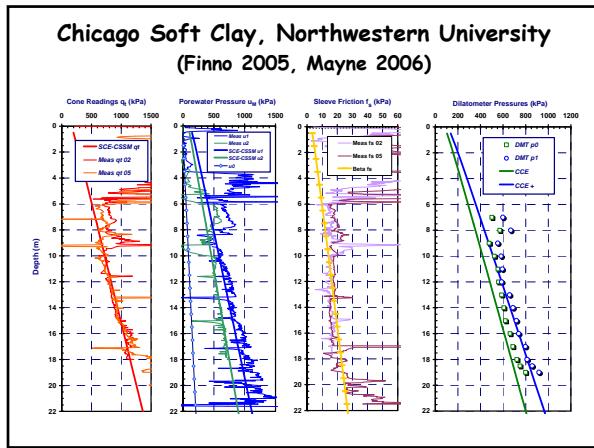


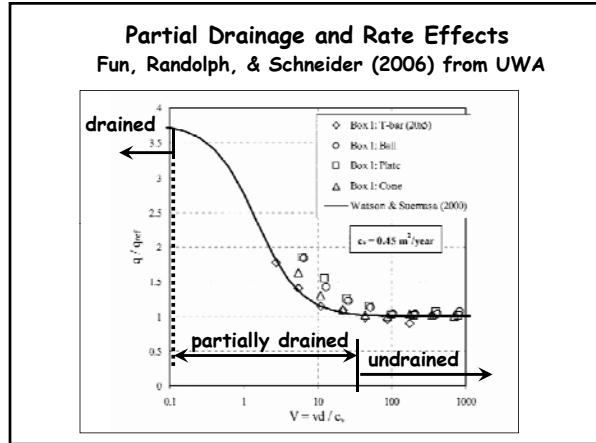
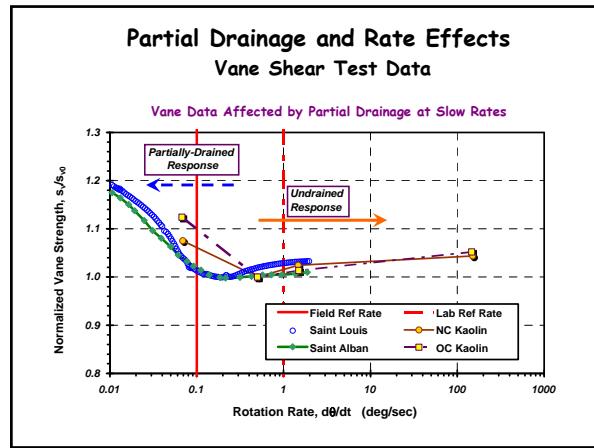
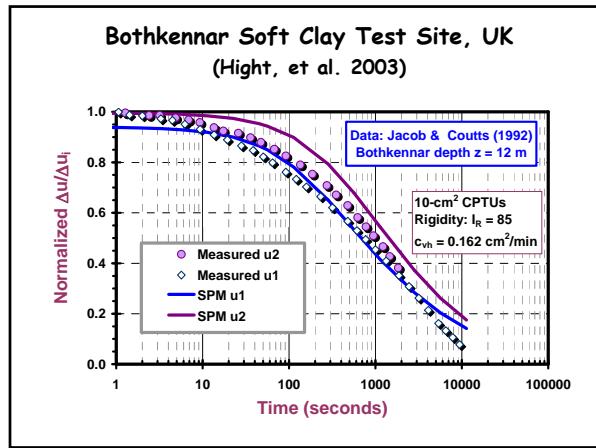
- $q_T = \sigma_{vo}' + (M/2) \sigma_{vo}' (OCR/2)^\Lambda [(4/3)(\ln I_R + 1) + \pi/2 + 1]$
- $\Delta u_2 = (2/3M) \sigma_{vo}' (OCR/2)^\Lambda \ln (I_R) + \sigma_{vo}' [1 - (OCR/2)^\Lambda]$
- $\Delta u_1 = \Delta u_2 + (4/3) \cdot M \cdot (OCR/2)^\Lambda \cdot \sigma_{vo}'$
- $f_s = 0.4 K_0 \sigma_{vo}' \tan \phi'$

Sarapuí Soft Clay, Rio de Janeiro, Brasil (Almeida and Marques, 2003)

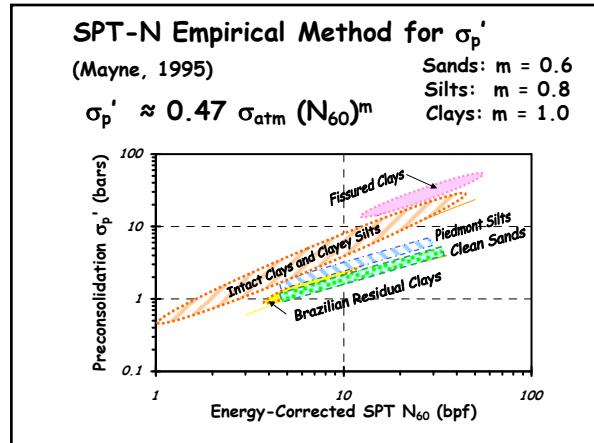


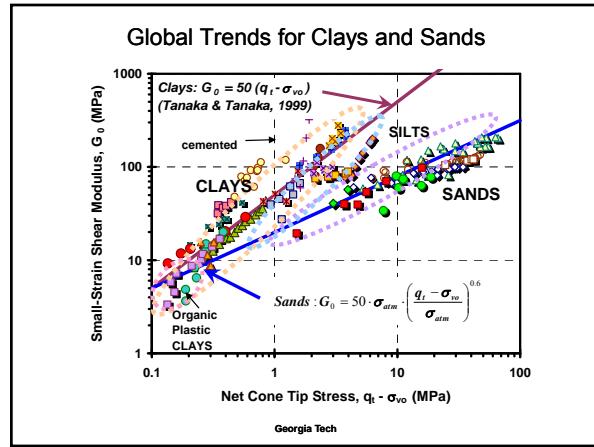
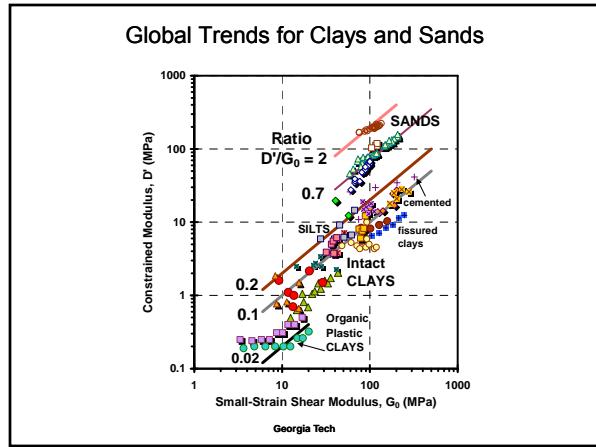
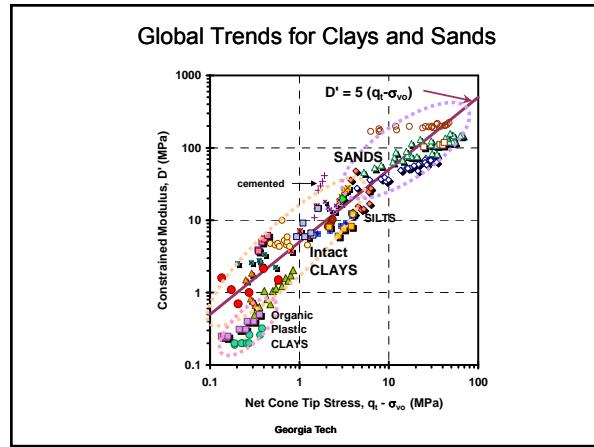
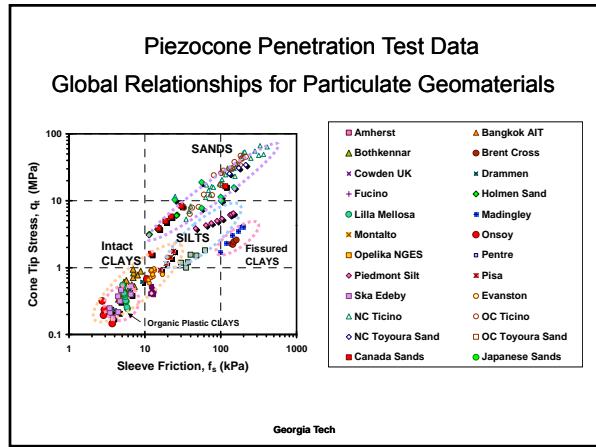
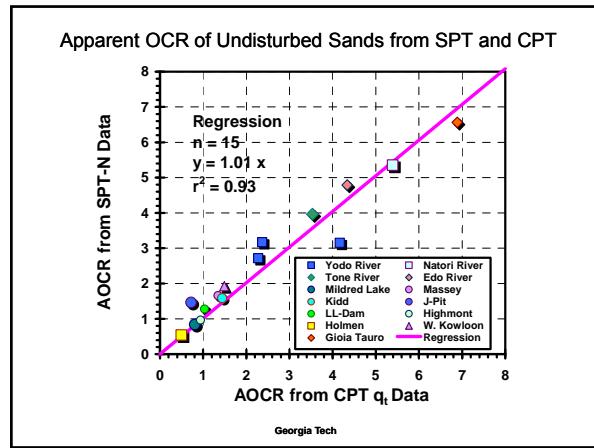
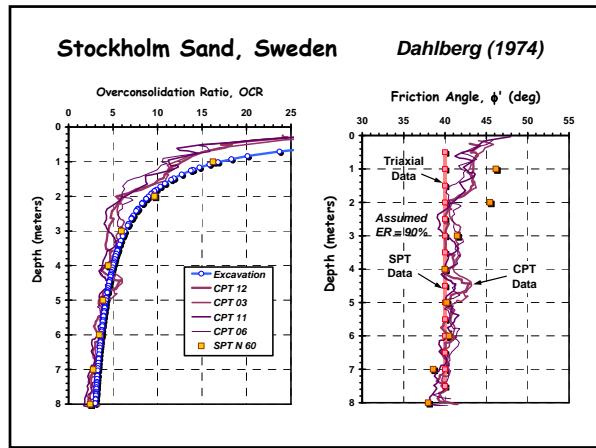


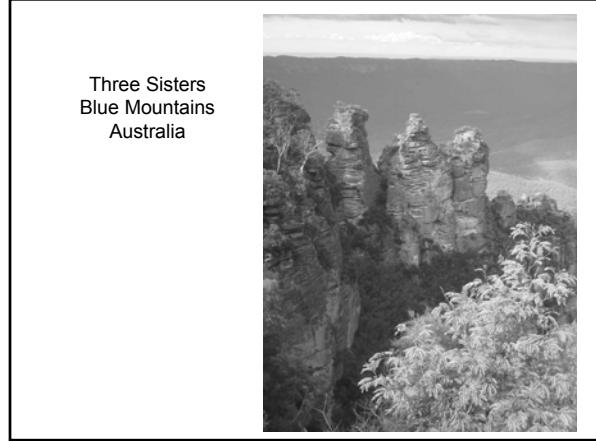
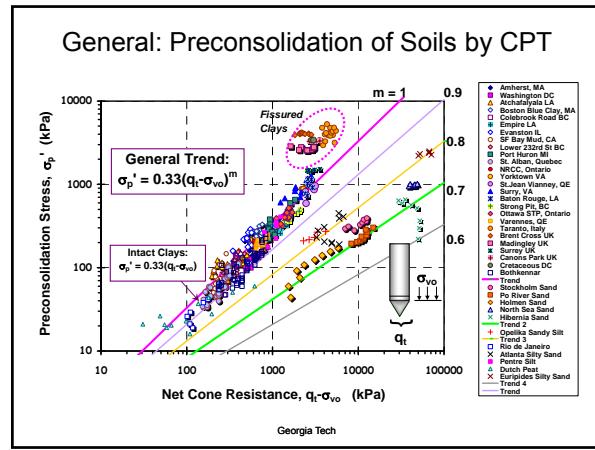
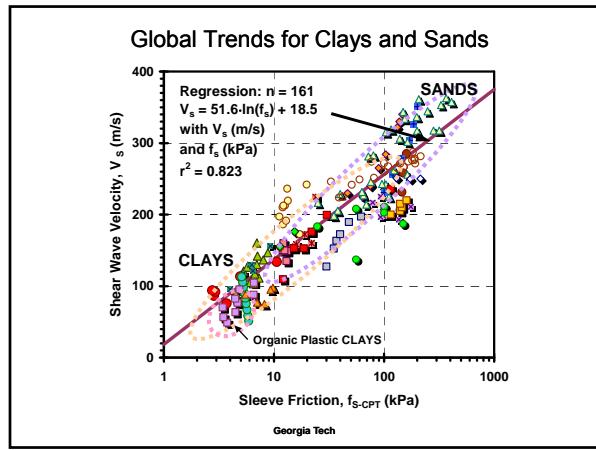




- In-Situ Test Calibrations in Clays**
- Use multiple types of tests (CPT, DMT, CPT_u, PMT)
 - Need consistent framework: SPM (Whittle & Aubeny), Cavity Expansion (Yu), FEM (Houlsby & Teh)
 - Constitutive modeling (MIT-E3, Cam-Clay, NorSand, Bounding Surface) based in critical-state soil mechanics







Cone Penetration Testing Special Applications

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Georgia Institute of Technology
Atlanta, GA 30332-0355



Special Applications of CPT

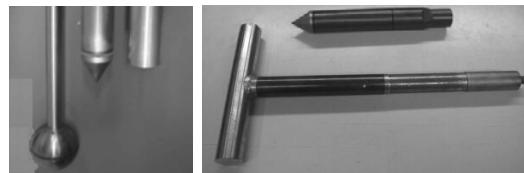
CPT Application	Reference Source
Environmental Site Investigation and Detection of Soil Contamination	<ul style="list-style-type: none"> Campanella & Weemees (1990) Auxt & Wright (1995) Bratton & Timian (1995) Campanella, et al. (1998) Lambson & Jacobs (1995) Lightner & Purdy (1995) Mlynarek, et al. (1995) Plumgraff, et al. (1995) Robertson, Lunne, & Powell (1998) Shinn and Bratton (1995)
Landslide Forensics and Slope Stability	<ul style="list-style-type: none"> Collotta, et al. (1989) Leroueil, et al. (1995) Romani, et al. (1995) Hight and Leroueil (2003)
Pavements Investigations	<ul style="list-style-type: none"> Badu-Tweneboah, et al. (1988) Newcomb & Birgisson (1999)
Sinkhole Detection in Limestone Terrain	<ul style="list-style-type: none"> Foshee & Bixler (1994)

Specialized Types of CPT

Specialized CPT System	Reference	Notes/Remarks
Acoustic Emission CPT	Houlby & Ruck (1998) Menge & Van Impe (1995)	Indicator of soil type Delineate soil type and lenses
AutoSeis Generator	Casey & Mayne (2002)	Portable remote shear wave source
CPT Soil Sampler		Obtains soil sample when needed
Dielectric CPT*	Shinn, et al. (1998)	Maps volumetric water contents
Horizontal CPT	Broere & Van Tol (2001)	Towards tunnel investigations
Lateral Stress Cone	Takesue & Isono (2001) Campanella, et al. (1990)	Measures total horizontal stress Total lateral stress during penetration
Multi-Element Piezocones	Juran & Tumay (1989) Skomedal & Bayne (1988) Danzinger, et al. (1997)	Dual-element piezocene Tripe-element piezocene Quad-element piezocene
Multi-Friction Sleeve Penetrometer	DeJong & Frost (2002) Hebeler, et al. (2004)	Four friction sleeves of different roughness for pile interface studies
Radio-Isotope CPT	Shrivastava & Mimura (1998) Dasari, et al. (2006)	Measures density and water content in real time
T-bar penetrometer	Randolph (2004)	100-cm ² area to increase load cell resolution in soft soils
Vibro-Piezocene	McGillivray, et al. (2000) Bonita, et al. (2000)	Site-specific soil liquefaction Vibration locally causes liquefaction
Vision Cone Penetrometer (VisCPT)	Hryciw, et al. (1998) Hryciw & Shin (2004)	Real-time videocam of soil profile Detection of thin layers & lenses

New Developments: Full-Flow Penetrometers

- T-bar
- Ball penetrometer
- Plate
- Developed for very soft soils (offshore)



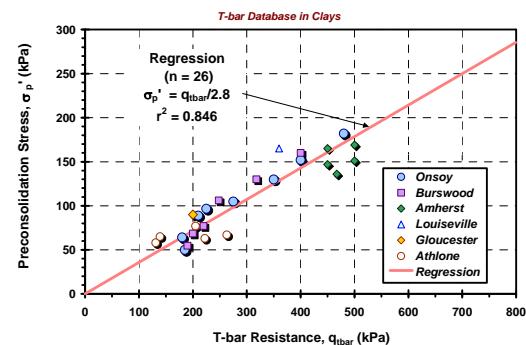
Watson, Newson & Randolph (1988), Proc. ISC-1, Atlanta

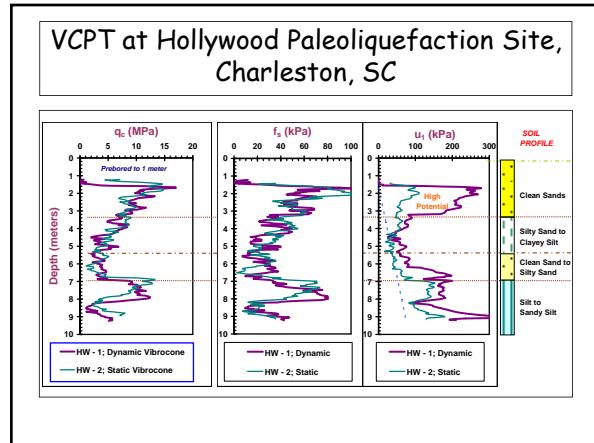
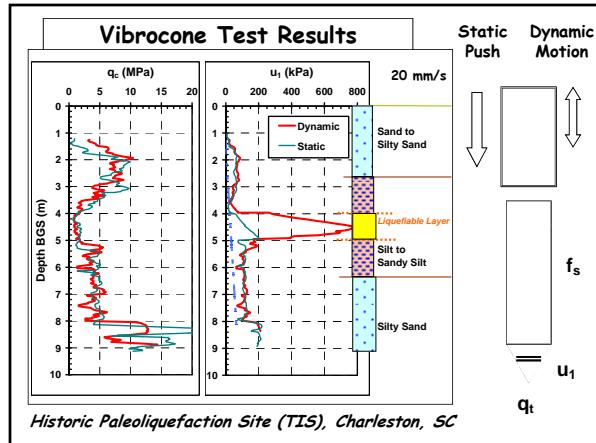
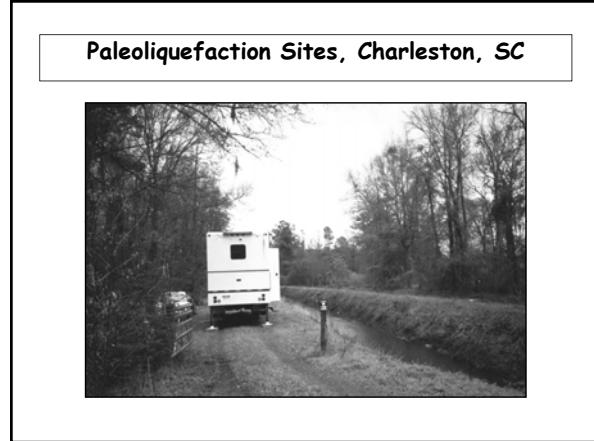
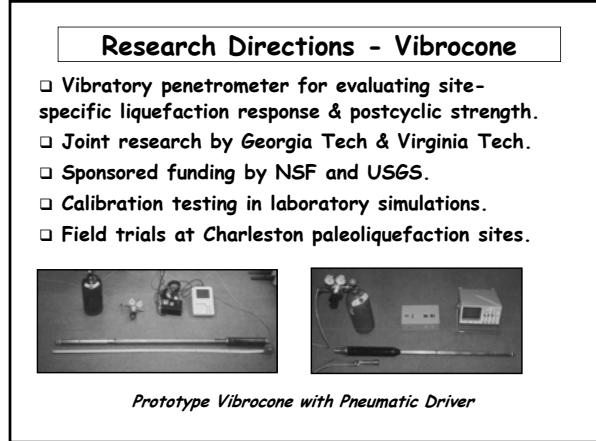
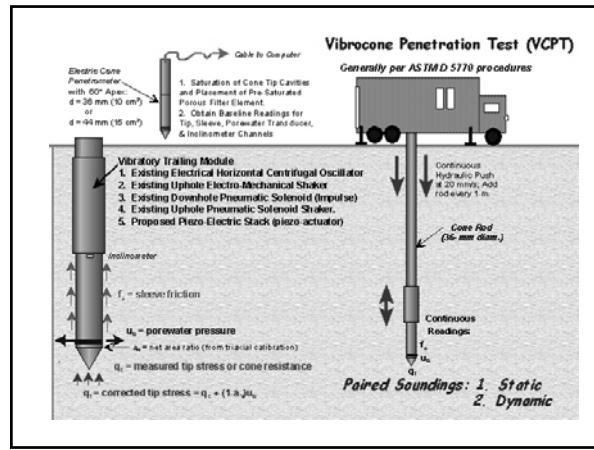
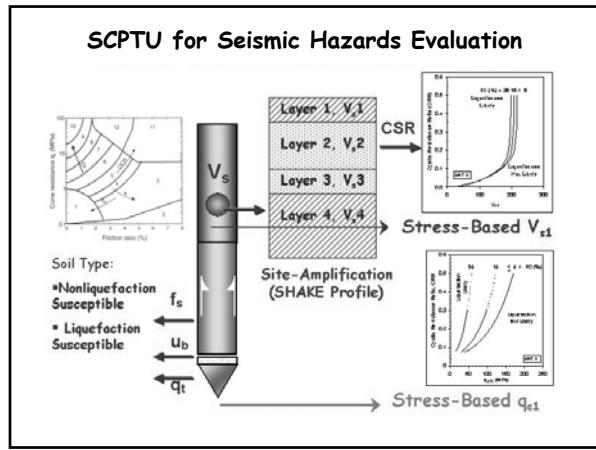
Full-Flow Penetrometers

- Applicable for very soft soils ($s_u < 10 \text{ kPa}$)
- Conventional CPT with larger head
- 100-cm² area (vs. the standard 10-cm²)
- Resolution of load cell improved 10-fold
- Correction for net area minimized
- Direct $q_{\text{T-bar}}$ rather than net (CPT: $q_t - \sigma_v$)
 $s_u = q_{\text{T-bar}}/10$
- Take readings during push and during pullout to investigate cyclic effects and remoulded strength

Ref: Randolph (2004), Proc. ISC-2, Portugal

T-bar evaluation of preconsolidation stress





Research Directions - Vibrocone



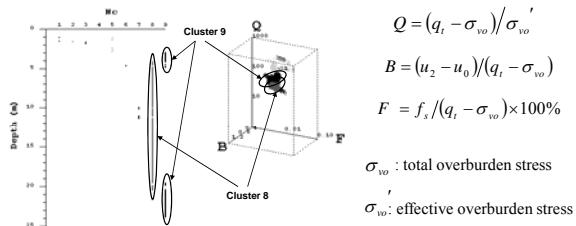
Electro-Vibrocone with Piezo-Actuator Driver and Dual-Element Piezocene Penetrometer

Three-Dimensional Cluster Analysis

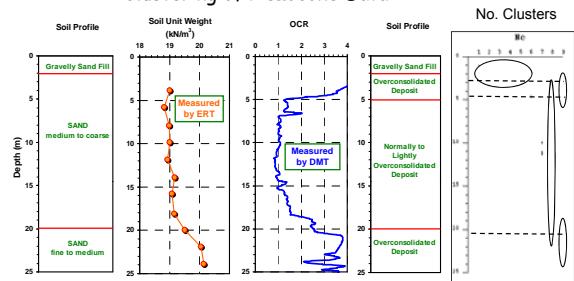
- The three-dimensional cluster analysis is based on the normalized CPT parameters, Q, B, and F, which are derived from tip resistance q_t , pore pressure u_2 , and sleeve friction f_s .
- Similarity between data points are measured by their Euclidean distance in three-dimensional space (Q, B, and F)
- The soils are classified objectively
- By grouping similar data points together, cluster analysis not only finds the apparent changes between soil types, but also subtleties in CPT measurements caused by change of soil properties, that are not readily evident by other available interpretation techniques

(Hegazy, 1998; Liao, 2005)

Clustering of CPTU Data for GeoStratification

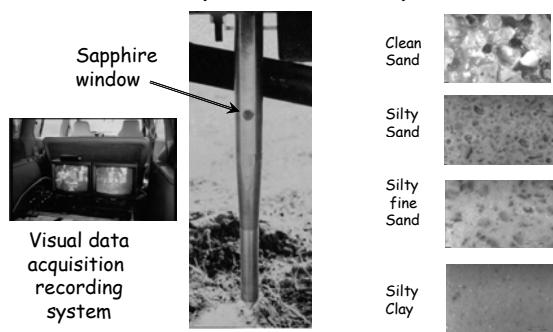


Clustering of Piezocene Data



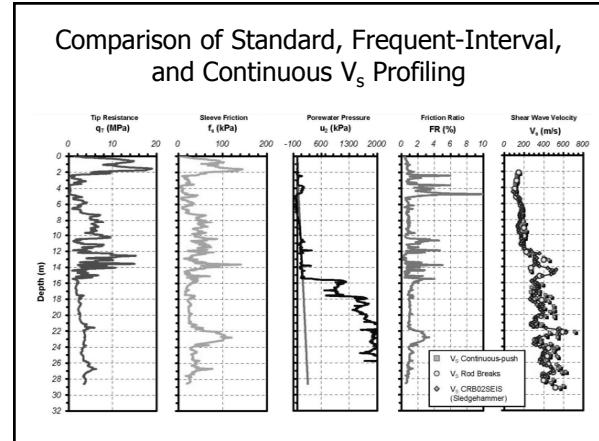
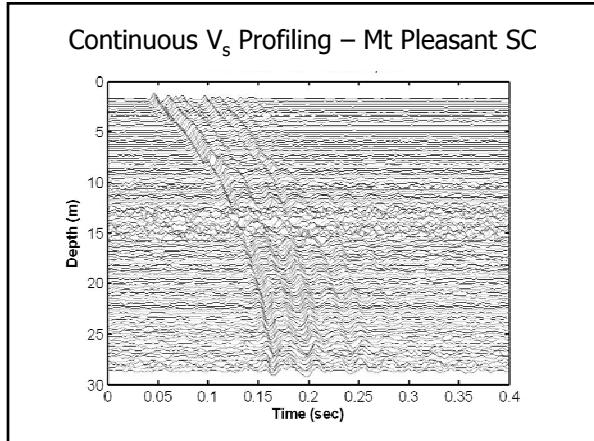
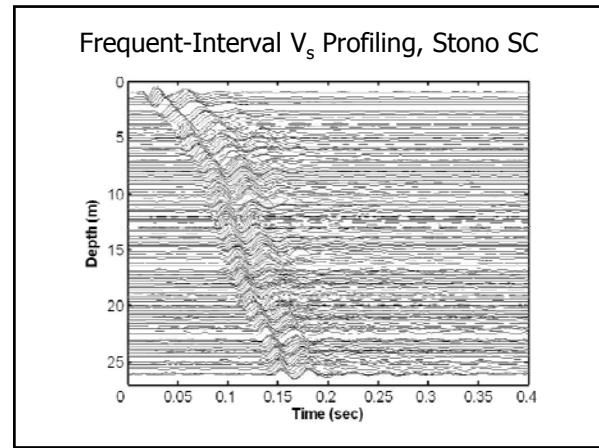
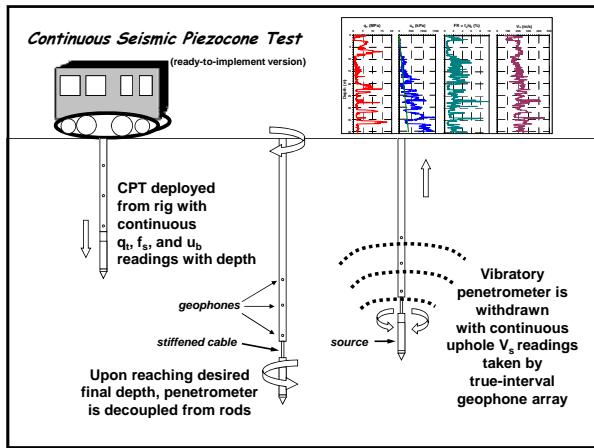
Specialized CPT Developments

VisCPT = vision penetrometer (Hryciw 2004)



Seismic Piezocene Pressuremeter (SCPMTÜ)

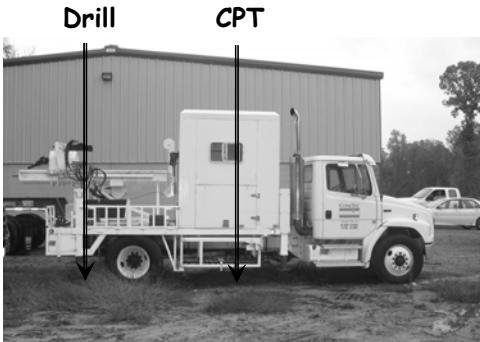




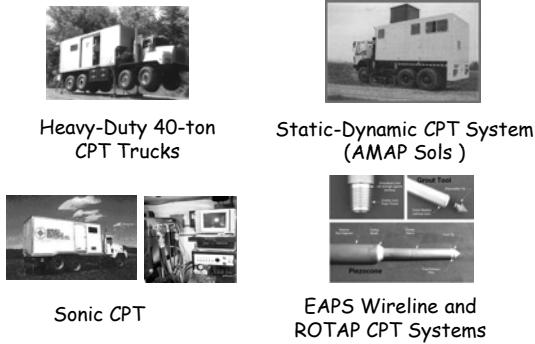
Special Methods for CPT in Hard Ground

Advancing Technique	Reference	Comments/Remarks
Heavy 20-tonne Dead Weight Vehicles	Mayne, et al. (1995)	Increased mass
Friction Reducer	van de Graaf & Schenck (1988)	Opens large diameter hole before rods
Cycling of Rods (up and down)	Shinn (1995, personal comm.)	Locally in thin hard zones of soil
Large 44-mm diameter penetrometer	van de Graaf & Schenck (1988)	Works like friction reducer
Guide Casing: Double set of rods; standard 36-mm rods supported inside larger 44-mm rods; prevents buckling	Peuchen (1988)	Works well in situations involving soft soils with dense soils at depth
Drill Out (Downhole CPTs)	NNI (1996)	Alternate between drilling and pushing
Mud Injection	Van Staveren (1995)	Needs pump system for bentonitic slurry
Earth Anchors	Pagani Geotechnical Equipment Geoprobe Systems	Increases capacity for reaction
Static-Dynamic Penetrometer	Sanglerat et al. (1995)	Switches from static mode to dynamic mode when needed
Downhole Thrust System	Zuidberg (1974)	Single push stroke usually limited to 2 or 3 m
Very Heavy 30- and 40-ton Rigs	Bratton (2000)	After large 20-ton rig arrives at site, added mass for reaction
ROTAP - outer coring bit	Sterckx & Van Calster (1995)	Special drilling capabilities through cemented zones
CPTWD	Sacchetto et al. (2004)	Cone penetration test while drilling
Sonic CPT	Bratton (2000)	Vibrator to facilitate penetration through gravels and hard zones
EAPS (enhanced access penetrometer system)	Farrington (2000); Shinn & Haas (2004); Farrington & Shinn (2006)	Wireline system for enhanced access penetrometer system

Combine Drill-CPT Rig for Hard Ground

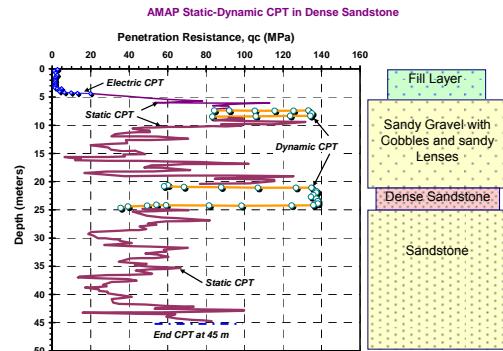


CPTs in Very Hard Ground



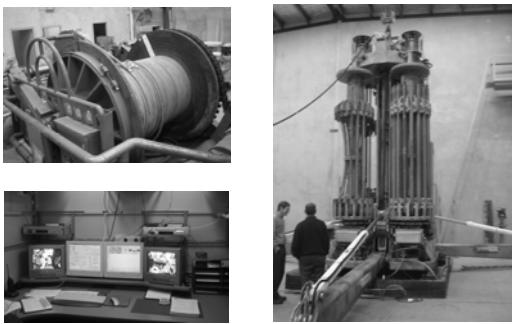
Static-Dynamic CPT System (AMAP Sols)

Sanglerat et al. (1995, 1999, 2004)



PROD = Portable Remotely Operated Drill

Benthic Geotech: <http://www.bgt.com.au/>



Enhanced In-Situ Testing



Seismic Geotechnics



Dodd Farm Paleoliquefaction Site
Steele, Missouri

 Mid-America
Earthquake Center

Paul W. Mayne, PhD, P.E.
Professor, Geosystems Group
Civil & Environmental Engineering
Georgia Institute of Technology
Atlanta, GA 30332-0355



Overview: Seismic Geotechnics

- ❑ Types of Seismic Ground Hazards
- ❑ Evaluate Ground Motions
- ❑ In-Situ Geotechnical Testing
- ❑ Liquefaction Assessment Procedures
- ❑ Shear Wave Velocity Measurements
- ❑ Paleoliquefaction Sites in Mid-America
- ❑ Research Directions
- ❑ Conclusions

Seismic Ground Hazards

- ❑ Bearing Capacity Failure
- ❑ Loss of Life & Injury
- ❑ Settlement & Subsidence
- ❑ Flow Failures (Dams, Reservoirs, Embankments; Flooding)
- ❑ Lateral Spreading (Slopes, Docks, Walls)
- ❑ Sand boils, dikes, sills, fissures, and vents.



High Levels of Ground Shaking

- ❑ Ground Motions
- ❑ Peak Ground Accelerations
- ❑ Site Amplification

Ahmedabad, India
January 2001



Derailed Shinkansen
Japan October 2004

High Levels of Ground Shaking



Mexico City Earthquake, 1985

High Levels of Ground Shaking



Copper River Bridge, Alaska, November 2002

Prerequisites

- Loose saturated granular geomaterials (sands, silty sands, gravel)
- High groundwater table ($z_w < 5$ meters)
- Young Holocene Sediments (< 10,000 yrs).
- Often capped by layer of low-permeability material (clay or silt)
- Generally restricted to upper 15 to 20 meters of overburden.

Soil Liquefaction

Alaska EQ, November 3, 2002



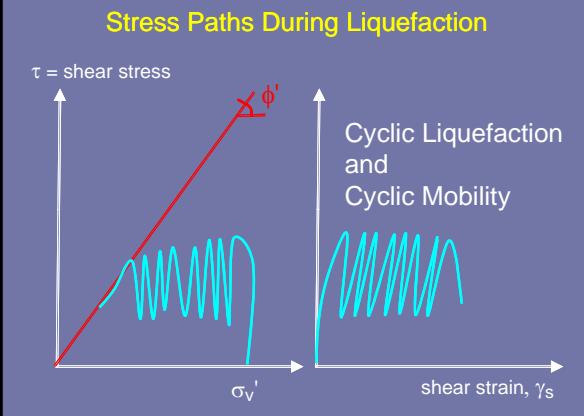
Liquefaction Defined

- **Flow Liquefaction:** First-time loading; Applies to strain-softening soils; Large deformations may occur even after triggering load ceases.
- **Cyclic Mobility:** One-way cyclic loading with softening. Deformations cease when load terminated.
- **Cyclic Liquefaction:** Contractive soils subjected to cyclic stress reversals. Applies to most EQ-related response. Most Common.

Cyclic Liquefaction

- Principle of effective stress: $\sigma_{vo}' = \sigma_{vo} - u$
- When porewater pressure (u) equals total overburden stress (σ_{vo}), effective stress is zero, thus causing liquefaction (Seed & Lee, 1966)
- "Quick Sand" when: $\sigma_{vo} = u_o$
- Cyclic stresses cause accumulation of positive excess porewater pressures ($u = \Delta u + u_o$) in saturated granular soils: $\sigma_{vo}' = \sigma_{vo} - (u_o + \Delta u)$

Stress Paths During Liquefaction



Consequences of Liquefaction

- Liquefaction-Induced Bearing Capacity Failure
- 1999 Izmit EQ ($M = 7.4$), Turkey



Seismic Ground Hazards

- Liquefaction of Foundation Soils
- Loss of Bearing Capacity

Chi-Chi EQ, Taiwan (1999)



Nigata (1964)



Consequences of Liquefaction

- Soil Liquefaction
- Lateral Spreading
- Dam Failure, Kobe, Japan 1995
- Landslide, Niigata 2004

Ground Motions Evaluation

New or Existing Projects

- Probability/Risk Maps (NEHRP) - FEMA International Building Code (IBC 2003)
- Boore & Joyner (1991), Bulletin of the Seismological Society of America
- Coordinates + Amplification Factor
- Site-Specific Evaluation to actual EQ event & epicenter using Code: Shake, Desra, Deepsoil

Ground Motion Attenuation

EPICENTER: Moment Magnitude (M_w) + Hypocentral Distance (d)

Attenuation Relationships

Peak Ground Acceleration (PGA or a_{max})

- Herrmann & Akinci (1999)
- Wen & Wu (1999)
- NEHRP (1997)
- Boore & Joyner (1991)
- SMSIM (Boore, 2002)

Cyclic Stress-Based Procedures

$$CSR = \frac{\tau_{ave}}{\sigma_{vo}} = 0.65 \cdot \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_{vo}}{\sigma_{vo}'} \right) \cdot r_d$$

- Cyclic Stress Ratio (CSR) = level of ground shaking using Seed & Idriss (1971) simplified procedure
- Peak Ground Acceleration, PGA = (a_{max}/g) :
 - General: NEHRP (1997; 2003) risk maps
 - Site-specific analysis (e.g. SHAKE, DEEPSOIL)
- Total and effective overburden stresses: σ_{vo} and σ_{vo}'
- Stress reduction coefficient, r_d

Cyclic Stress Based Procedures

- Level of Ground Shaking:
$$CSR = \frac{\tau_{ave}}{\sigma_{vo}} = 0.65 \cdot \left(\frac{a_{max}}{g} \right) \cdot \left(\frac{\sigma_{vo}}{\sigma_{vo}'} \right) \cdot r_d$$
- Soil resistance based on field performance data from tests:

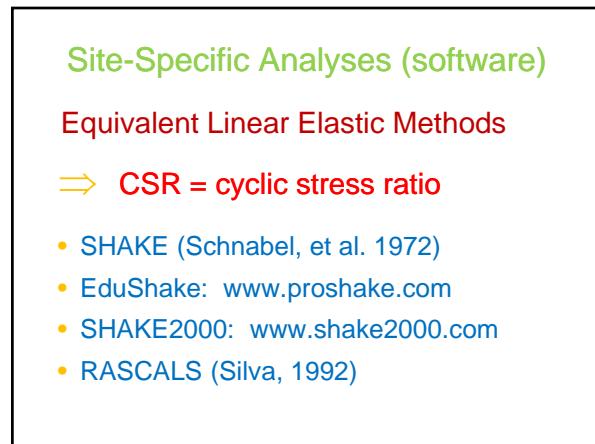
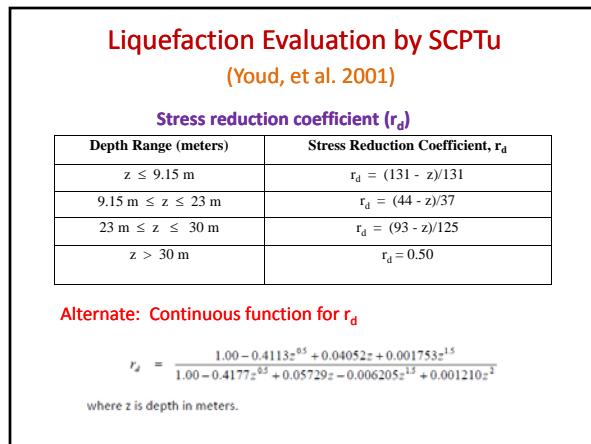
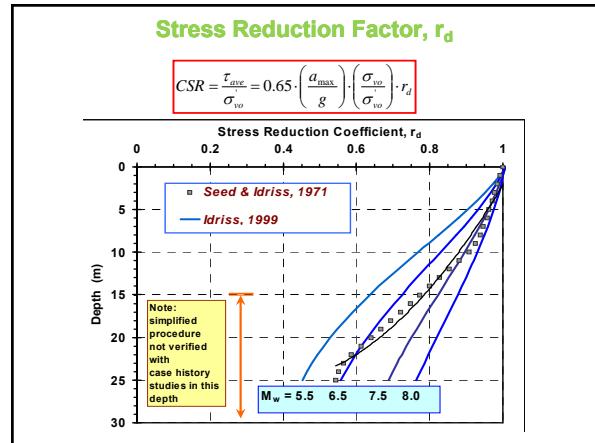
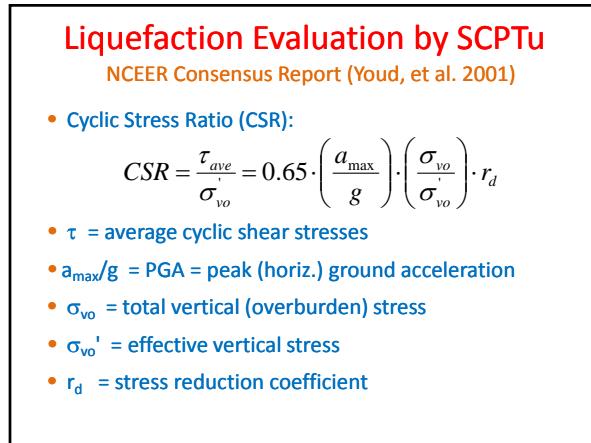
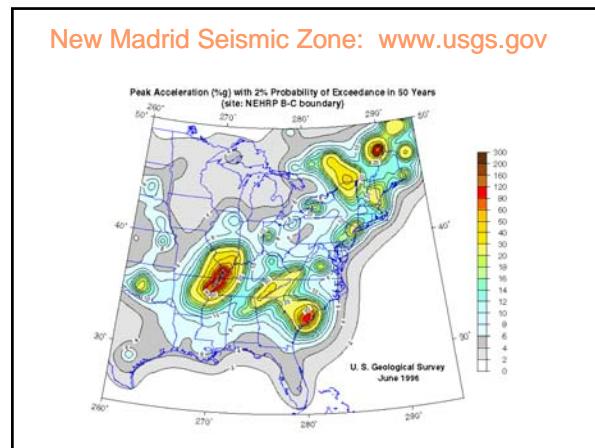
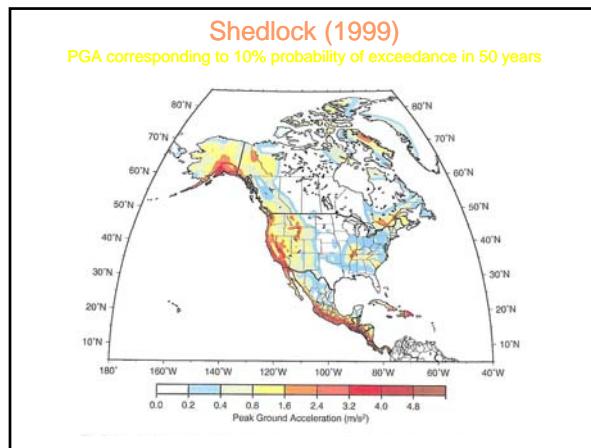
SPT = standard penetration
CPT = cone penetration
 V_s = shear wave velocity
DMT = flat dilatometer

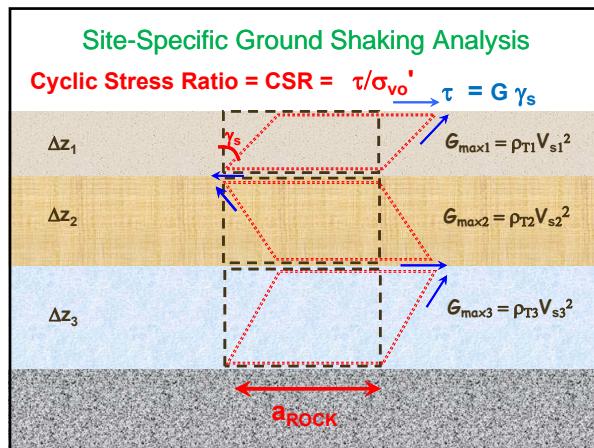
- Compare CSR with Cyclic Resistance Ratio (CRR) that defines likelihood for liquefaction.

Cyclic Stress-Based Procedures

- Reference CRR given for $M_w = 7.5$
- Magnitude Scaling Factor (MSF) for scaling to other EQ sizes (Seed et al., 1985)
- Idriss (1999): $MSF = (M_w/7.5)^{-1.72}$
- Calculate Factor of Safety (FS) against liquefaction (Youd & Noble, 1997):

$$FS = (CRR_{7.5}/CSR) * MSF$$
- Alternative: Weighting Factors applied to CSR



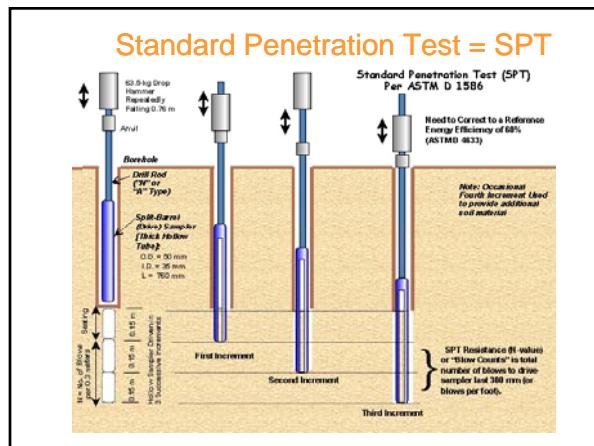


Site-Specific Analyses (Software)

Nonlinear Ground Response Methods

\Rightarrow CSR = cyclic stress ratio

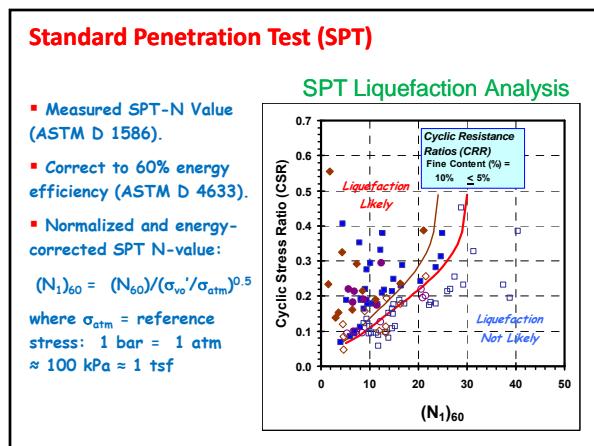
- CHARSOIL (Streeter, et al. 1973)
- MASH (Martin & Seed, 1978)
- DESRA (Lee & Finn, 1978)
- TESS (Pyke 1985)
- DYNA1d (Prevost, 1989)
- SUMDES (Li, et al. 1992)
- D-MOD (Matasovic, 1993)
- DESRAMOD2 (Vucetic, 1998)
- DESRMUSC (Qiu, 1998)
- DEEPSOIL (Park & Hashash, 2004): www.uiuc.edu/~deepsol



Corrections to SPT N-value

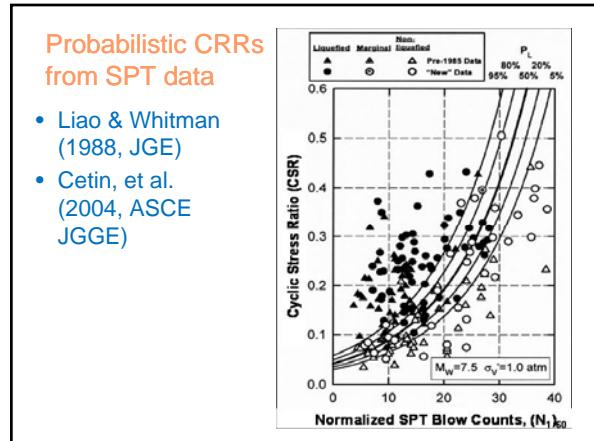
- N_{measured} = Raw SPT Resistance (ASTM D 1586).
- $N_{60} = (ER/60) N_{\text{measured}}$ = Energy-corrected N value where ER = energy ratio (ASTM D 4633). Note: $30\% < ER < 100\%$ with average ER = 60% in the U.S.
- $N_{60} = C_E C_B C_S C_R N_{\text{measured}}$ (additional factors affecting data)
- $(N_1)_{60} = C_N N_{60}$ = Stress-normalized and energy-corrected SPT resistance, as referenced to an effective stress of 1 atm:
$$(N_1)_{60} = (N_{60}) / (\sigma_{vo}' / \sigma_{atm})^{0.5}$$

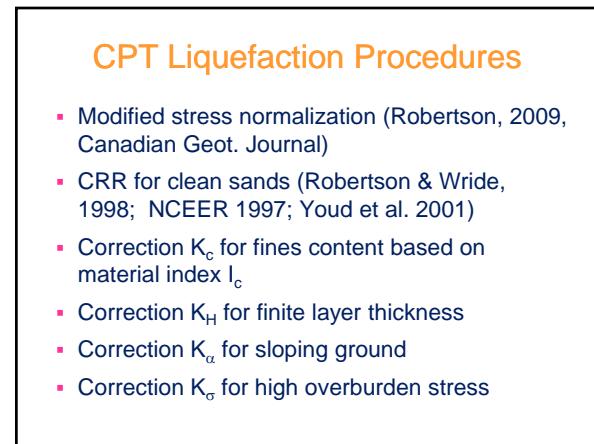
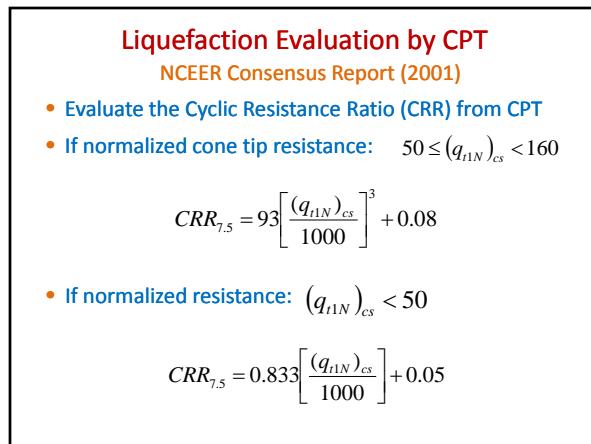
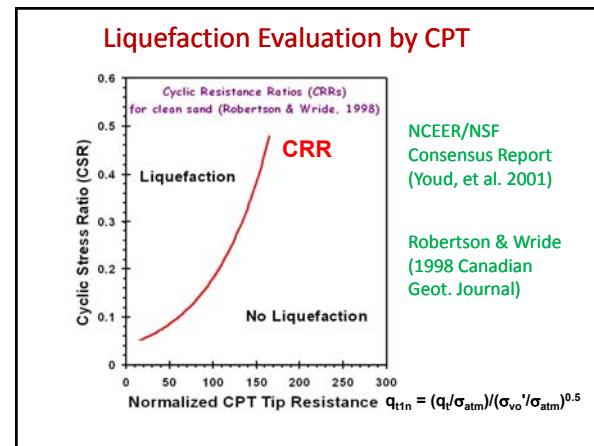
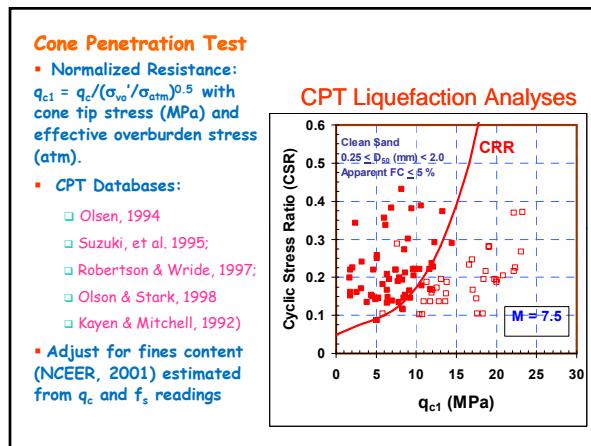
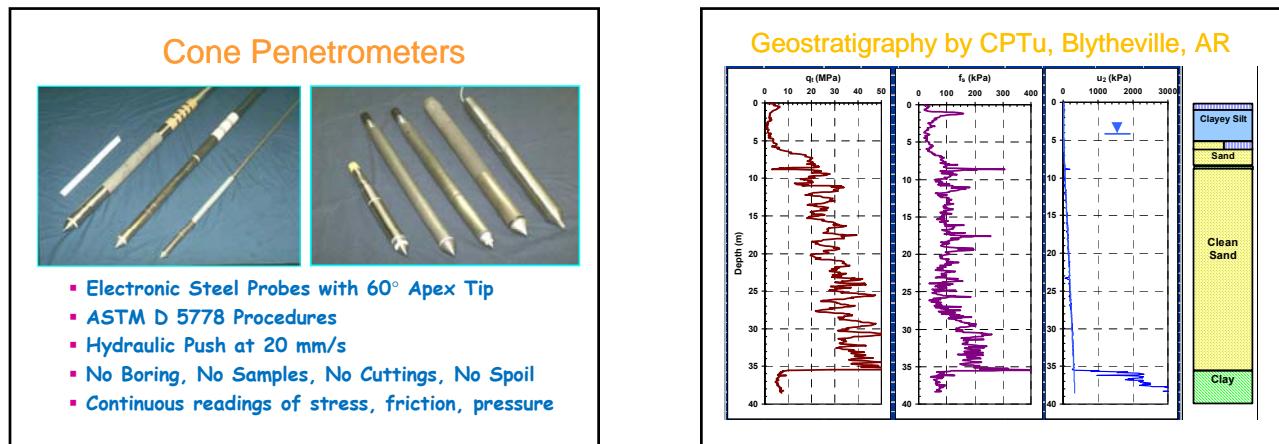
where: $\sigma_{atm} = 1 \text{ bar} \approx 100 \text{ kPa} \approx 1 \text{ tsf}$



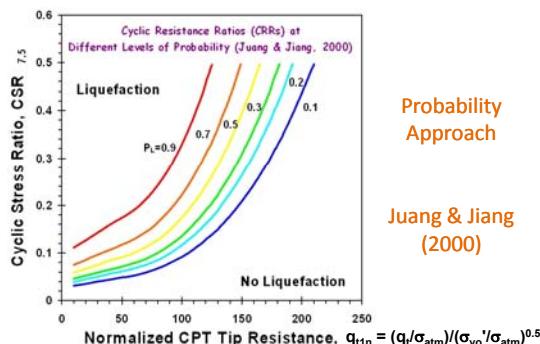
Probabilistic CRRs from SPT data

- Liao & Whitman (1988, JGE)
- Cetin, et al. (2004, ASCE JGGE)





Liquefaction Evaluation by CPT



Overview

- ❑ Seismic Ground Hazards
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- ❑ Conclusions

Shear Wave Velocity, V_s

- ❑ Fundamental measurement of solids in all types of geomaterials (clay, silt, sand, gravel, saprolite, weathered bedrock, intact rock, tailings).
- ❑ Provides small-strain stiffness in terms of shear modulus: $G_0 = G_{dyn} = G_{max} = \rho_T V_s^2$
- ❑ Necessary input for site-specific evaluation of ground amplification in soil column.
- ❑ Indicator of liquefaction susceptibility
- ❑ Can be measured in laboratory on high-quality samples or in field by variety of tests

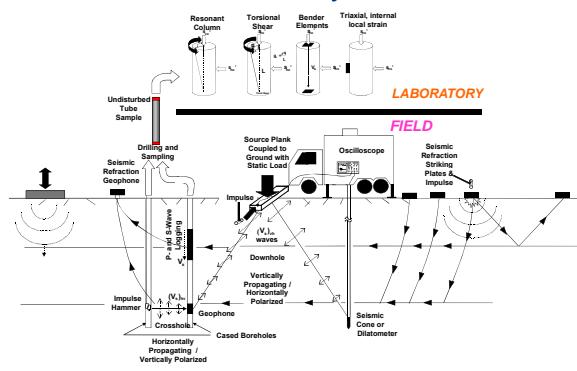
Field V_s Measurements

- ❑ Test Methods:
 - Crosshole Test (CHT)
 - Downhole Test (DHT)
 - Surface Waves (SASW, CSW)
 - Suspension Logger (SL)
 - Seismic Refraction (SR)
 - Reflection/Refraction (RR)
- ❑ Difficulties
 - "Murphy's Law" and weather
 - Cased & grouted boreholes needed for CHT and DHT
 - Non-unique interpretation for non-invasive tests



HP Spectrum Analyzer

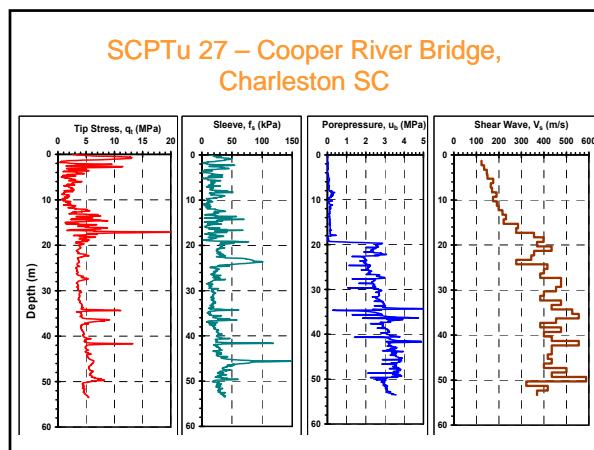
Shear Wave Velocity Measurements



I-17 Bridge - Charleston, South Carolina



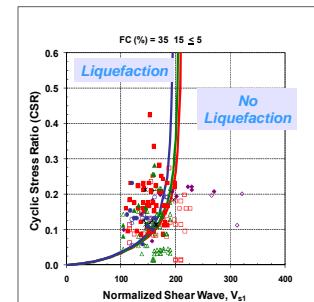
ConeTec Investigations, 25-tonne Cone Truck



V_s Liquefaction Analyses

Shear Wave Velocity

- Field method for evaluating liquefaction potential can be applied to all soil types of concern: sands, silty sands, gravels (NCEER, 1997).
- Resistance Parameter: $V_{s1} = V_s / (\sigma_{vo} / \sigma_{atm})^{0.25}$ where $\sigma_{atm} = 1$ atmosphere.
- Liquefaction database and normalization per Andrus & Stokoe (1997, 2000). Also see Youd et al. (2001, JGGE)



Shear Wave Velocity, V_s

- Site Class (A, B, C, D, E, F) for IBC 2003
- Provides necessary $G_{max} = G_0 = \rho_T V_s^2$ for site-specific site amplification analyses (i.e., SHAKE, DEEPSOIL, RASCALS)
- Normalized V_{s1} for soil resistance in liquefaction analyses

IBC 2008 SITE CLASS

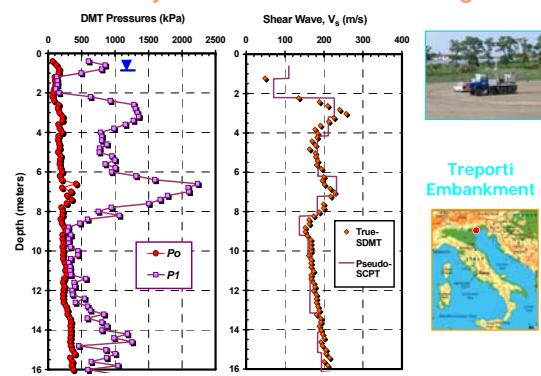
NEHRP = National Earthquake Hazard Reduction Program

Average Properties in Upper 100 feet**				
Site Class	Subsurface Profile	Shear Wave Velocity, V_s (ft/sec)	SPT-N (bpf)	Undrained Shear Strength, s_u (tsf)
A	Hard Rock	$V_s > 5000$	NA	NA
B	Rock	$2500 < V_s < 5000$	NA	NA
C	Dense soil to soft rock	$1200 < V_s < 2500$	$N > 50$	$s_u > 1$
D	Stiff soil	$600 < V_s < 1200$	$15 < N < 50$	$0.5 < s_u < 1$
E	Soft soil	$V_s < 600$ feet/sec	$N < 15$	$s_u < 0.5$ tsf
F	Special* Concerns	Very Low*	Low	Low

True-Interval Seismic Dilatometer (SDMT)



SDMT in Layered Soils of Venetian Lagoon



Liquefaction Evaluation by SCPTu

NCEER Consensus Report (Youd, et al. 2001)

- Using stress-normalized shear wave velocity:

$$V_{sl} = \frac{V_s}{(\sigma_{vo}' / \sigma_{atm})^{0.25}}$$

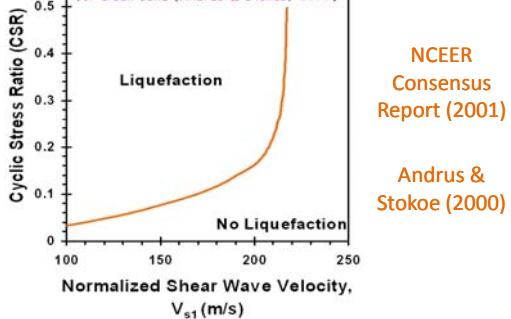
- Evaluate the Cyclic Resistance Ratio (CRR):

$$CRR_{7.5} = 0.03 \cdot (V_{sl}/100)^2 + 0.9 \cdot \left[\frac{1}{V_{slc} - V_s} - \frac{1}{V_{slc}} \right]$$

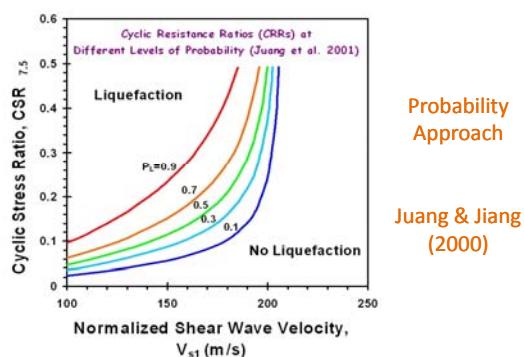
- where V_{slc} is an asymptote related to fines contents (FC): $V_{slc} = 220$ m/s for FC 5%; $V_{slc} = 210$ m/s for FC 20%; and $V_{slc} = 200$ m/s for FC 35%.

Liquefaction Evaluation by Shear Wave

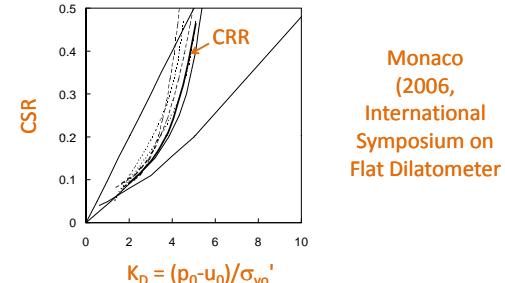
Cyclic Resistance Ratios (CRRs) for clean sand (Andrus & Stokoe, 1997)



Liquefaction Evaluation by SCPTu



Liquefaction Evaluation by DMT

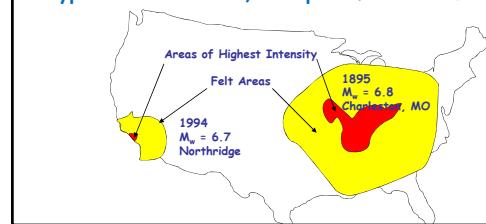


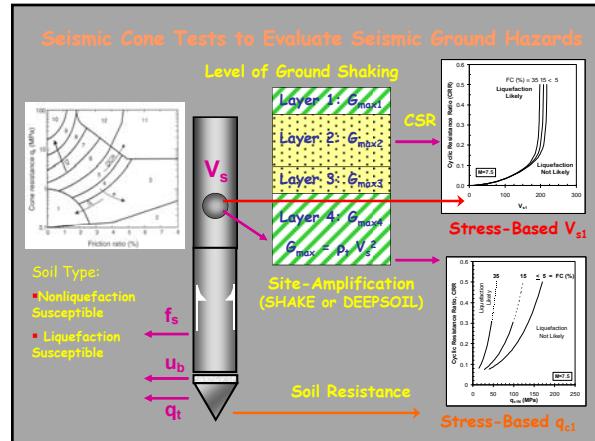
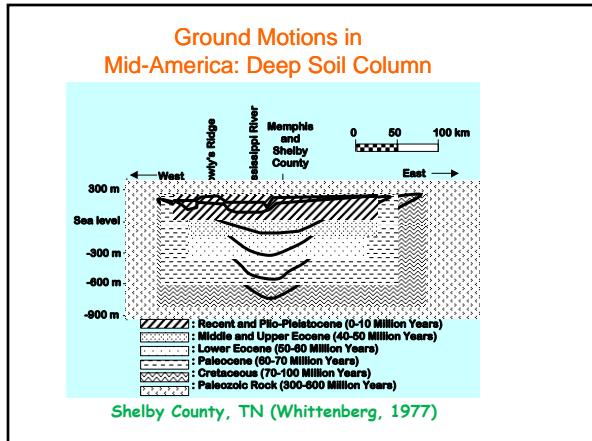
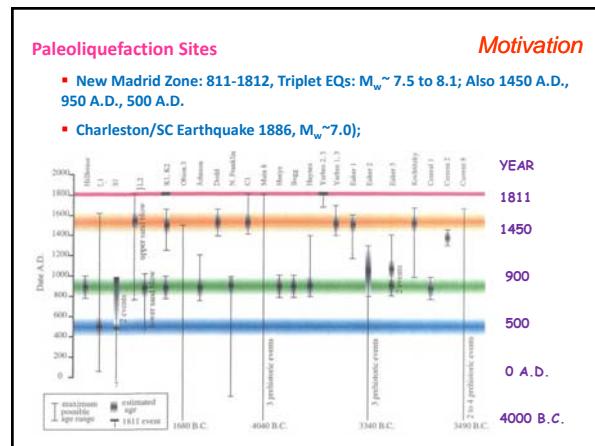
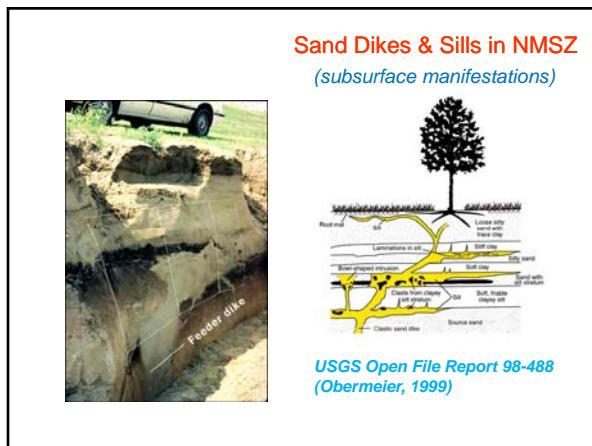
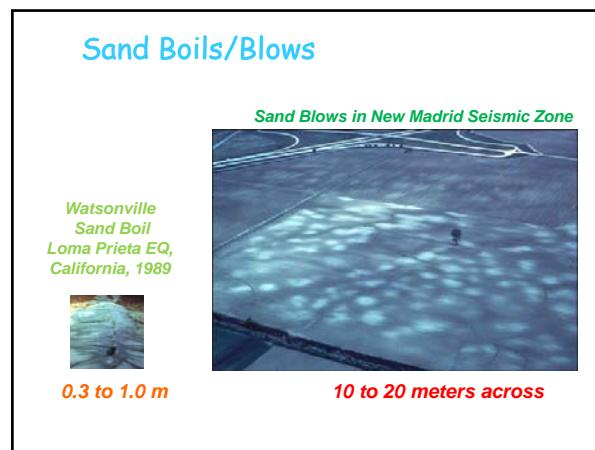
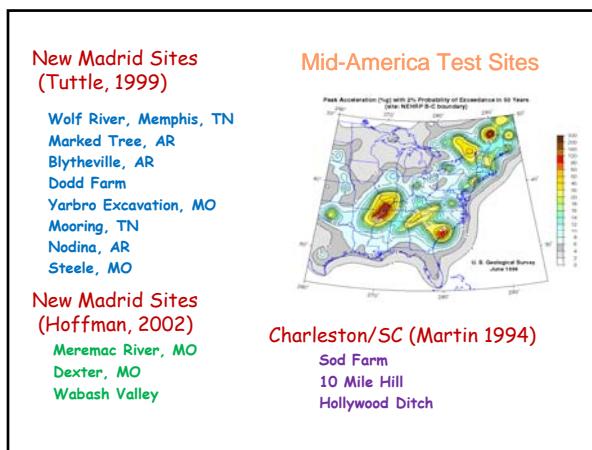
Overview: Seismic Geotechnics

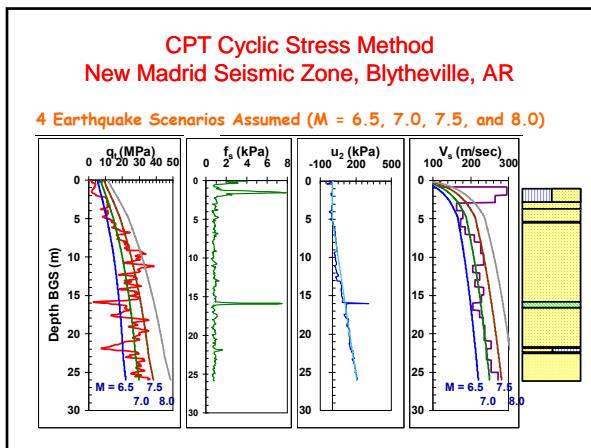
- ❑ Seismic Ground Hazards
- ❑ Evaluate Ground Motions
- ❑ In-Situ Geotechnical Testing
- ❑ Liquefaction Assessment Procedures
- ❑ Shear Wave Velocity Measurements
- ❑ Paleoliquefaction Sites in Mid-America
- ❑ Research Directions
- ❑ Conclusions

Ground Motions in Mid-America

- New Madrid and Charleston/SC Seismic Zones
- Deep soil column overlaying bedrock
- Infrequency of large events to calibrate models
- Soil model a function of moment magnitude, hypocentral distance, & depth of soil column

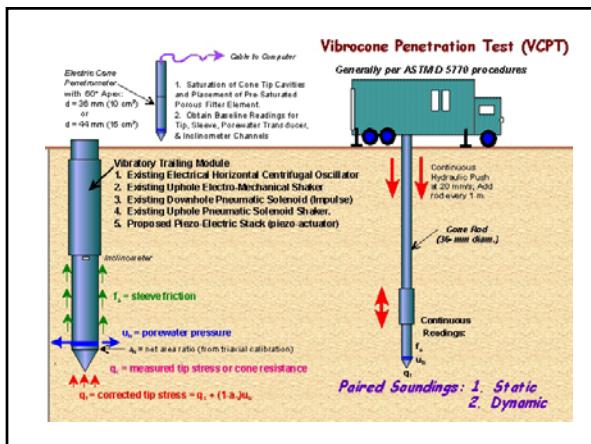




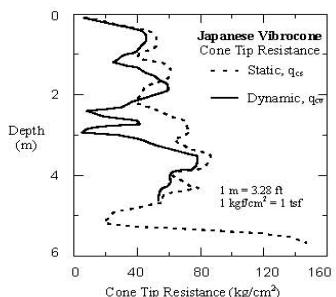


Research Directions

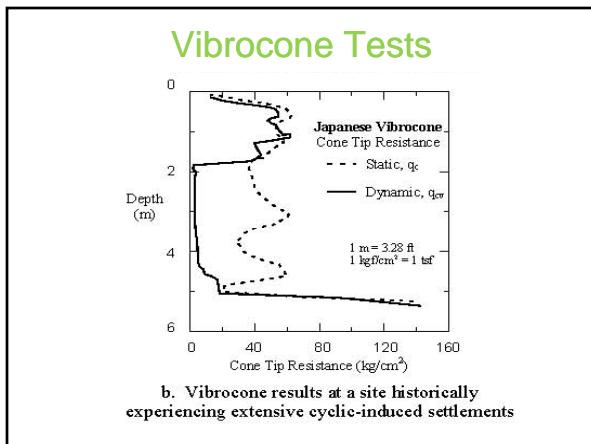
- ❑ Development of special Vibrocone Penetration Tests (VCPT) for localized liquefaction
- ❑ Explosion-induced liquefaction experiments to quantify soil properties and foundation response (ESEE) by USGS and MAE.



Vibrocone Tests



a. Vibrocone results at a seismic site with no apparent settlement during cyclic loading

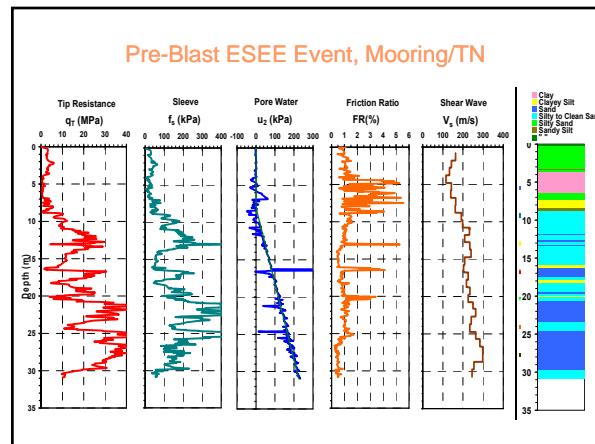
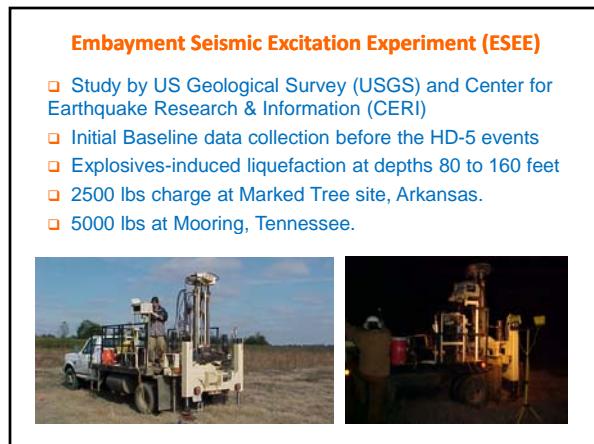
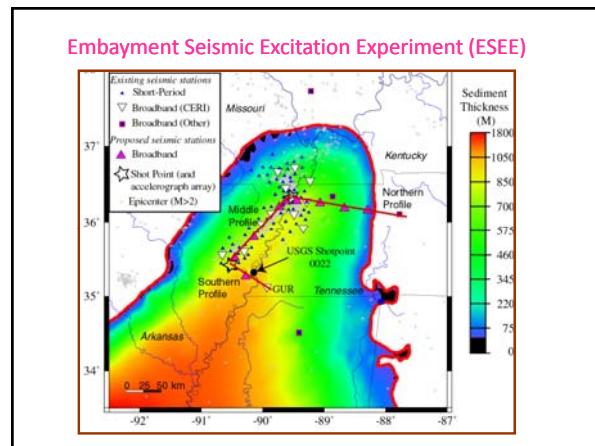
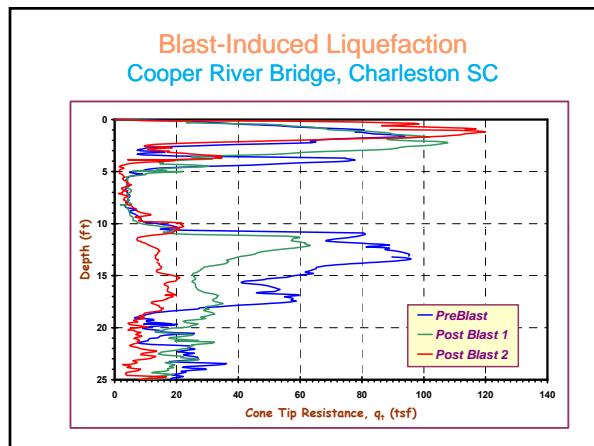
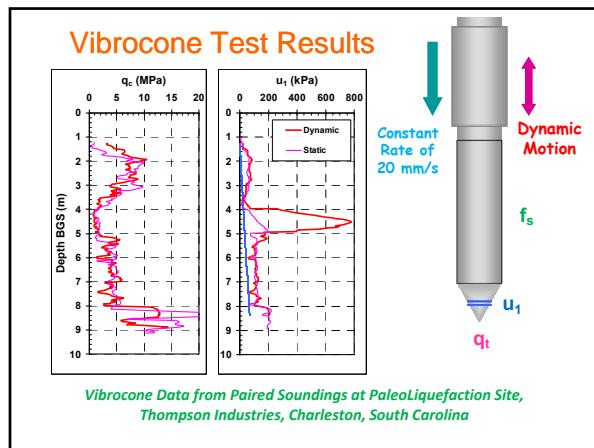


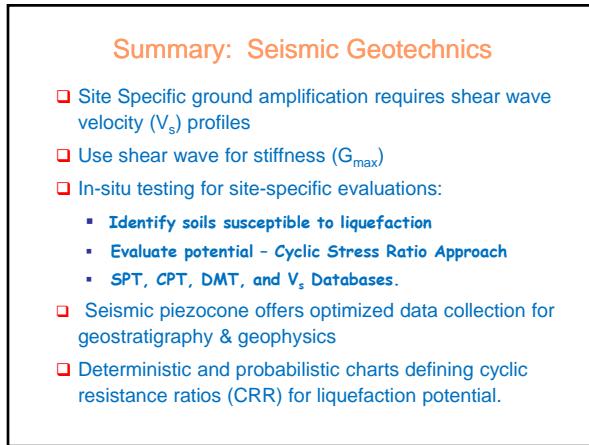
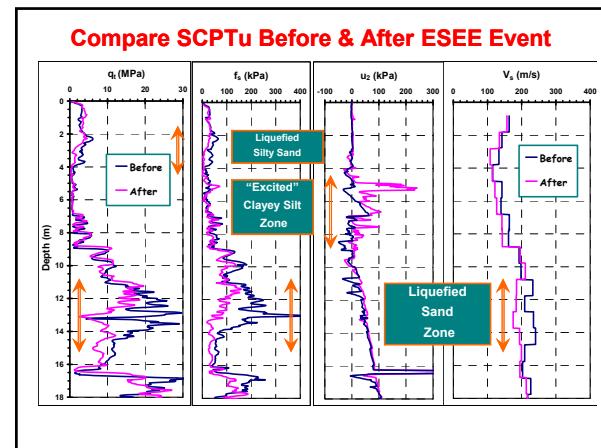
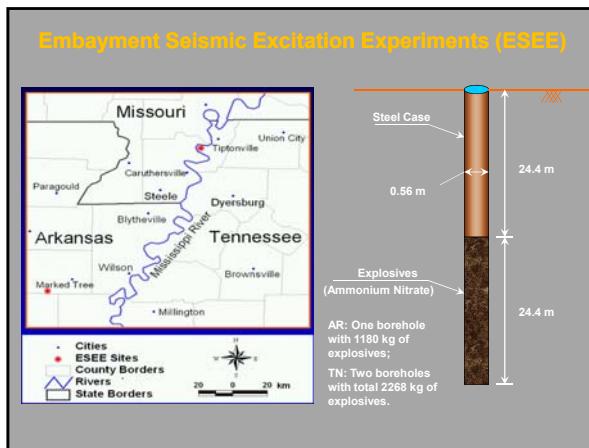
Research Directions - Vibrocone

- ❑ Vibratory penetrometer for evaluating site-specific liquefaction response & postcyclic strength.
- ❑ Joint research by Georgia Tech & Virginia Tech.
- ❑ Sponsored funding by NSF and USGS.
- ❑ Calibration testing in laboratory simulations.
- ❑ Field trials at Charleston paleoliquefaction sites.



Prototype Vibrocone with Pneumatic Driver





Evaluation of Liquefaction Potential and Ground Deformations from Cone Penetration Tests

Abstract

The high ground motions and amplified shaking that accompanies earthquakes can result in soil liquefaction in loose clean to silty sands below the water table. The cone penetration test (CPT) is an excellent means for detecting the presence of sandy soils that are susceptible to liquefaction because it provides continuous logging of strata and layering with three separate readings (q_t , f_s , and u_2). The identification of loose granular soils is accomplished using a CPT material index that determines soil behavioral type. The available soil resistance (i.e., cyclic resistance ratio, CRR) in the ground is evaluated from the stress-normalized cone resistance (Q_{tn}) which is compared with the level of seismic ground motions (i.e., cyclic stress ratio, CSR) to calculate the factor of safety (FS) at this location. The magnitude of ground subsidence and amount of lateral movement can also be estimated from these results.

Methodology

The level of ground shaking is represented by a cyclic stress ratio (CSR), as conventionally estimated from the horizontal peak ground acceleration (PGA), or a_{max}/g (Seed, 1976, 1979; Housner et al. 1985). Proceedings follow the NCEER/NSF workshops on soil liquefaction evaluation (Youd, et al. 2001). The CPT readings are used to evaluate geostratigraphy and soil type from the material index, I_c (Robertson, 2004). Soil unit weight is evaluated from the sleeve friction reading (Mayne, et al. 2010). Soils having a CPT $I_c < 2.6$ are sandy soils that might be prone or susceptible to liquefaction. The available resistance of the ground is quantified using a stress-normalized cone tip resistance (Q_{tn}), as detailed by Robertson and Wride (1998) and Robertson (2009). Liquefaction-induced settlements in level ground are estimated using a method developed by Zhang et al. (2002) while sloped sites and/or free-face conditions are handled by a procedure outlined by Zhang et al. (2004).

Liquefaction Analysis Procedures

Cyclic Stress Ratio

During a particular earthquake event, the local ground motions can be expressed by an equivalent cyclic shear stress ratio ($\tau_{cyclic}/\sigma_{vo}'$), or CSR, using the simplified approach:

$$CSR = \frac{\tau_{cyclic}}{\sigma_{vo}'} = 0.65 \cdot \left(\frac{a_{max}}{g}\right) \cdot \left(\frac{\sigma_{vo}'}{\sigma_{vo}}\right) \cdot r_d \quad (1)$$

where (a_{max}/g) = horizontal peak ground acceleration (PGA) in g's, σ_{vo} = total overburden stress, σ_{vo}' = effective overburden stress, and r_d = stress reduction coefficient given by:

$$r_d = \frac{1.00 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.00 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2} \quad (2)$$

where z is depth in meters.

The value of PGA = (a_{max}/g) can be obtained from either: (a) output from a site-specific soil amplification analyses using shear wave velocity (V_s) profiles and software packages (e.g. SHAKE, DEEPSOIL), or (b) approximate estimates from geologic hazard maps, such as available from the Natural Resources of Canada (NRC) and the United States Geological Survey (USGS):

<http://earthquakescanada.nrcan.gc.ca/>

www.usgs.gov

Figure 1 shows a general seismic map for North America where the a_{max} is in units of m/s^2 and therefore the scaled values must be divided by the gravitational constant ($g = 9.81 m/s^2$) to obtain the normalized value (a_{max}/g) when used in equation (1).

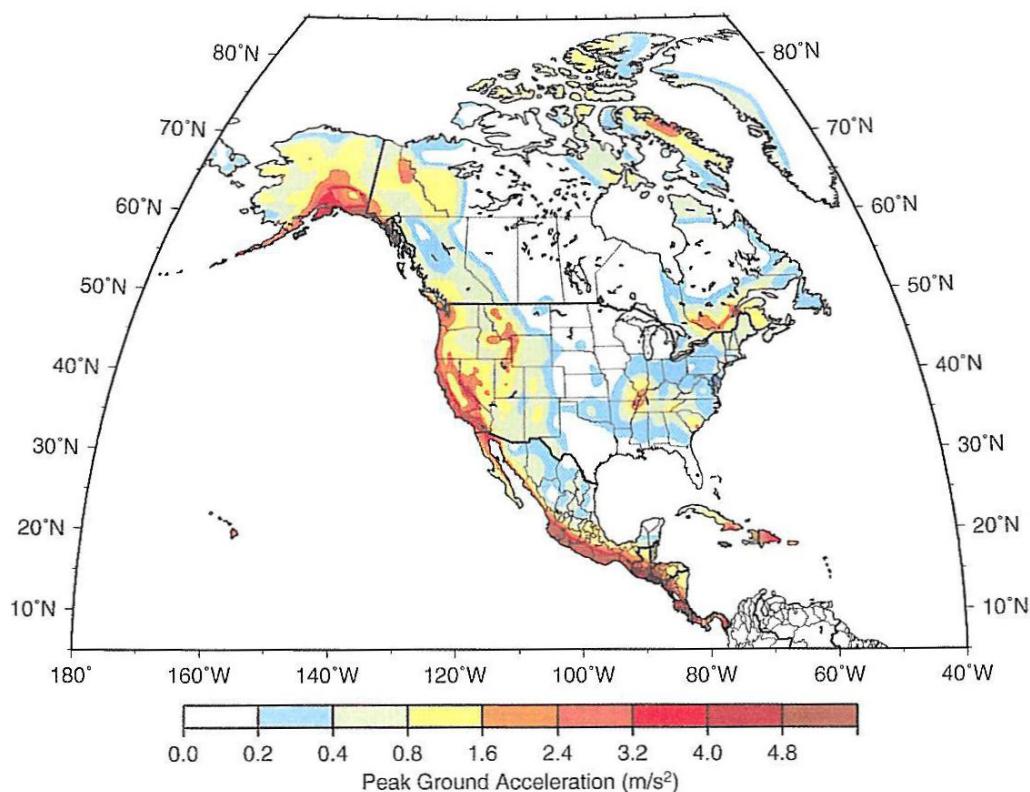


Figure 1. Peak ground accelerations (PGA) for 10% probability in 50 years for North America (Shedlock 1999).

Overburden Stresses

The total vertical overburden stresses are calculated from the accumulation of soil unit weights (γ_t) with depth:

$$\sigma_{vo} = \int_0^z \gamma_t dz \quad (3)$$

The hydrostatic porewater pressure (u_0) is obtained from the water table information. If the depth to the groundwater table is designated z_w , then the value of u_0 below this depth ($z > z_w$) is given by:

$$\text{Below water table:} \quad u_0 = (z - z_w) \cdot \gamma_w \quad (4a)$$

where γ_w = unit weight of water (= 9.8 kN/m³ for freshwater). Above the water table ($z < z_w$), the two simplest scenarios include:

$$\text{Dry soil above water table (no capillarity):} \quad u_0 = 0 \quad (4b)$$

$$\text{Saturated soil above water table (full capillarity):} \quad u_0 = (z - z_w) \cdot \gamma_w \quad (4c)$$

The normal assumption is that clean sands are dry above the water table, while clays can be saturated (or dry) above the water table. In the case of partially-saturated soils, the conditions could vary anywhere in-between, depending on many factors, including: the degree of saturation, recent rainfall or drought activity, humidity, temperature, and other variables.

At any particular depth, the effective vertical overburden stress is obtained as the difference between the total overburden stress and the hydrostatic porewater pressure:

$$\sigma_{vo}' = \sigma_{vo} - u_0 \quad (5)$$

The total unit weight of soil layers may be estimated using the CPT sleeve friction (Mayne, et al. 2010):

$$\gamma_t = 1.95 \cdot \gamma_w \cdot \left(\frac{f_s}{\sigma_{atm}} \right)^{0.06} \left(\frac{\sigma_{vo}'}{\sigma_{atm}} \right)^{0.06} \quad (6)$$

where σ_{atm} = 1 bar = 100 kPa ≈ 1 tsf = reference stress equal to one atmosphere. Since σ_{vo}' appears within the expression, either an iterative approach can be used for solving (6), else a one-lag calculation on the associated γ_t and σ_{vo}' can be adopted using an assumed unit weight for the first soil layer.

Soil Classification by CPT

The identification of soil type may be made on the basis of a CPT material index (I_c) that is defined by (Robertson & Wride, 1998):

$$I_c = \sqrt{\{3.47 - \log(Q_{tn})\}^2 + \{1.22 + \log(F)\}^2} \quad (7)$$

where the normalized cone resistance (Q_{tn}) and normalized sleeve friction (F_r) are given by:

$$Q_{tn} = \frac{(q_t - \sigma_{vo})}{\sigma_{atm}} \cdot \left(\frac{\sigma_{atm}}{\sigma_{vo}} \right)^n = \frac{(q_t - \sigma_{vo})}{(\sigma_{atm} \cdot \sigma_{vo})^n} \quad (8)$$

$$F_r = \frac{f_s}{q_t - \sigma_{vo}} \cdot 100\% \quad (9)$$

where n = exponent dependent upon soil type. The soil behavioral type and zone number are obtained from Table 1 and depicted graphically in Figure 2.

Table 1. Soil behavioral type and zone number as defined by CPTu material index, I_c

Soil Classification	SBT Zone Number	Range CPT Material Index I_c
Stiff clays and sands	8 and 9	(see note a)
Sands with gravels	7	$I_c < 1.31$
Sands: clean to silty	6	$1.31 < I_c < 2.05$
Sandy mixtures	5	$2.05 < I_c < 2.60$
Silty mixtures	4	$2.60 < I_c < 2.95$
Clays	3	$2.95 < I_c < 3.60$
Organic soils	2	$I_c > 3.60$
Sensitive soils	1	(see note b)

Notes:

- a. Zones 8 and 9 found when $1.4\% < F_r < 10\%$ and the following criterion:

$$Q_{tn} \geq \frac{1}{+0.006(F_r - 0.9) - 0.0004(F_r - 0.9)^2 - 0.002}$$

- b. Sensitive soils of Zone 1 identified by when:

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

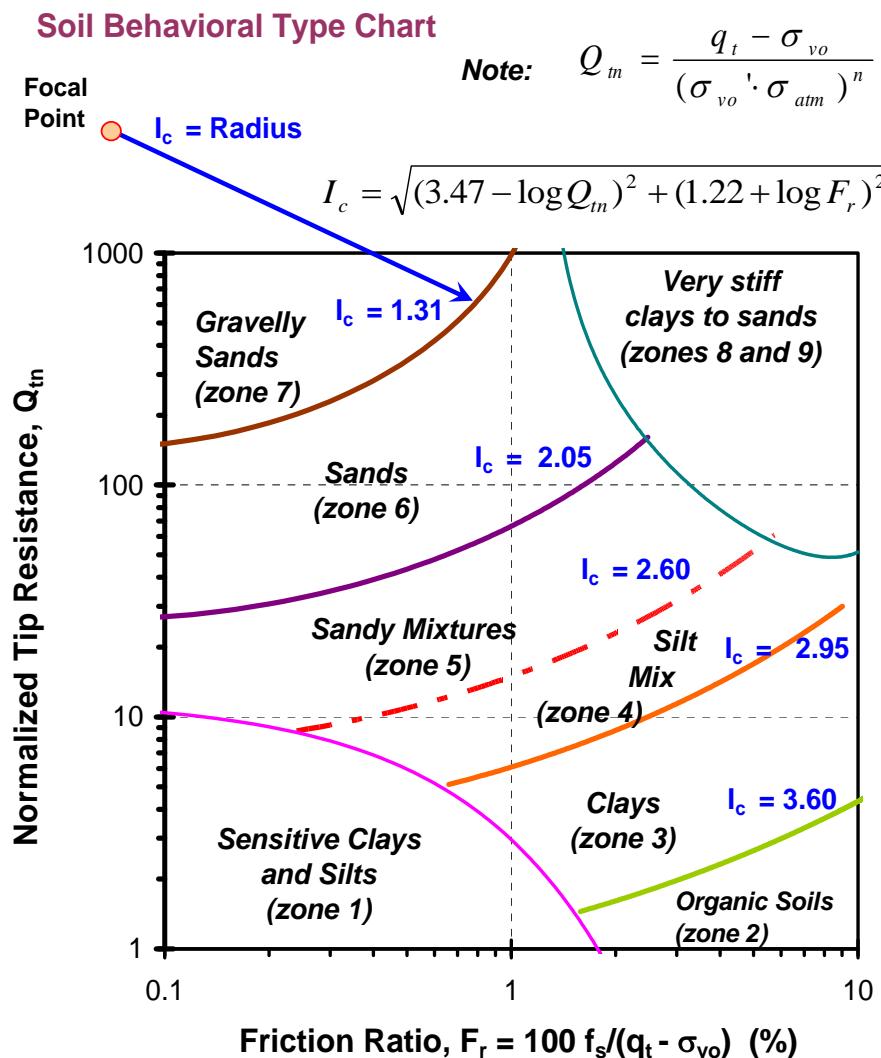


Figure 2. Soil behavioral type evaluated from CPT material index (after Robertson, 2004)

The original value of n (and assumed initial value) is taken as $n = 1$ (Robertson, 1990), but then adjusted to specific soil type by trial and error, where clean sands exhibit $n \approx 0.5$, silts have $n \approx 0.75$, and clays show $n \approx 1.0$. After the first estimate of I_c is made using $n = 1$, the results can be quickly modified using the algorithm provided by Robertson (2009):

$$n = 0.381 \cdot I_c + 0.05 \cdot (\sigma_{vo}' / \sigma_{atm}) - 0.15 \quad (10)$$

Iteration gives the final value of I_c . In general, sandy soils that might liquefy exhibit $I_c < 2.6$. Other considerations concerning problematic soils (e.g., sensitive soils, silts) during high seismicity are discussed elsewhere (Robertson, 2009, 2010).

Equivalent CPT Sand Resistance

The original studies of sand liquefaction under controlled laboratory studies focused on clean "hourglass" sands, primarily with grains of quartz or silica (approx. equal parts quartz and feldspar) composition. Since natural sands often contain some percentage of fines (soil particles < 0.075 mm in size), a modification of the stress-normalized cone tip resistance is made on the basis of fines content:

$$Q_{tn-cs} = K_c \cdot Q_{tn} \quad (11)$$

$$\text{For } I_c \leq 1.64: \quad K_c = 1.0 \quad (12a)$$

$$\text{For } I_c > 1.64: \quad K_c = -0.403 \cdot I_c^4 + 5.581 \cdot I_c^3 - 21.63 \cdot I_c^2 + 33.75 \cdot I_c - 17.88 \quad (12b)$$

where Q_{tn-cs} is the equivalent normalized cone resistance for clean sand (also designated q_{t1n-cs}). The apparent fines content (FC in %) can also be estimated from the following:

$$\text{For } I_c < 1.64: \quad FC (\%) = 0 \quad (13a)$$

$$\text{For } 1.64 \leq I_c \leq 3.5: \quad FC (\%) = 1.75 \cdot I_c^{3.25} - 3.7 \quad (13b)$$

$$\text{For } I_c > 3.5: \quad FC (\%) = 100 \quad (13c)$$

Cyclic Resistance Ratio

While the cyclic stress ratio (CSR) expresses the level of driving forces to cause soil liquefaction, the stress-normalized cone resistance (Q_{tn-cs}) represents the available strength of the ground to resist liquefaction. As such, there is a threshold value, termed the cyclic resistance ratio (CRR), such that driving forces equal the resisting forces (i.e. factor of safety, FS = 1). If the earthquake ground motions are sufficiently strong, they will exceed the available resistance (CSR > CRR) and liquefaction may be induced. On the other hand, if the level of seismic shaking is less than the available soil strength (CSR < CRR), then liquefaction is unlikely.

The relationship between CSR, CRR, and Q_{tn-cs} is shown in Figure 3 with data from seismic sites that have either *liquefied* (solid dots) or *not liquefied* (open dots). Evidence of liquefaction includes the development of sand boils, sand dikes, settlements, subsidence, and/or general ground failure. The CRR can be represented by (Robertson & Wride, 1998):

$$\text{If } Q_{tn-cs} < 50: \quad CRR_{7.5} = 0.833 \cdot [(Q_{tn-cs})/1000] + 0.05 \quad (14a)$$

$$\text{If } 50 \leq Q_{tn-cs} < 160: \quad CRR_{7.5} = 93 \cdot [(Q_{tn-cs})/1000]^3 + 0.08 \quad (14b)$$

where $CRR_{7.5}$ is the cyclic resistance ratio for a moment magnitude earthquake of $M_w = 7.5$.

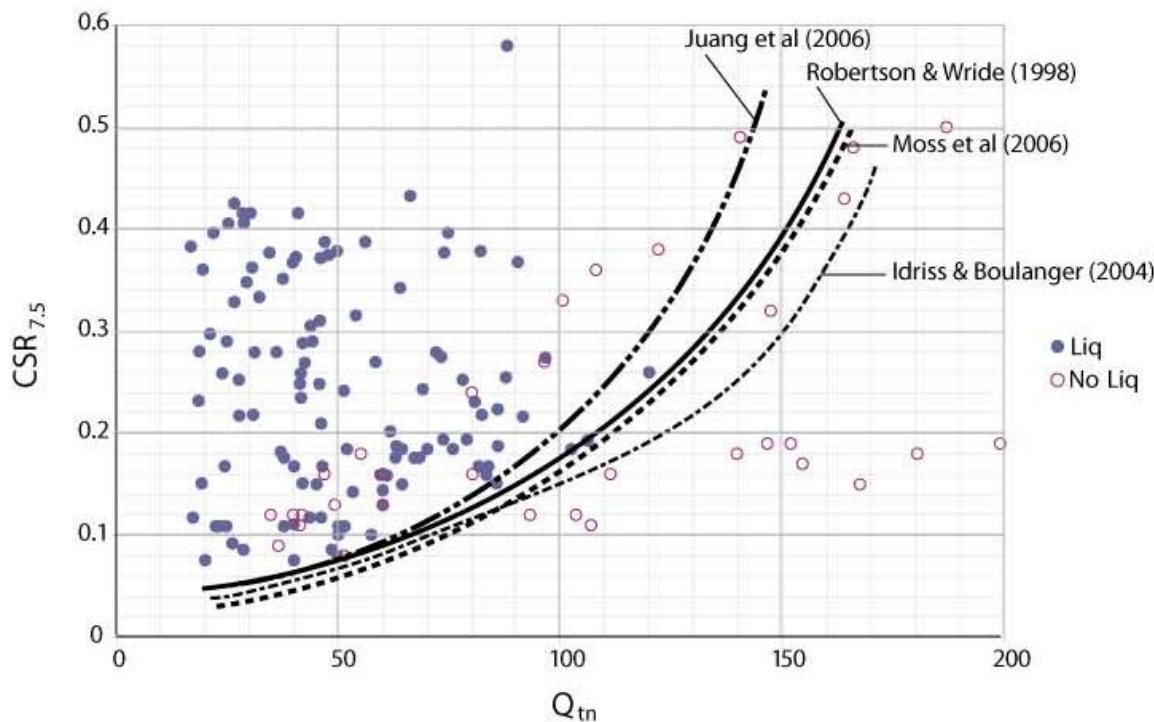


Figure 3. Cyclic stress ratio vs. normalized cone resistance with recommended CRR lines from different liquefaction research studies (from Robertson 2009).

Magnitude Scaling Factors

For earthquakes of moment magnitude other than $M_w = 7.5$, the cyclic resistance ratio can be scaled up or down using a Magnitude Scaling Factor (MSF):

$$\text{CRR} = \text{MSF} \cdot \text{CRR}_{7.5} \quad (15)$$

As discussed by Youd et al. (2001), the recommended expression is:

$$\text{MSF} = 173.8 \cdot M_w^{-2.56} \quad (16)$$

Factor of Safety and Probability of Liquefaction

The factor of safety (FS) can be calculated in terms of the cyclic resistance ratio given by eqn(15) and cyclic stress ratio expressed by eqn(1):

$$\text{FS} = \text{CRR}/\text{CSR} \quad (17)$$

The risk of liquefaction (P_L) can be quantified. Using the reference CRR for liquefaction defined by Robertson & Wride (1998), Juang & Jiang (2000) conducted a statistical analysis to obtain:

$$P_L = \frac{1}{1 + FS^{3.34}} \quad (18)$$

References

- Chen, C.J. and Juang, C.H. (2000). Calibration of SPT- and CPT-based liquefaction evaluation methods. *Innovations and Applications in Geotechnical Site Characterization* (Proc. GeoDenver), GSP 97, ASCE, Reston/VA: 49-64.
- Housner, G.W. et al. (1985). *Liquefaction of Soils During Earthquakes*. Committee on Earthquake Engineering, Commission on Engineering & Technical Systems, National Research Council, National Academy Press, Washington, DC: 240 p.
- Juang, C.H. and Jiang, T. (2000). Assessing probabilistic methods for liquefaction potential evaluation. *Soil Dynamics and Liquefaction 2000* (Proc. GeoDenver), GSP 107, ASCE, Reston, Virginia: 148-162.
- Mayne, P.W., Peuchen, J. and Bouwmeester, D. (2010). Unit weight evaluation from CPT. *Proceedings, 2nd Intl. Symposium on Cone Penetration Testing (CPT'10)*, Vol. 2, Huntington Beach, California: 169-176.
- Robertson, P.K. (1990). Soil classification using the CPT. *Canadian Geotechnical Journal* 27 (1): 151-158.
- Robertson, P.K. and Wride (Fear), C.E. (1998). Evaluating cyclic liquefaction potential using the cone penetration test. *Canadian Geotechnical Journal* 35 (3): 442-459.
- Robertson, P.K. (2004). Evaluating soil liquefaction and post-earthquake deformations using the CPT. *Geotechnical and Geophysical Site Characterization*, Vol. 1 (Proc. ISC-2, Porto), Millpress, Rotterdam: 233-252.
- Robertson, P.K. (2009). Performance-based earthquake design using the CPT. *Performance-Based Design in Earthquake Geotechnical Engineering (Proceedings, International Symposium - Tokyo)*, CRC Press - Taylor & Francis Group, London: 3-20.
- Robertson, P.K. (2010). Evaluation of flow liquefaction and liquified strength using the cone penetration test. *Journal of Geotechnical & Geoenvironmental Engineering* 136 (6): 842-853.

Seed, H.B. (1976). State of the art paper: Evaluation of soil liquefaction effects on level ground during earthquakes. *Liquefaction Problems in Geotechnical Engineering*, (Proc. ASCE National Convention, Philadelphia), Preprint 2752, American Society of Civil Engineers, Reston/VA: 1-104.

Seed, H.B. (1979). Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. *Journal of Geotechnical Engineering*, Vol. 105 (GT2): 201-256.

Shedlock, K.M. (1999). Seismic hazard map of North and Central America and the Caribbean. *Annali di Geofisica*, Vol. 42 (6): 977-997.

Youd, T.L., Idriss, I., et al. (2001). Liquefaction resistance of soils: summary report from the NCEER/NSF workshops. *Journal of Geotechnical and Geoenvironmental Engineering* 127 (10): 817-833.

Youd, T.L. and Idriss, I.M., editors (1997). *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils* (Salt Lake City, Utah). Technical Report NCEER-97-0022, National Center for Earthquake Engineering Research, Buffalo, NY: 276 p.

Zhang, G., Robertson, P.K., and Brachman, R.W.I. (2002). Estimating liquefaction-induced ground settlements from CPT for level ground. *Canadian Geotechnical Journal* 39 (5): 1168-1180.

Zhang, G., Robertson, P.K., and Brachman, R.W.I. (2004). Estimating liquefaction-induced lateral displacements using the standard penetration test of cone penetration test. *Journal of Geotechnical and Geoenvironmental Engineering* 130 (8): 861-871.

CPT'10
HUNTINGTON BEACH, CA
CPT'10 - Huntington Beach CA
www.cpt10.com

Epilogue: CPT'10

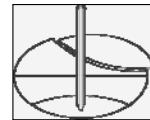
2nd International Symposium on
Cone Penetration Testing

Paul W. Mayne
Georgia Institute of Technology
ISSMGE TC10

TC16 - In-Situ Testing

ISSMGE Technical Committee 16
Ground Property Characterization by In-Situ Tests
<http://www.webforum.com/tc16>

International Society for Soil Mechanics & Geotechnical Engineering
<http://www.issmge.org>



Core Members (2005-2009):

Paul W. Mayne	Georgia Inst. Technology, USA
Antonio Viana da Fonseca	Univ. of Porto, Portugal
John J.M. Powell	BRE and GeoLabs, UK
Martin Fahey	Univ. Western Australia
An-Bin Huang	National Chiao Tung Univ, Taiwan
Emoke Imre	Budapest University, Hungary
Tom Lunne	Norwegian Geotechnical Institute
Zbigniew Mlynarek	Poznan University, Poland
Fernando Schnaid	Fed. Univ. Grande do Sol, Brazil

Advisors:

Mike Kamiolkowski	Studio Geotecnico, Italy
Peter K. Robertson	Gregg Drilling, California, USA
K. Rainer Massarsch	GeoEngineering AB, Sweden

1st International Symposium on
Cone Penetration Testing (CPT'95)
Linköping, Sweden
Host: K. Rainer Massarsch



2nd International Symposium on
Cone Penetration Testing (CPT'10)
Huntington Beach, California, USA
Hosts: Peter K. Robertson
Kelly Cabal



ISSMGE Technical Committee 16
Ground Property Characterization by In-Situ Tests
<http://www.webforum.com/tc16>

International Society for Soil Mechanics & Geotechnical Engineering
<http://www.issmge.org>

www.webforum.com/tc16

New ISSMGE Board (2009-2013)

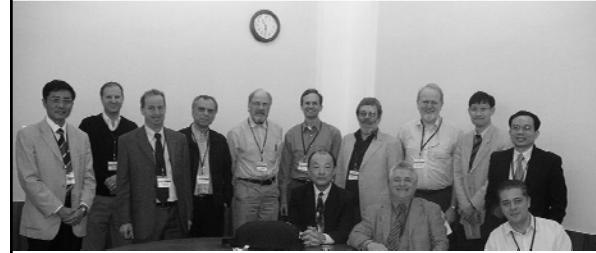
TC16 = TC102

"TC on In-Situ Testing"

Letter to
CPT'10 from
ISSMGE President
Jean-Louis Briaud



ISSMGE Technical Committee
TC 16 (TC102) - In-Situ Testing
www.webforum.com/tc16



CPT'10 - Summary Regional Reports

OUTLINE for Regional Reports

- Introduction: Geography, Topography, Climate, Population
- Geology of Region
- Geotechnical Challenges:
 - Overview of site investigation practices
 - Major issues: settlement, piles, problem soils, seismicity
- Equipment and Procedures
 - Test Standards
 - Special measures or methods
 - Equipment used
 - Percentage use of CPT in region
- CPT Interpretation
 - Soil type and stratigraphy
 - Geotechnical parameters
 - Other derived information
- Applications of CPT
- Summary - Acknowledgments
- References

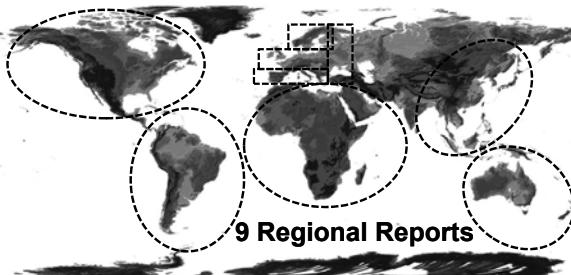


Dr. Peter K. Robertson

**Host for CPT'10
Kelly Cabal**



CPT'10 - Summary Regional Reports



9 Regional Reports

World Map by NASA and Japan's Ministry of Economy, Trade, & Industry

CPT

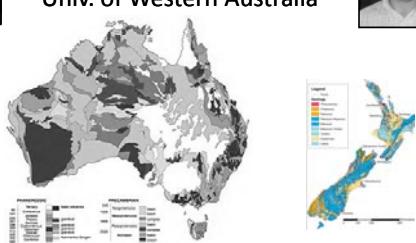
- Current Phase Transformer
- Cross Product Team
- Cellular Paging Teleservice
- Chest Percussion Therapy
- Crisis Planning Team
- Consumer Protection Trends
- Computer Placement Test
- Current Procedural Terminology
- Cost Per Treatment
- Choroid Plexus Tumor
- Cardiopulmonary Physical Therapy
- Corrugated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

CPT'10 - Summary Regional Reports

Regional Report for Australia - New Zealand

Martin Fahey and Barry Lehane
Univ. of Western Australia





CPT'10 - Summary Regional Reports

Regional Report for Asia

An-Bin Huang
National Chiao Tung Univ. - Taiwan

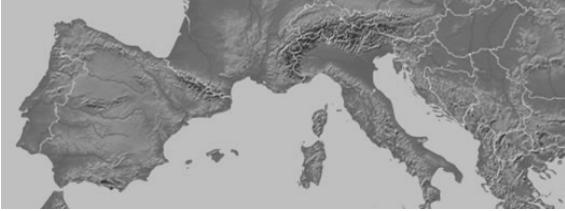




CPT'10 - Summary Regional Reports

Regional Report for Southern Europe

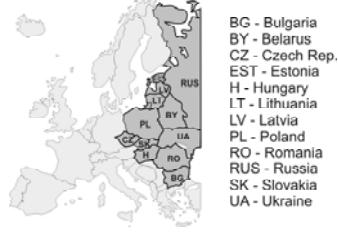
António Viana da Fonseca
Univ. of Porto, Portugal

CPT'10 - Summary Regional Reports

Regional Report for Eastern Europe

Zbigniew Mlynarek
Poznan University, Poland

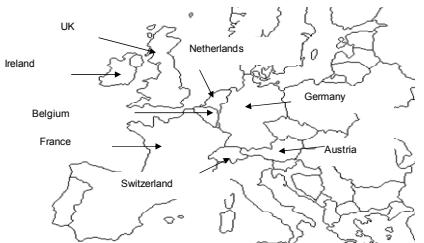



BG - Bulgaria
BY - Belarus
CZ - Czech Rep.
EST - Estonia
H - Hungary
LT - Lithuania
LV - Latvia
PL - Poland
RO - Romania
RUS - Russia
SK - Slovakia
UA - Ukraine

CPT'10 - Summary Regional Reports

Regional Report for Northern Europe

Mike Long
University of Dublin, Ireland

CPT'10 - Summary Regional Reports

Regional Report for Africa and Middle East

Tamer Elkateb
Ains Sham University, Egypt




CPT'10 - Summary Regional Reports

Regional Report for Nordic Europe

Hjördis Löfroth
Swedish Geotechnical Institute



with
Rolf Sandven
Norway



Carsten Bonde
Denmark



Hannu Halkola
Finland




CPT'10 - Summary Regional Reports

Regional Report for South America

Roberto Coutinho
Univ. Pernambuco



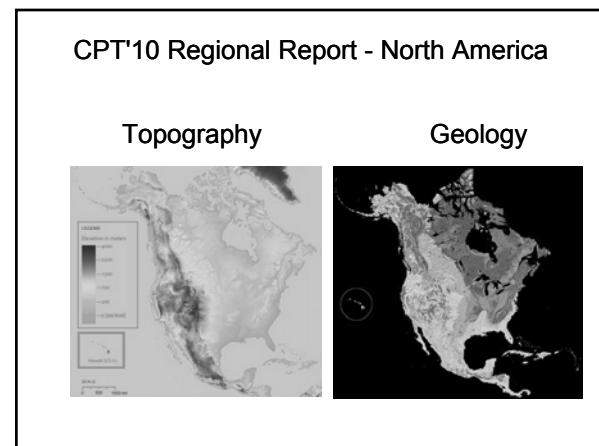
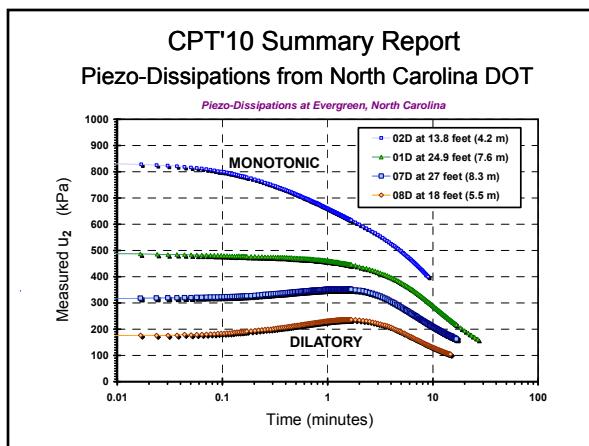
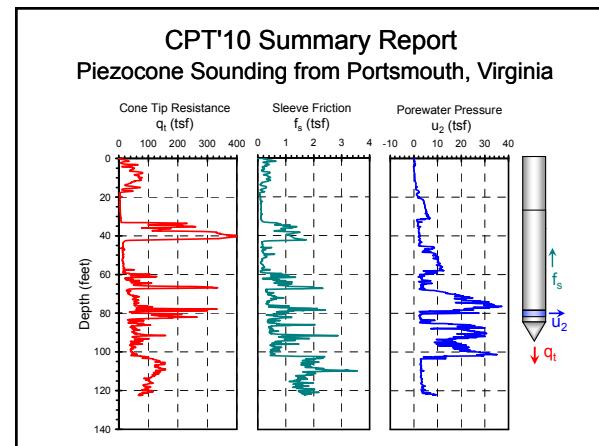
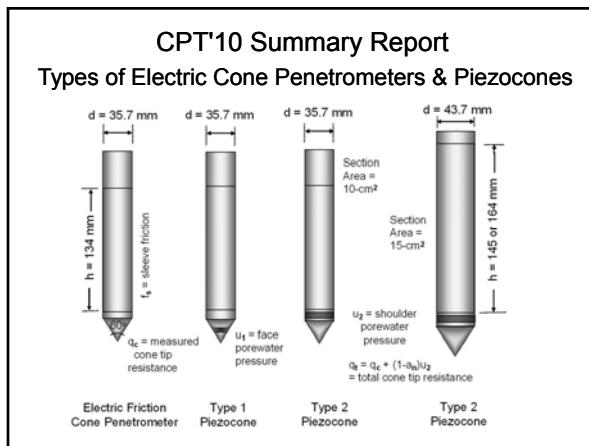
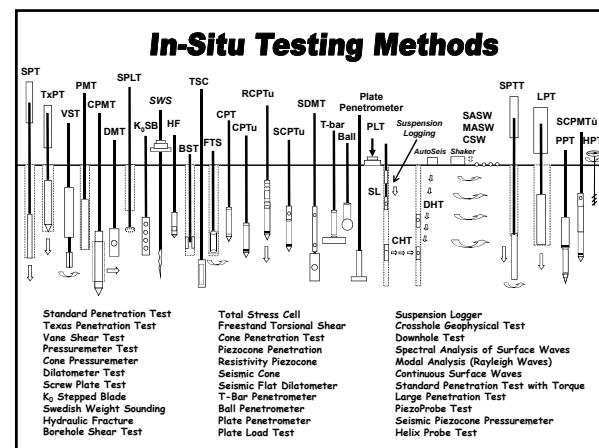

Fernando Schnaid
Fed. Univ. Rio Grande do Sol Brazil

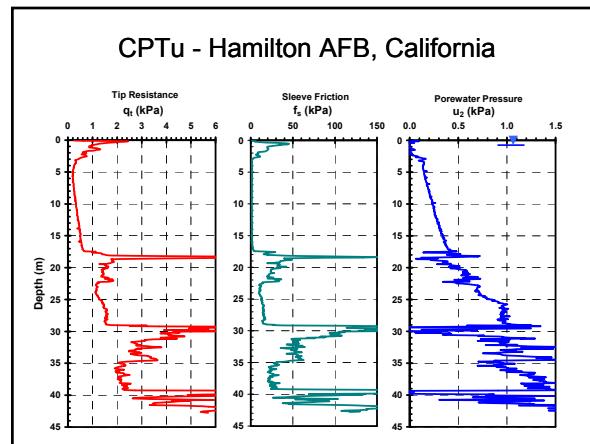
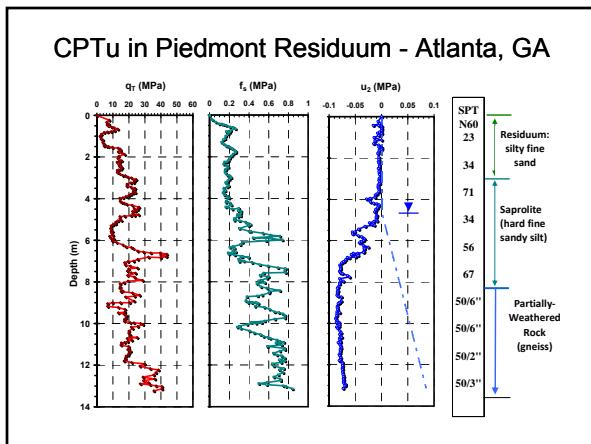
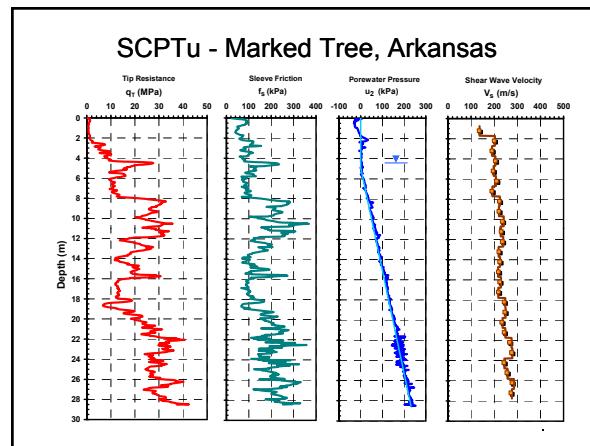
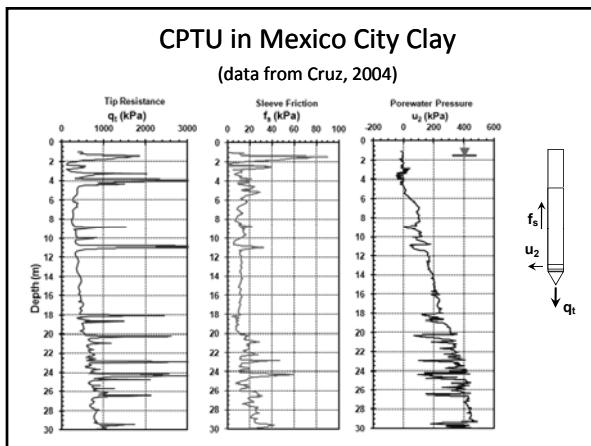
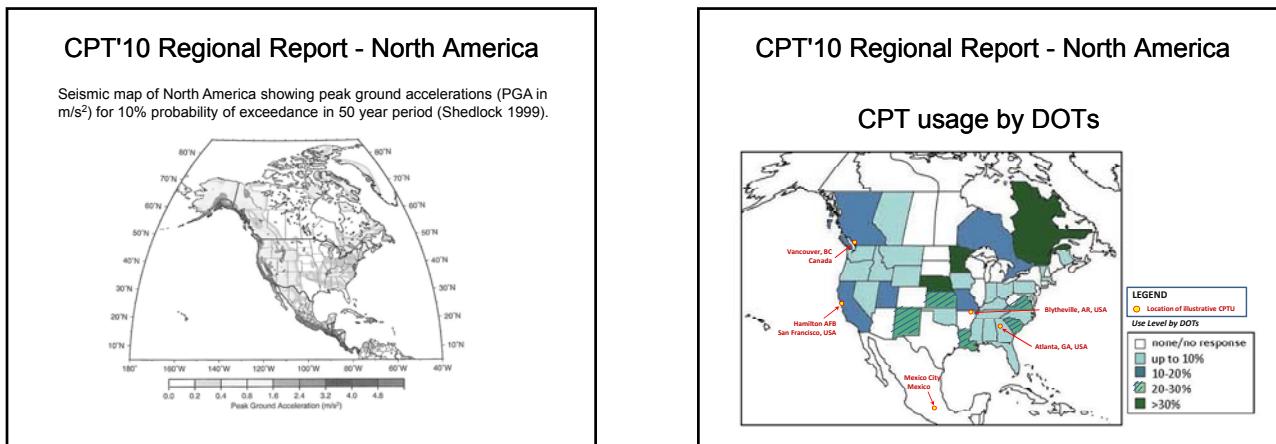


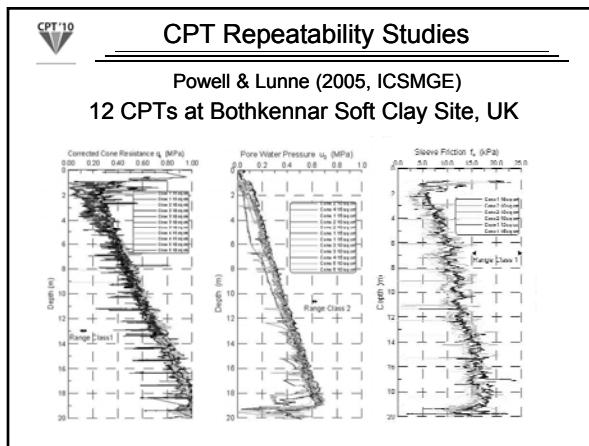
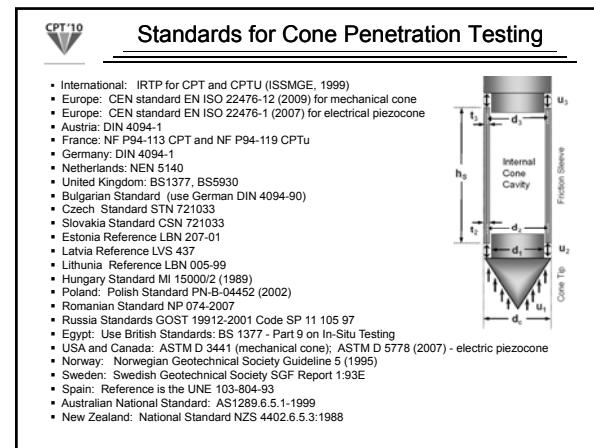
CPT'10 - Summary Regional Reports

Regional Report for North America

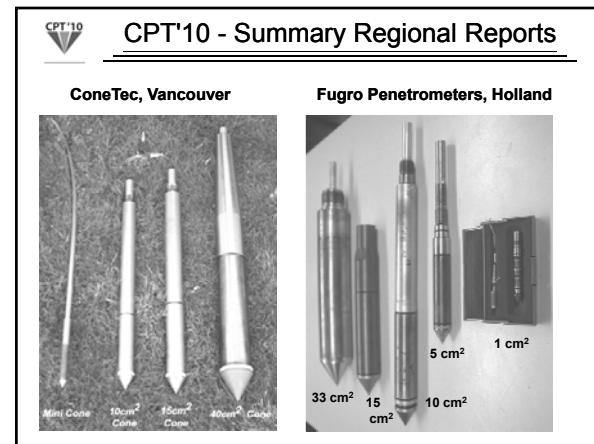
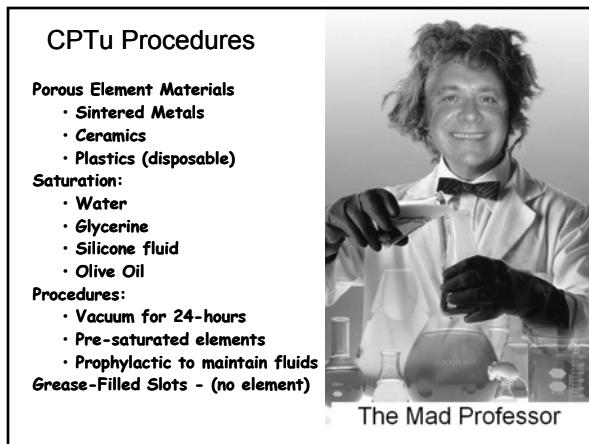
Paul Mayne
Georgia Tech,
Atlanta, USA







- Calibrations of Penetrometers**
- Two types of calibrations necessary.
 - Load cell verification in compression machine (proving ring) for q_c and f_s
 - Penetrometer in pressurized triaxial cell to detail porewater effects on geometry to obtain q_t and $f_t \rightarrow a_{net}$ and b_{net}
 - Pressure triaxial calibrations also verify porewater transducer output.



CPT'10 - Summary Regional Reports

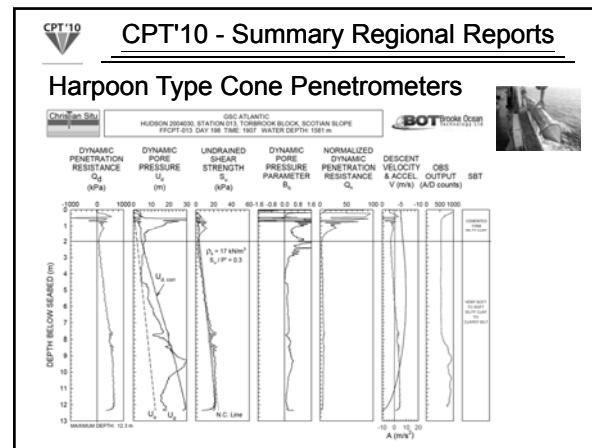
Harpoon Type Cone Penetrometers

"Free-Fall Cone Penetration Test"

- US Navy XDP
- Canadian FFCPT
- German MARUM

Diagram of the Harpoon FFCPT-660 probe:

- Tether Point
- DRP
- DRP
- Ball Model
- Tail Fin
- U.P.
- Electronics
- Retracted Probe Module
- Central Cone
- Porous Ring
- Mass Core Sheath
- Harpoon
- FFCPT-660



CPT'10 - Summary Regional Reports

CPT Probes for Centrifuge - Univ. Western Australia

0 50 mm

Mini-Cone Penetrometers

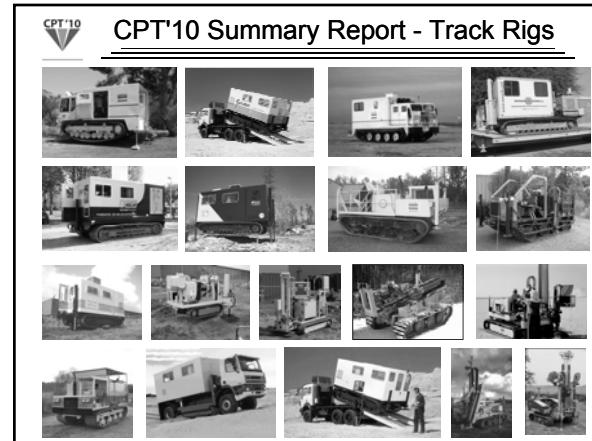
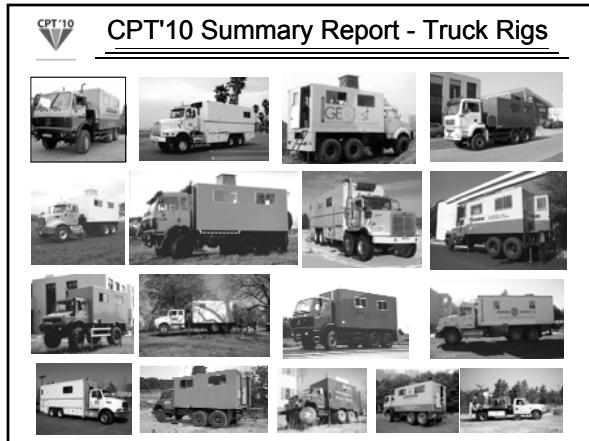
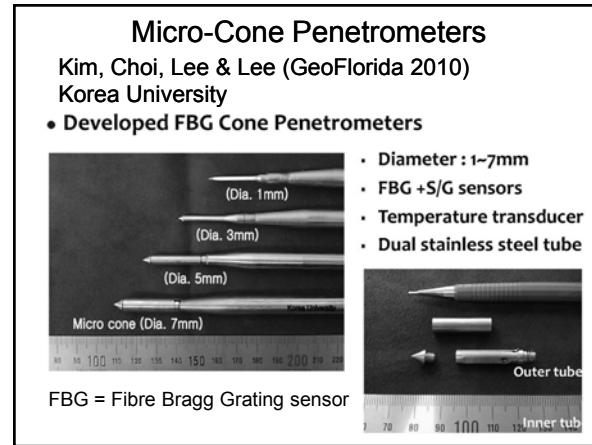
T-bar

Ball

Plate

Main Centrifuge

Drum Centrifuge



CPT'10 - Summary Regional Reports

Geoprobe Systems Model 6625 CPT
 (courtesy: Troy Schmidt and Tom Christy)



Used as a track rig Or left on truck carrier

CPT'10 - Special CPT Vehicles



Combine Drill-CPT Rig for Hard Ground
 Drill CPT

CPT'10 - Summary Regional Reports

Allan McConnell
 Insitu Geotech Services



Real Cool Cone Stuff



Ice-Cream Cone Truck

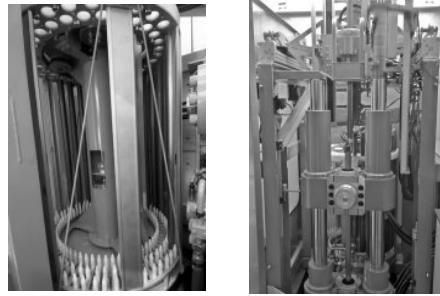
CPT'10 - Summary Regional Reports

Definition: Rodman in Civil Engineering



CPT'10 - Summary Regional Reports

AutoCoson - Automated robotic CPT system
 A.P. van den Berg, Holland



CPT'10 - Special Small CPT Rigs

CPT'10 - Summary Regional Reports

江苏省如皋市
如城精密工程勘测机械厂

圆锥贯入试验
Chinese CPT Equipment
www.madeinchina.com

CPT'10 - Summary Regional Reports

Railway CPT Deployment

Recent Economic Losses in Our Pensions
The NEW GEOTECHNICAL RETIREMENT PLAN

CPT'10 - Summary Regional Reports

Geologic Map of the World: www.OneGeology.org

NearShore and Offshore CPT Deployment

Fugro Engineers

CPT'10 - Summary Regional Reports

CPT Near Shore Jack-up Platforms

IG5 Jackup Fugro Seacore USAE Explorer
Geomil Jackup iGeotest DS3 IG3 Platform

CPT'10 - Summary Regional Reports

Offshore CPTs by Dynamic Positioning Ships

Explorer Fugro
Markab Bucentaur

CPT'10 - Summary Regional Reports

Systems for Seabed Deployment of CPT

Roson Gregg Igeotest Wilson
Neptune Seacalf Roson Sage Sidewinder

PROD = Portable Remotely Operated Drill
Benthic Geotech: <http://www.bgt.com.au/>

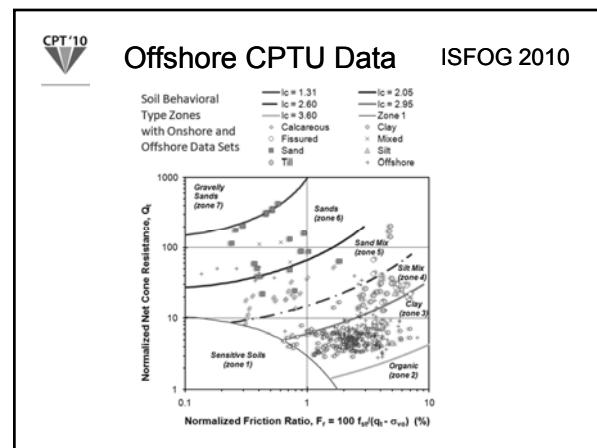
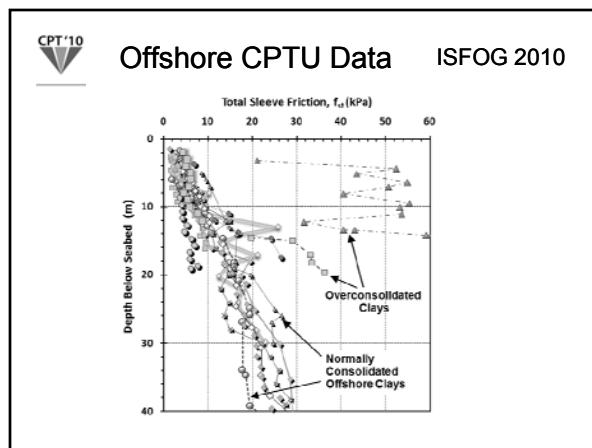
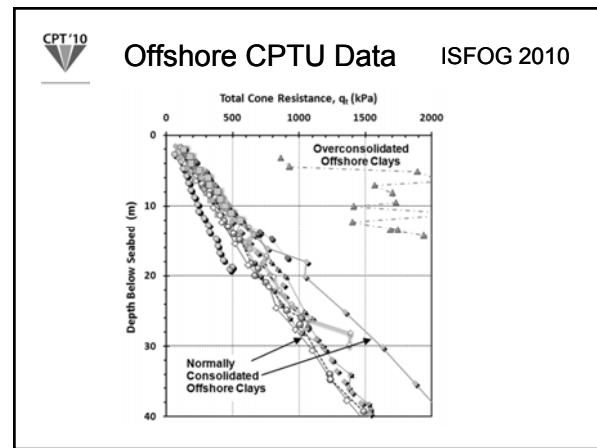
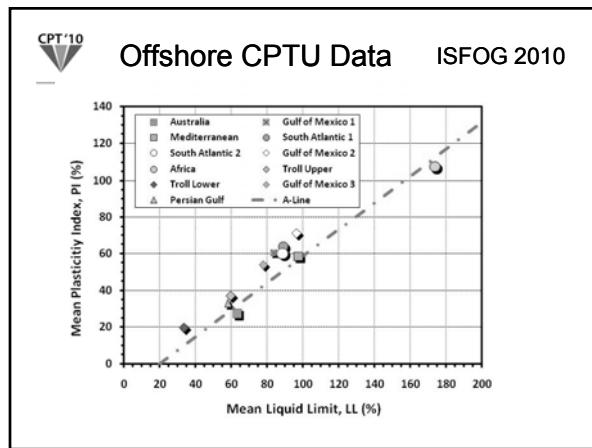
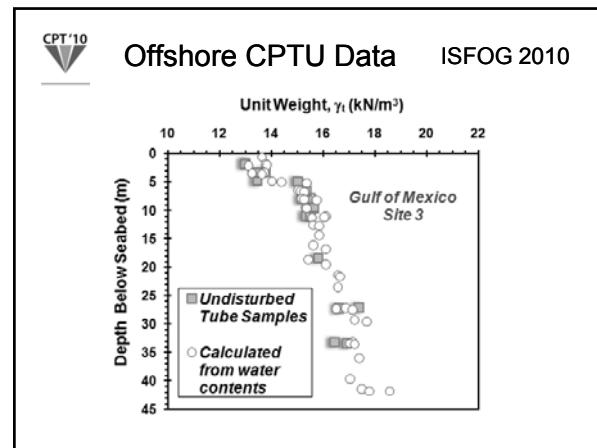
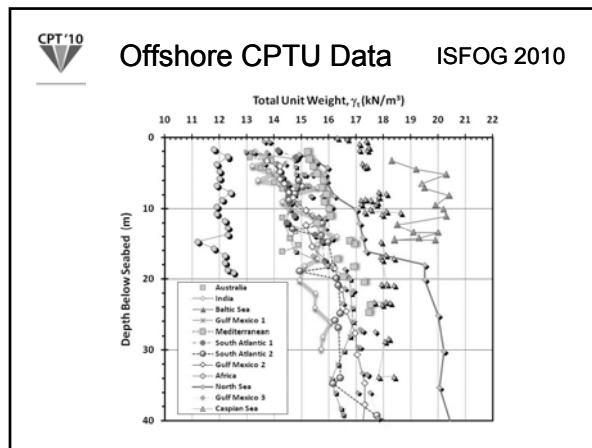
2000-m water depths
150-m exploration depths below mudline

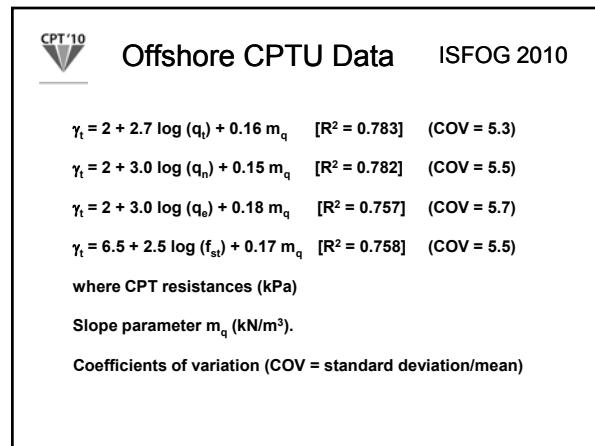
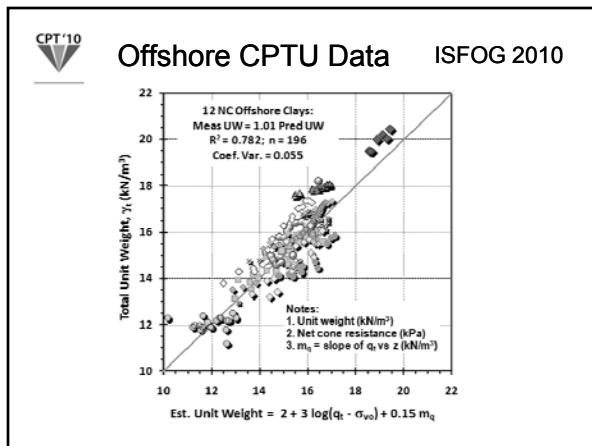
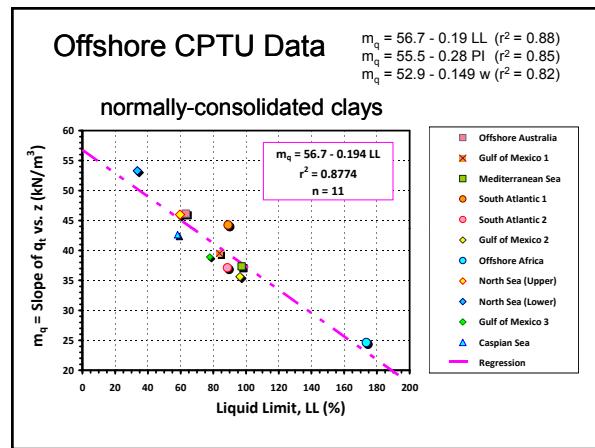
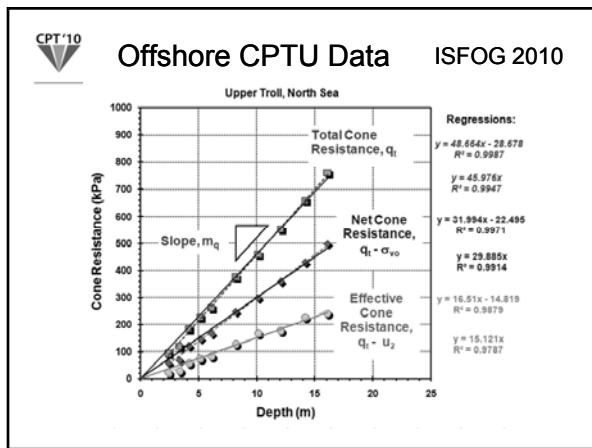
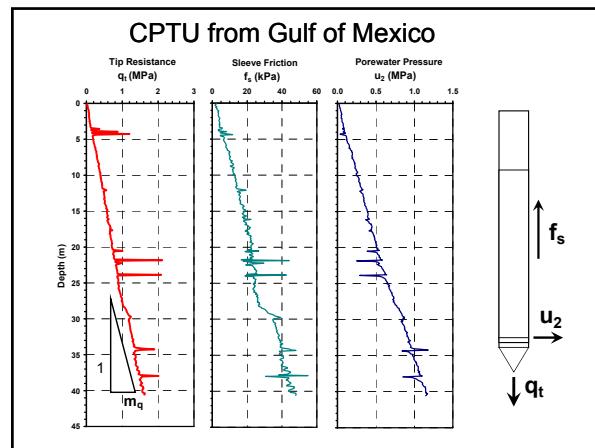
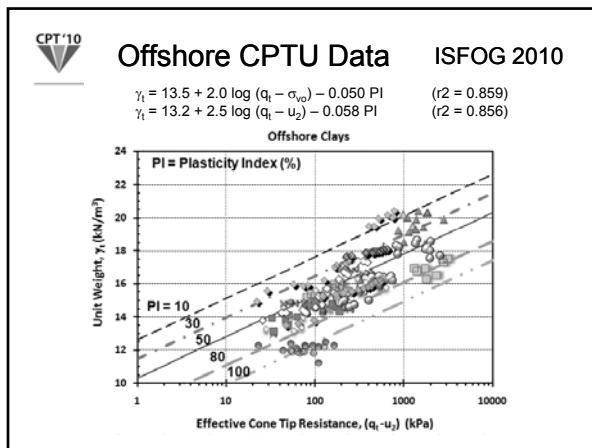
Continuous Coil Tube Injectors

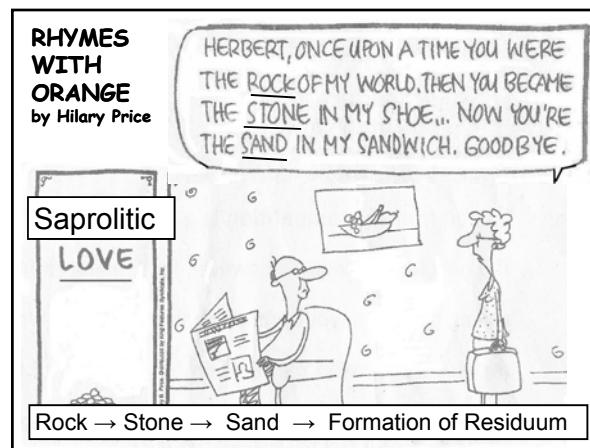
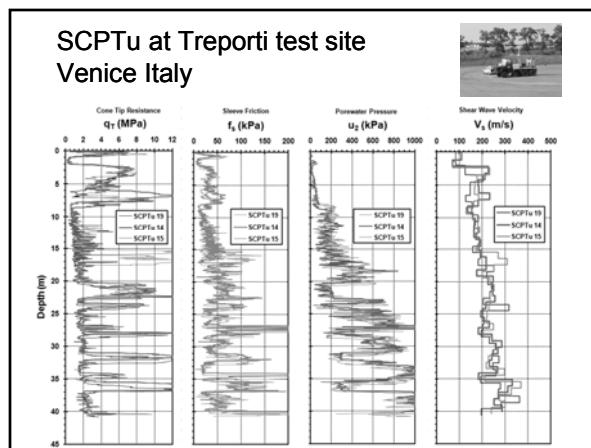
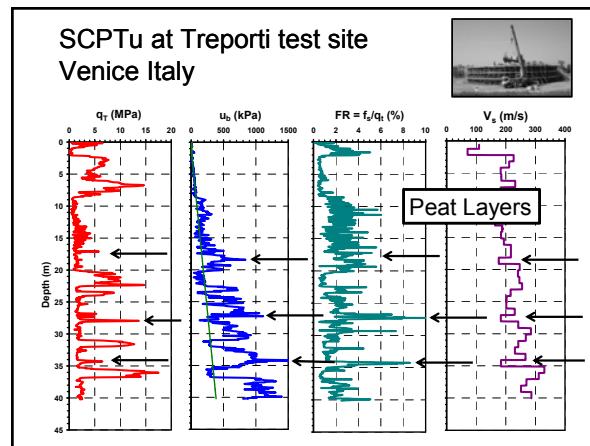
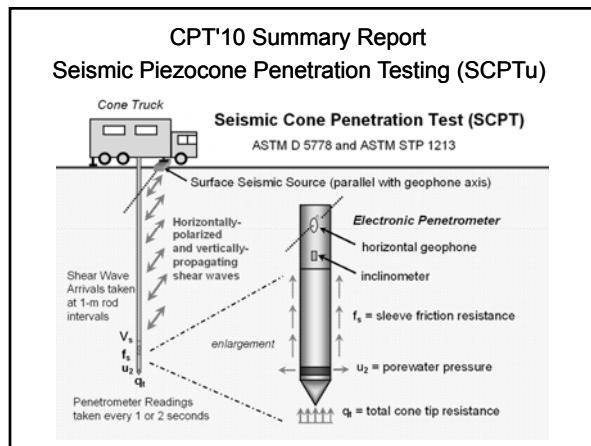
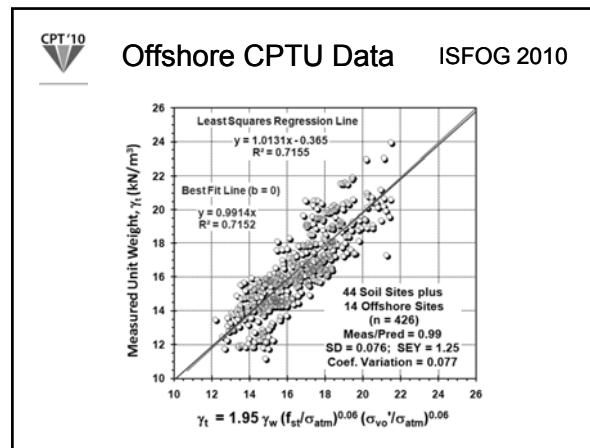
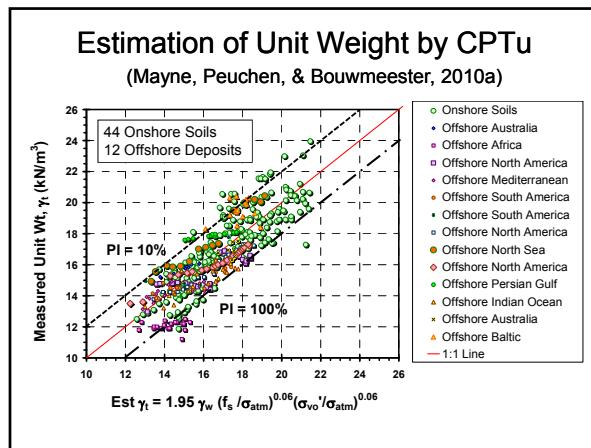
Gregg Drilling Sage-Engineering
Louisiana Transportation Research Center iGeotest Neptune

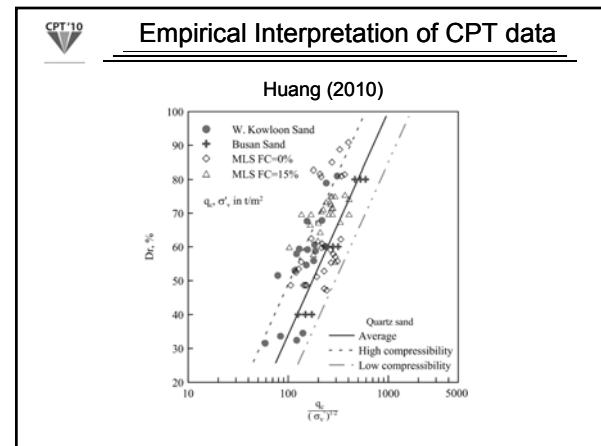
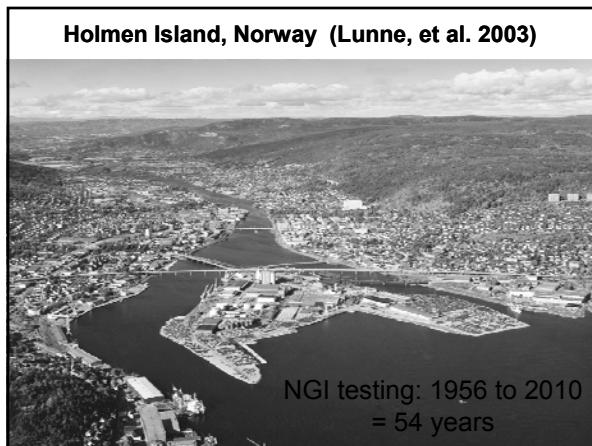
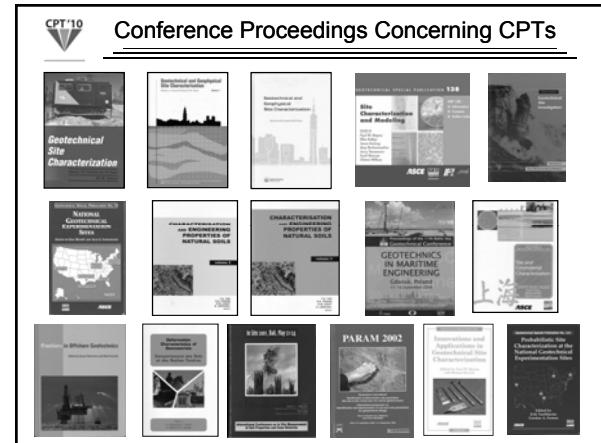
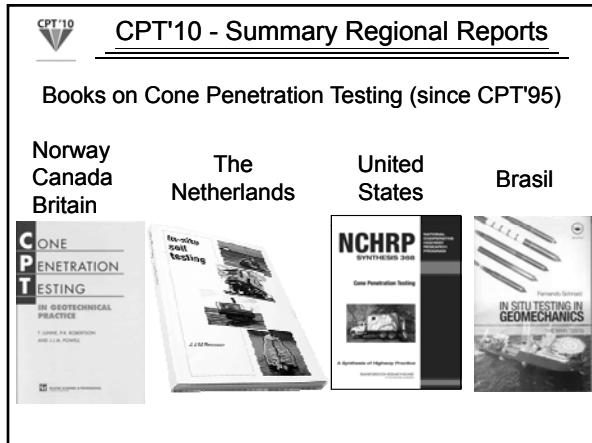
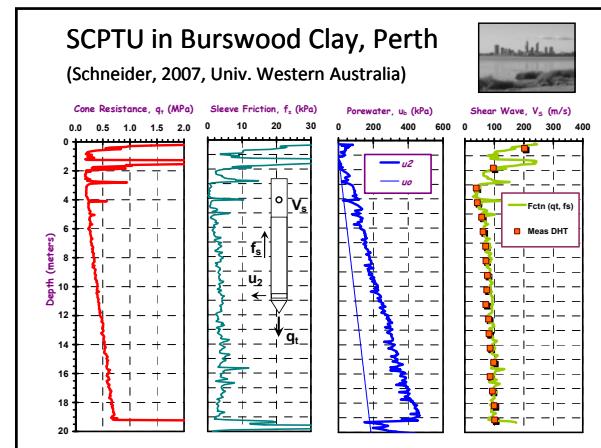
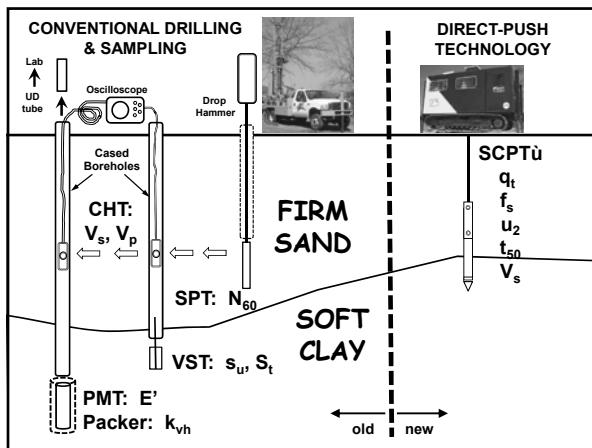
Continuous Coil Tube Injectors

Fluid Design Solutions Inc
Edmonton









CPT'10 Interpretation of CPT by Analytical Methods

Coupled Cavity Expansion-Critical State Model

$$OCR = 2 \cdot \left[\frac{1}{1.95M + 1} \left(\frac{q_i - u_2}{\sigma_{vo}} \right) \right]^{(1/\lambda)}$$

Soil Properties:
 $M = 6 \sin\phi' / (3 - \sin\phi')$
 ϕ' = effective stress friction angle
 C_c = compression index
 C_s = swelling index
 $A = 1 - C_s/C_c$
 $I_R = G/s_u$ = Undrained Rigidity Index
 G = shear modulus
 s_u = undrained shear strength

CPT'10 Finite Element Modeling of CPT

LSU - Tumay et al. (2008)
Univ. Michigan Susila & Hryciw (2003)

CPT'10 Discrete Element Modeling of CPT

Jiang, Yu, & Harris (2006)
Univ. Nottingham; Univ. Manchester

CPT'10 Strain Path Modeling of CPT

MIT E-3 and SPM Dissipation Curves (Whittle, 2005)

CPT'10 Dislocation Analysis of CPT

Lee, Elsworth, and Hryciw (JGGE 2008)

CPT'10 - Summary Regional Reports

Soil Parameter Interpretation from CPT

Soil Parameter	Number of Replies
Coef. Secondary, G _s	~2
Constrained Mod, D'	~2
Swelling Index, C _s	~2
Bearing Ratio, CBR	~3
Resilient Modulus, MR	~3
Subgrade Coef, k _s	~4
Rigidity Index, IR	~4
Poisson's Ratio, v	~4
Horizontal Coef, k _o	~5
Effective Cohesion Intercept, c _e	~7
Small Strain Gmod, G _c	~8
Compressibility, k	~10
Permeability, k _v	~12
Coef. Consolidation, cv _h	~13
Elastic Modulus, E	~15
Overconsolidation Ratio, OCR	~17
Relative Density, Dr	~19
Undrained Shear Strength, s _u	~22
Effective Friction Angle, φ	~24

64 DOTs

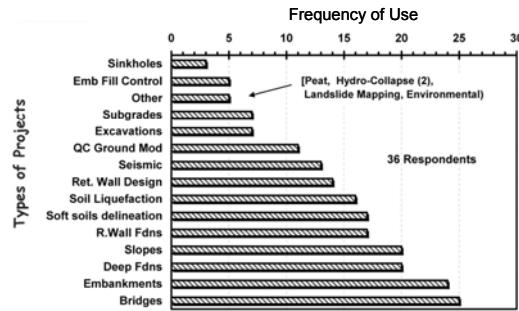
CPT Computer Programs for Interpretation

- GEO5 Pile CPT (gintsoftware)
- GLAMCPT (Omer 2006)
- CPET-IT (geologismiki.gr)
- LPD-CPT (Louisiana State University)
- Conrad (Swedish Geotech Institute)
- CPTINT (Dick Campanella)
- CPTLiq (Geosoftwaresolutions)
- CPT-Log (Geotech.se)
- DCCONE (dc-software.com)
- Edison - SGI
- MFoundation (GeoDelft)
- CPT-Pro (www.geosoft.com.pl)
- P-Class - (LSU and LTRC, Louisiana)
- PL-aid (McTrans at Univ. Florida)
- RapidCPT (dataforensics.net)
- Shake2000 (CPT for soil liquefaction)
- Static Probing (www.geostru.com)
- Unipile (www.unisoftitd.com)
- Georit (Geotech AB)
- CPT7 (GeoMil Equipment)
- ScreenZw (ConeTec)
- CPTGL (Geotech AB reduction)
- Gol (van den Berg)
- Almeid (UWA)
- CPTPlot
- CLiq (geologismiki.gr)
- GeoExplorer (Gouda-Geo Equip)
- SPAS 2009 (geologismiki.gr)
- INSITU (www.geo&soft.com)
- OpenSounding (sourceforge.net)
- PCL-pro (Geotech)
- PlotCPT (GeoMil Equipment)
- SCPT-DAA (Baziw)
- Static Penetrometer (Alpes-Geo)
- LPC360A (ConeTec)
- FB-Deep (Univ. of Florida)
- Gorilla (A.P. van den Berg)
- NovoCPT 1.2 (novosoftware.com)

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Applications of CPT

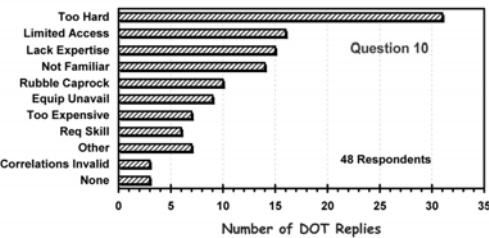
NCHRP 368 (2007)



CPT'10 - Summary Regional Reports

Reasons why DOTs not using CPT

Obstacles to Use of CPT



NCHRP 368 (2007)

Special Methods for CPT in Hard Ground

Advancing Technique	Reference	Comments/Remarks
Heavy 20-tonne Dead Weight Vehicles	Mayne, et al. (1995)	Increased mass
Friction Reducer	van de Graaf & Schenk (1988)	Opens large diameter hole before rods
Cycling of Rods (up and down)	Shinn (1995, personal comm.)	Locally in thin hard zones of soil
Large 44-mm diameter penetrometer	van de Graaf & Schenk (1988)	Works like friction reducer
Guide Casing: Double set of rods; standard 36-mm rods supported inside larger 44-mm rods; prevents buckling	Peuchen (1988)	Works well in situations involving soft soils with dense soils at depth
Drill Out (Downhole CPTs)	NNI (1996)	Alternate between drilling and pushing
Mud Injection	Van Staveren (1995)	Needs pump system for bentonitic slurry
Earth Anchors	Pagani Geotechnical Equipment Geoprobe Systems	Increases capacity for reaction
Static-Dynamic Penetrometer	Sanglerat et al. (1995)	Switches from static mode to dynamic mode when needed
Downhole Thrust System	Zuidberg (1974)	Single push stroke usually limited to 2 or 3 m
Very Heavy 30- and 40-ton Rigs	Bratton (2000)	After large 20-ton rig arrives at site, added mass for reaction
ROTAPE - outer coring bit	Sterckx & Van Calster (1995)	Special drilling capabilities through cemented zones
CPTWD	Sacchett et al. (2004)	Cone penetration test while drilling
Sonic CPT	Bratton (2000)	Vibrator to facilitate penetration through gravels and hard zones
EAPS (enhanced access penetrometer system)	Farrington (2000); Shinn & Haas (2004); Farrington & Shinn (2006)	Wireline systems for enhanced access penetrometer system



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Vertek 40-tonne CPT Truck

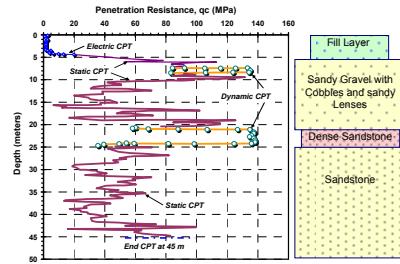


Static-Dynamic CPT System (AMAP Sols)



Mlynarek (2010)

AMAP Static-Dynamic CPT in Dense Sandstone



CPT'10 - Summary Regional Reports

CPTs in Mine Tailings - Australia

Marooka 12t tracked CPT rig on red tailings storage with desiccated crust, Kalgoorlie, WA

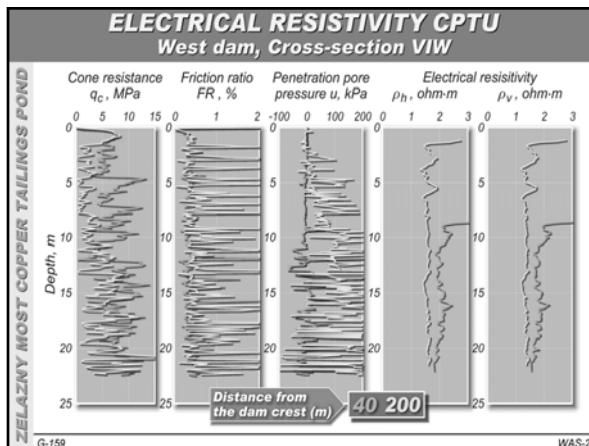
Portable 'Amphiroll' CPT vehicle on a bauxite residue tailings storage area, Kwinana, WA (Courtesy of Probedrill: www.probedrill.com.au).

Fahey & Lehane (2010)

Zelazny Most Copper Tailings Pond
(courtesy Mike Jamiolkowski)

Maximum dam height: 50.5 m
Total volume stored: $372 \times 10^6 \text{ m}^3$
Storage rate: $17.5 \times 10^6 \text{ m}^3/\text{annum}$
Area covered: 14.2 km²
Dam's perimeter: 14.5 km
Operation time: 1977 throughout 2035

G-159 WAS-01

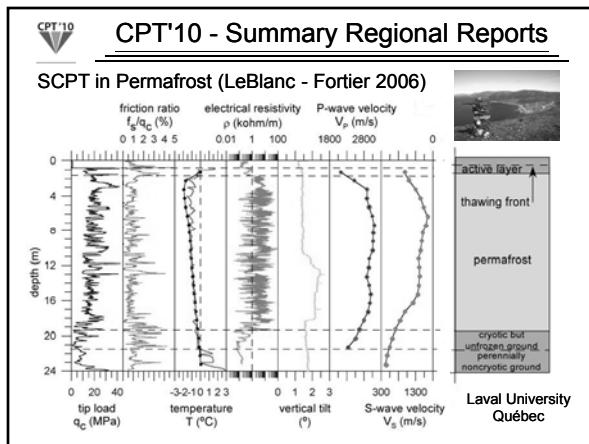


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Permafrost Regions of Canada
www.ec.gc.ca

Test Site
Laval University Québec
(LeBlanc & Fortier)

Cone Penetration Rig in Arctic Permafrost (courtesy ConeTec)



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CPTs Halley Research Station - Antarctica

CPT'10 - Summary Regional Reports

CPTs to Measure Strength of Polar Ice - Antarctica

Cone Resistance, q_c (MPa)

Depth (meters)

Adrian McCallum
Scott Polar Research Institute (SPRI)
Lankelma, UK

CPT'10 - Summary Regional Reports

Hand-held electronic cone penetrometers

Spectrum Scout SC 900 Eijkelcamp Digital Penetrometer

Rimik CP40 ii digital penetrometer

CPT'10 - Summary Regional Reports

Hand-held electronic cone penetrometers

Rimik CP40

Depth (mm)

Penetration Resistance

Rimik CP40 ii penetrometer

Excellent Repeatability

New Developments

VisCPT = vision penetrometer (Hryciw 2004)

Sapphire window

Visual data acquisition recording system

Clean Sand
Silty Sand
Silty fine Sand
Silty Clay

New Developments: Full-Flow Penetrometers

- T-bar
- Ball penetrometer
- Plate
- Developed for very soft soils (offshore)

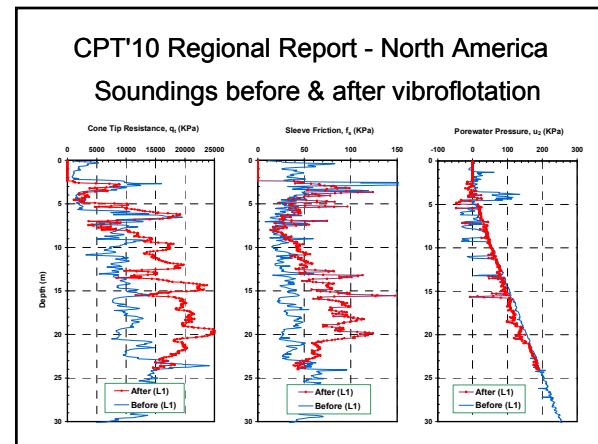
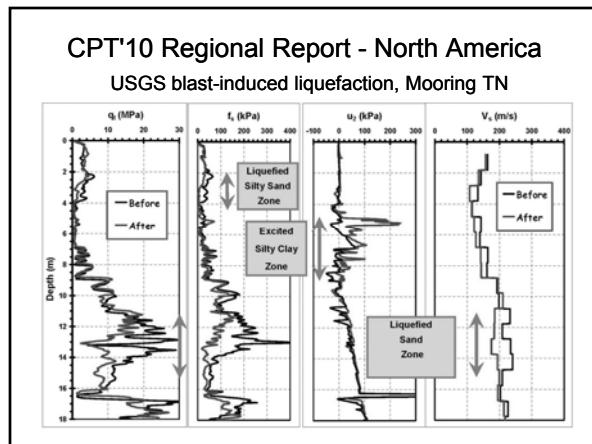
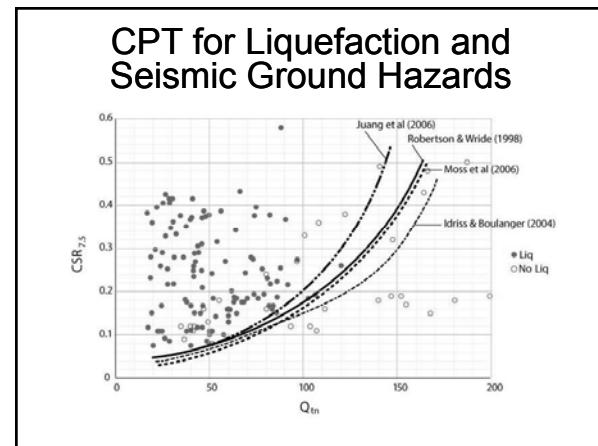
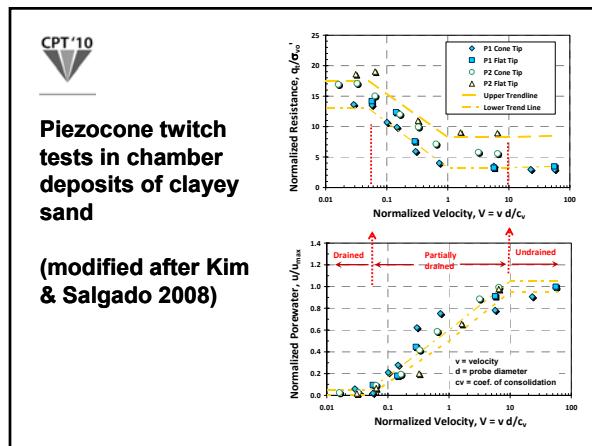
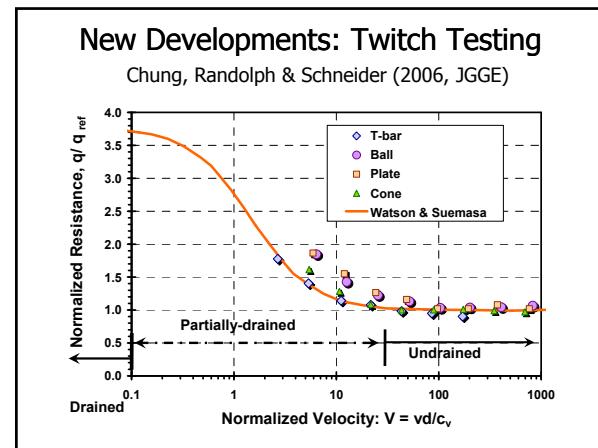
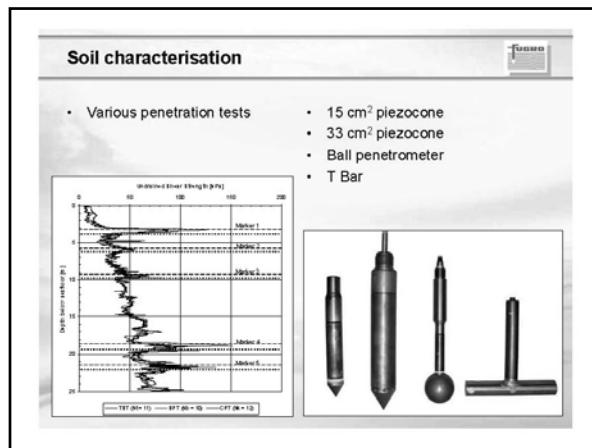
House, Stewart, & Randolph (ISC-1998)

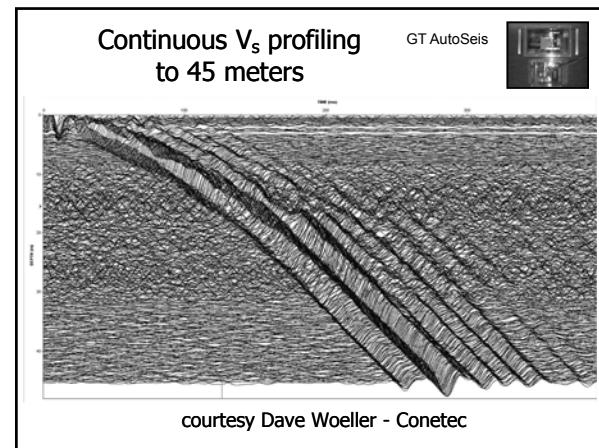
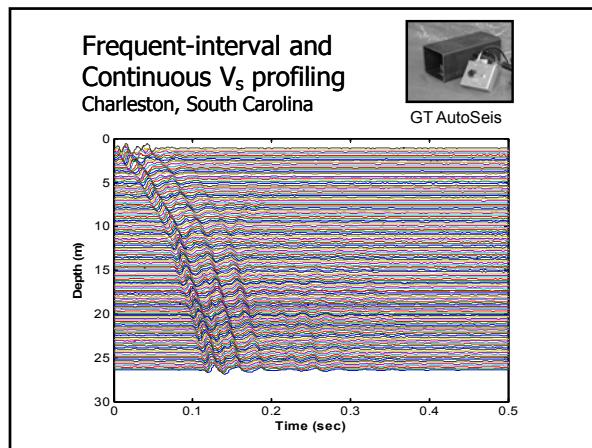
CPT'10 - Full-Flow Penetrometers

- Applicable for very soft soils ($s_u < 10$ kPa)
- Conventional CPT with larger head
- 100-cm² area (vs. the standard 10-cm²)
- Resolution of load cell improved 10-fold
- Correction for net area minimized
- Direct q_{tbar} rather than net (CPT: $q_t - \sigma_v$)

$$s_u = q_{tbar}/10$$
 (Yafrate & DeJong 2006)
- Take readings during push and during pullout to investigate cyclic effects and remoulded strength

Randolph (ISC-2, Porto, 2004)





CPT'10 - Summary Regional Reports



No Confirmation by the French



Will wait for extraterrestrial CPT report by Jim Mitchell

ConeHeads are from ...FRANCE

Parody on "You might just be a redneck"

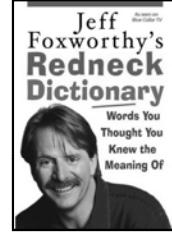
Comedian defines "redneck" as "a person who gloriously lacks of sophistication"

If you come home from the garbage dump with more than you went in with.....

...You might just be a redneck

CPT'10 - Geotechnical Parody - - -

You might just be a ConeHead



Jeff Foxworthy's Redneck Dictionary Words You Thought You Knew the Meaning Of

CPT'10

CPT'10 Parody on "You might just be a redneck"

If you believe that the best means to evaluate N_{60} is from CPT data.....



...then you just might be a ConeHead

CPT'10

CPT'10 Parody on "You might just be a redneck"

If you think our professional image may be tarnished by the field drilling crew



...then you just might be a ConeHead

CPT'10

CPT'10 Parody on "You might just be a redneck"

If you believe our professional image may be improved by using CPT

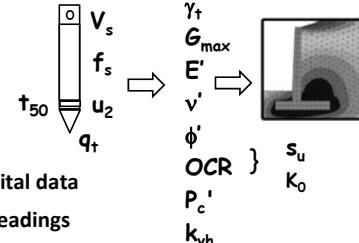


....then you just might just be a ConeHead

CPT'10

If you believe in.....

- Fast
- Economical
- Efficient
- Continuous
- Collection of digital data
- from Multiple Readings
- Logged to your computer



...then you just might be a ConeHead

CPT'10 Parody on "You might just be a redneck"

If you think the Washington monument is a national tribute to cone penetration testing



....then you just might just be a ConeHead

CPT'10

CPT'10 - Summary Regional Reports



Epilogue: CPT'10

What about the Redneck ConeHeads?



Ethan Cargill
ConeTec
Richmond, Virginia

CPT'10 - Summary Regional Reports

	James K. Mitchell Univ. California - Berkeley		Dick Campanella Univ. British Columbia		Peter K. Robertson University of Alberta		Miguel Pando Univ. Mayaguez, Puerto Rico		Miguel E. Ruiz Universidad Nacional de Córdoba, Argentina
					5 Generations of ConeHeads				
					STUDY OF AXIALLY LOADED POST GROUTED DRILLED SHAFTS USING CPT (2005)				

Epilogue: CPT'10 www.cpt10.com

- 4 Keynote Papers, including the 2010 James K. Mitchell Lecture by Tom Lunne
- 9 Regional Reports on CPT
- Panel Discussion sessions headed by:
 - Mark Randolph - Univ. Western Australia
 - Joek Peuchen - Fugro, The Netherlands
 - Peter K. Robertson - Gregg Drilling
- 3 Session Reporters
- 120 Technical Papers
- Closing Talk by Jim Mitchell, Em. Prof. - Virginia Tech
- Workshop on Liquefaction Evaluation by CPT



Tom Lunne and Guy Sanglerat

Epilogue: CPT'10 www.cpt10.com

- Proceedings: 3 Volumes, total 1371 pages Omnipress
- CD version
- Online



Epilogue: CPT'10 www.cpt10.com



2nd International Symposium on Cone Penetration Testing

May 9-11, 2010 • Huntington Beach, California



Welcome
Dates/Deadlines
Program
Registration
Submissions
Destination
Hotel
Transportation
International Travel
Speakers
Exhibitors & Sponsors
Cooperating Organizations
Organizing Committee

Theme

The theme of the Symposium was the solution of geotechnical and geo-environmental problems using the Cone Penetration Test (CPT) and related methods. The emphasis was on the application of practical experience and the application of research results from key international studies. The technical and social program provided an opportunity for meeting new contacts and an exchange of ideas and experience.

ATTENDEE LIST NOW AVAILABLE

Please click the link below to download a PDF with the attendee list of CPT'10.
[Attendee list](#)

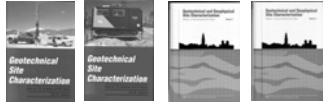
PAPERS ON-LINE

All final papers are now available online for your review. Over 120 papers from 35 countries have been published. In addition 4 keynote lectures, 9 regional reports and 3 session reports are available.
[Papers On-line](#)

PRESENTATIONS ON-LINE

4th International Conference on Site Characterization - 2012 Recife, Brazil (ISC-4)

- Professor Roberto Quental Coutinho
- Federal University of Pernambuco
- Brazilian Soc. Soil Mechanics & Geotechnical Engineering
- TC 102/TC 16 - In-Situ Testing: webforum.com/tc16

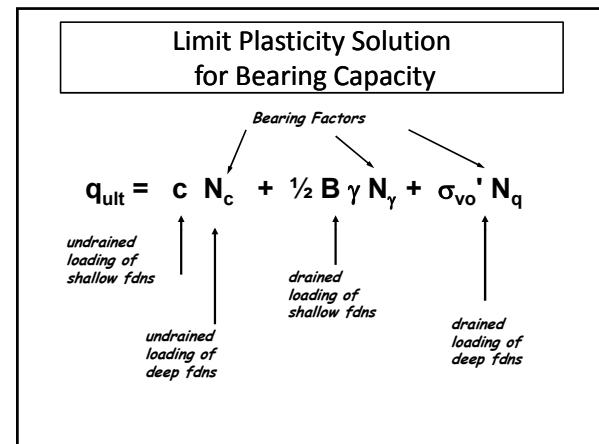



www.isc-4.com

GeoEngineering Design Using the CPT 

Direct CPT Method for Shallow Footings (2009)

Paul W. Mayne, PhD, P.E.
Georgia Institute of Technology



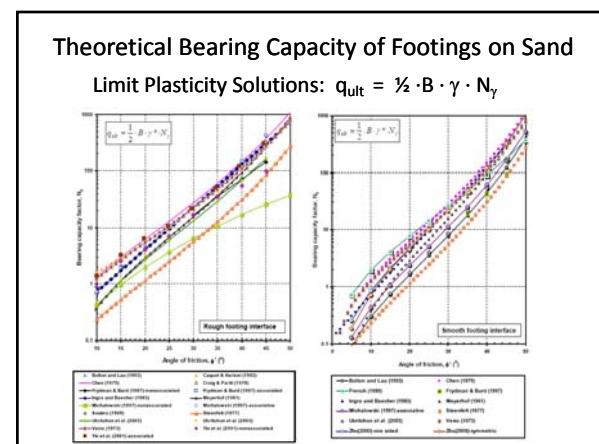
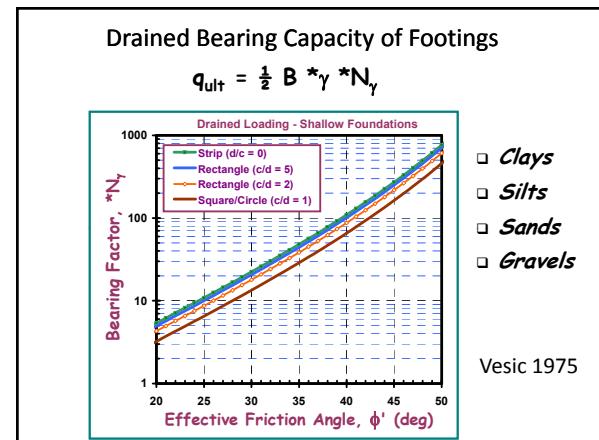
Bearing Capacity of Spread Footings

UNDRAINED LOADING ($\Delta V/V_o = 0$)

$$q_{ult} = *N_c s_u \quad \begin{matrix} \triangleright \text{Strip Footing: } *N_c = 5.14 \\ \triangleright \text{Square/Circle: } *N_c = 6.14 \end{matrix}$$

(a) CSSM: $s_u = \frac{1}{2} \sin \phi' \sigma_{vo}' \text{OCR}^{(1-C_s/C_c)}$
 (b) SHANSEP: $s_u = 0.22 \text{OCR}^{0.80} \sigma_{vo}'$
 (c) NGI/UIUC: $s_u = 0.22 \sigma_p'$ in soft clays with $\text{OCR} < 2$

DRAINED LOADING ($\Delta u = 0$)

$$q_{ult} = \frac{1}{2} B * \gamma * N_\gamma$$


Theoretical Bearing Capacity of Footings on Sand

$$\text{Ultimate stress: } q_{\text{ult}} = \frac{1}{2} \cdot B \cdot \gamma \cdot N_y$$

However N_y has been shown to decrease with B

- Golder (1941)
- de Beer (1965)
- Vesic (1973)
- Habib (1974)
- Ovesen (1975)
- Yamaguci et al. (1977)
- Ingra & Baecher (1982)
- Kimura et al. (1985)
- Hettler & Gudehus (1988)
- Bolton & Lau (1989)
- Shiraishi (1990)
- Tatsuoka et al. (1991)
- Kusakabe et al. (1992)
- Tatsuoka et al. (1994)
- Herle & Teichman (1997)
- Ueno et al. (1998)
- Zhu, Clark & Phillips (2001)
- Lee & Salgado (2005)
- Cerato & Lutenegger (2007)
- Kumar and Khatri (2008)
- Salgado (2008)
- Mase & Hashiguchi (2009)
- Teichman & Górska (2010)

Theoretical Bearing Capacity of Footings on Sand

However N_y has been shown to decrease with B

Model and full-scale footings (de Beer 1965)

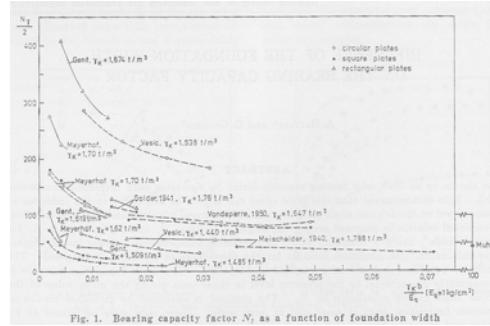
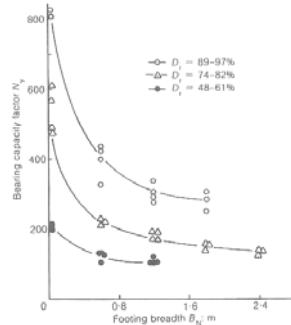


Fig. 1. Bearing capacity factor N_y as a function of foundation width

Theoretical Bearing Capacity of Footings on Sand

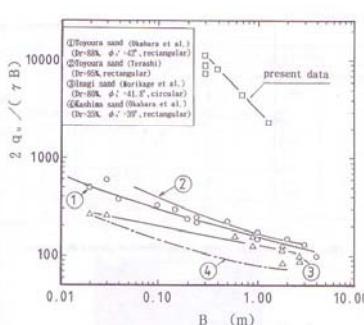
However N_y has been shown to decrease with B



Footing size effects on bearing factor N_y
(a) centrifuge experiments
(Kimura et al. 1985);

Theoretical Bearing Capacity of Footings on Sand

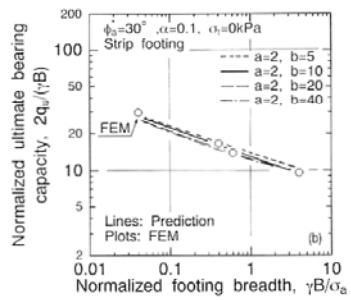
However N_y has been shown to decrease with B



Footing size effects on bearing factor N_y
(b) small and large scale footing tests
(Kusakabe et al. 1992)

Theoretical Bearing Capacity of Footings on Sand

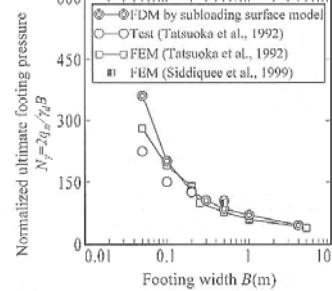
However N_y has been shown to decrease with B



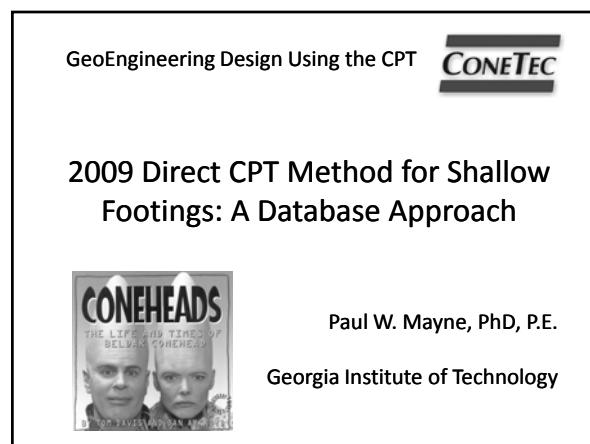
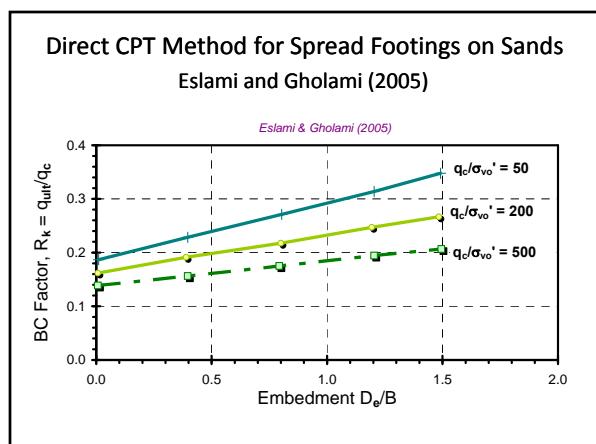
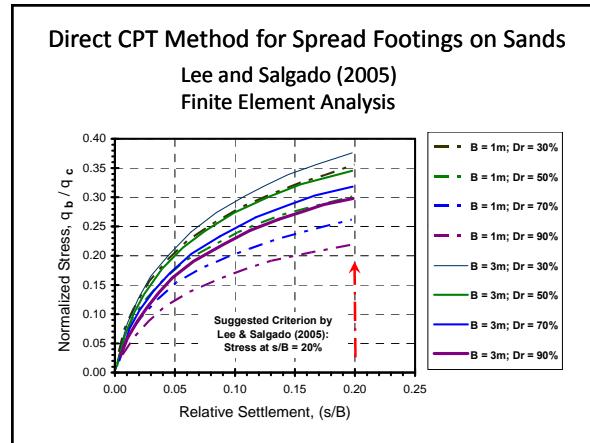
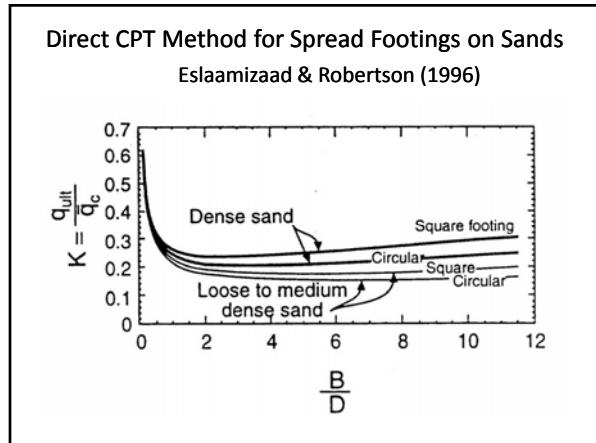
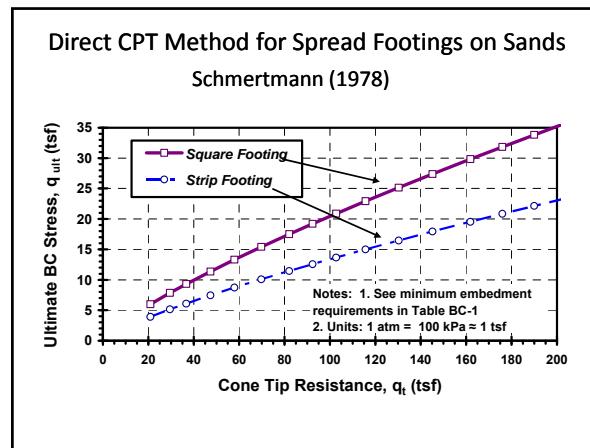
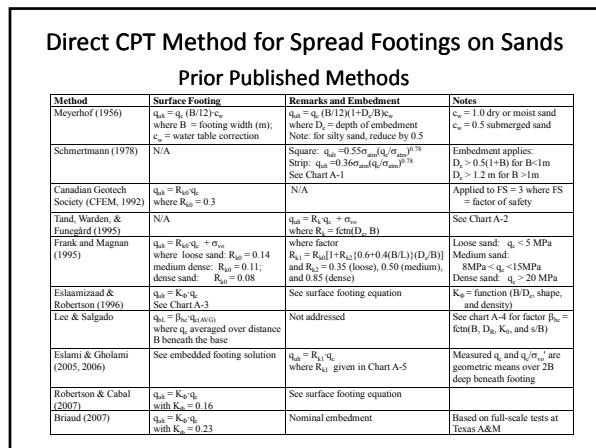
Footing size effects on bearing factor N_y
(c) Finite Element Analyses
(Ueno et al. 1998).

Theoretical Bearing Capacity of Footings on Sand

However N_y has been shown to decrease with B



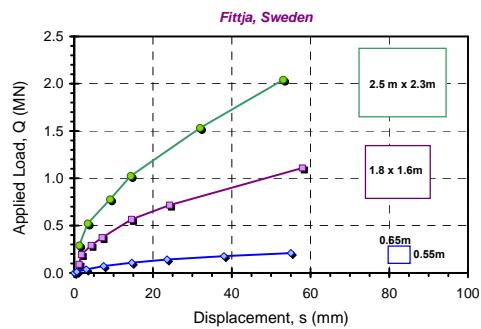
Footing size effects on bearing factor N_y
(d) finite difference simulations
(Mase & Hashiguchi, 2009).



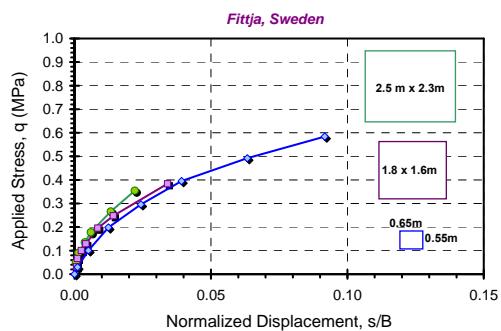
Direct CPT Method for Spread Footings on Sands

- New 2009 Method - A Database Approach
- 31 Large Footings ($0.5 \text{ m} < B < 6 \text{ m}$)
- 13 Sand Sites - Clean to slightly silty Sands
- All sites tested by CPT
- Characteristic Load-Displacement Curve (Fellenius, 1994; Briaud & Gibbens, 1994; Lutenegger & Adams, 1998; 2003; Briaud 2007)
- Stress (q) vs. normalized displacement (s/B)

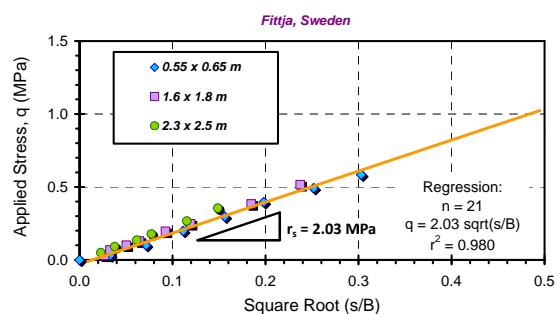
Direct CPT Method for Spread Footings Characteristic Stress-Normalized Displacement Curves



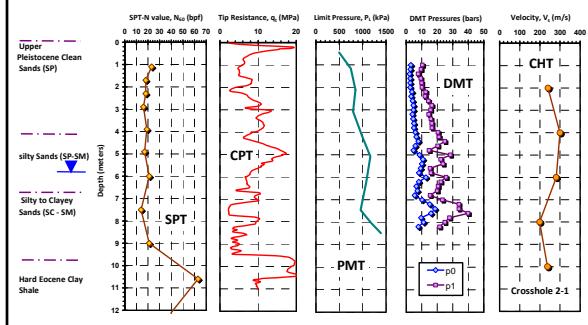
Direct CPT Method for Spread Footings Characteristic Stress-Normalized Displacement Curves



Direct CPT Method for Spread Footings Characteristic Stress-Normalized Displacement Curves

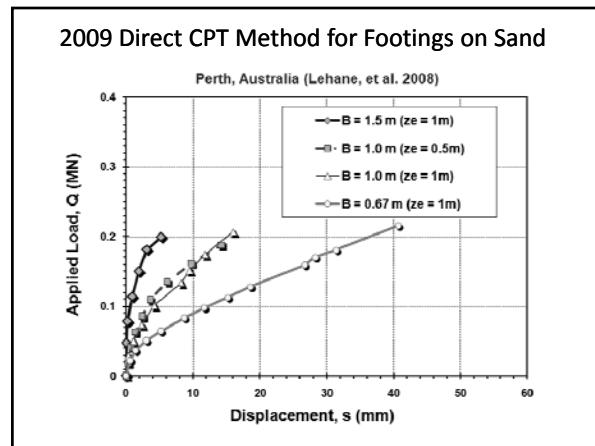
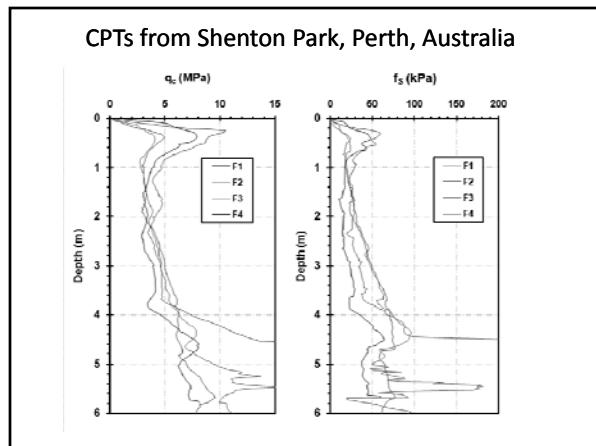
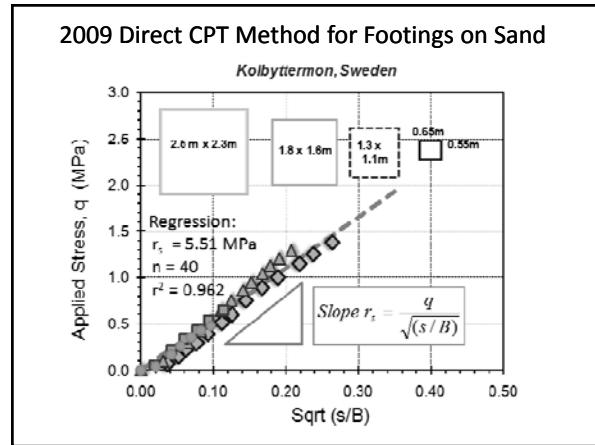
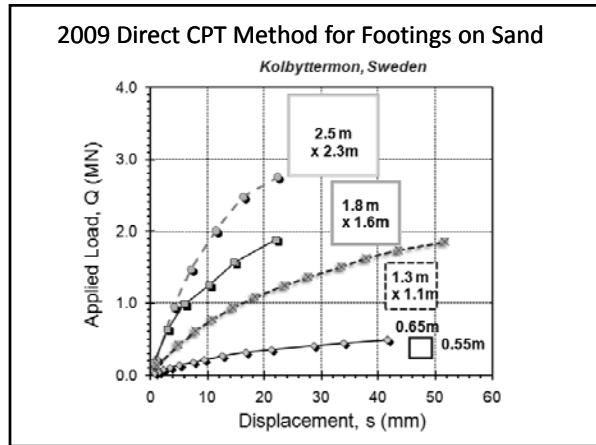
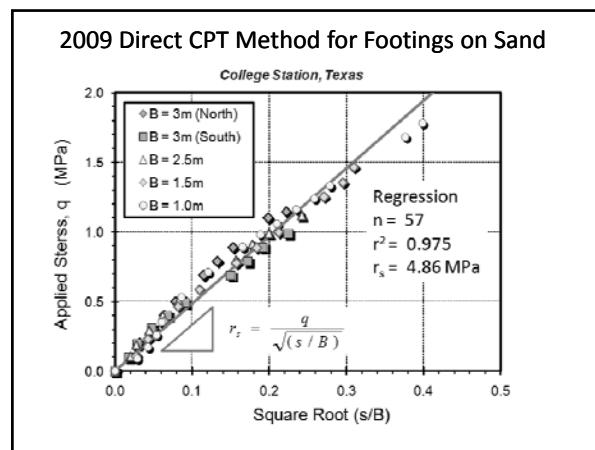
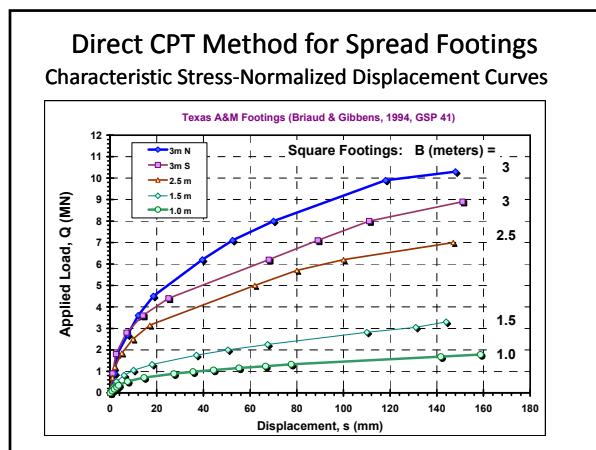


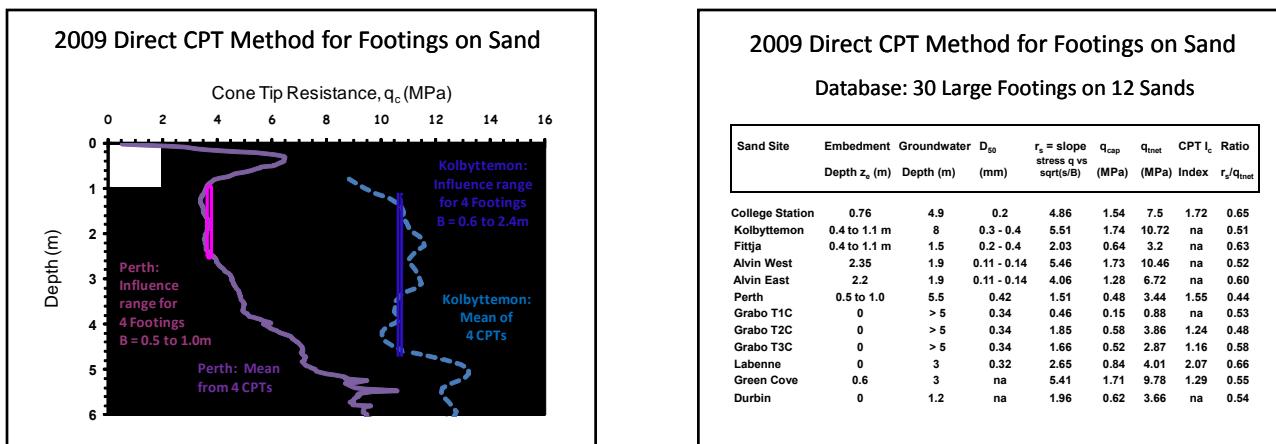
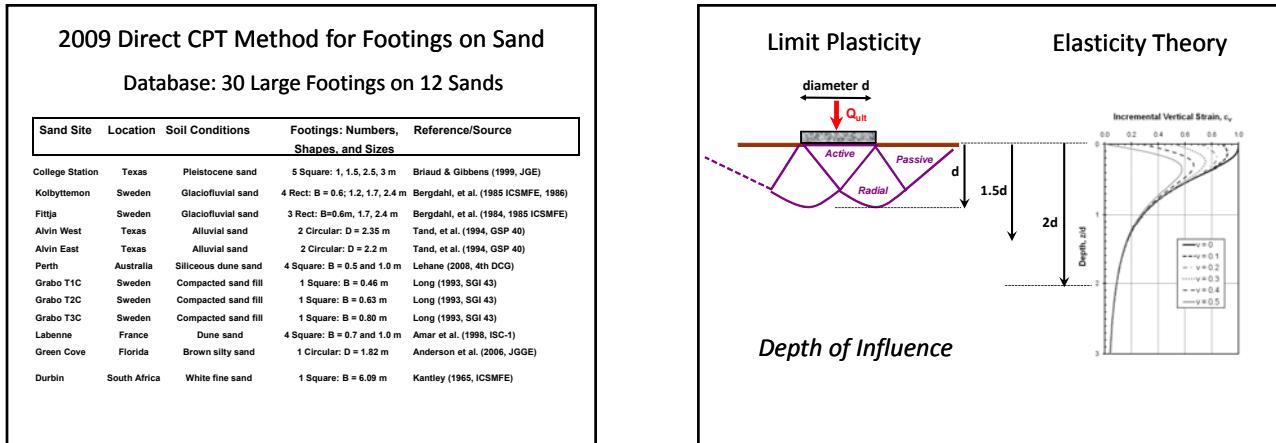
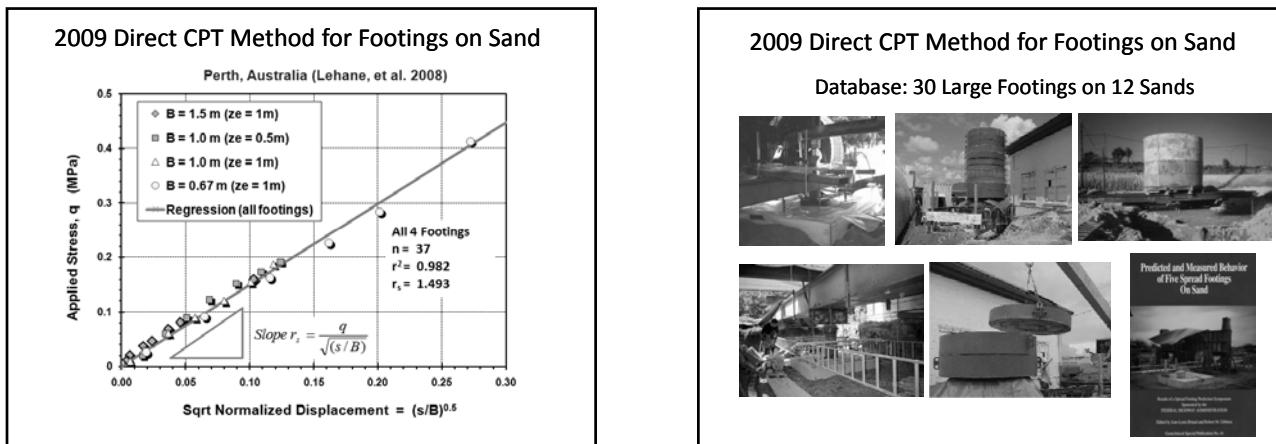
Texas A&M Sand Site US National Geotechnical Test Site (Briaud & Gibbens, 1999; Briaud, 2007, ASCE JGGE)

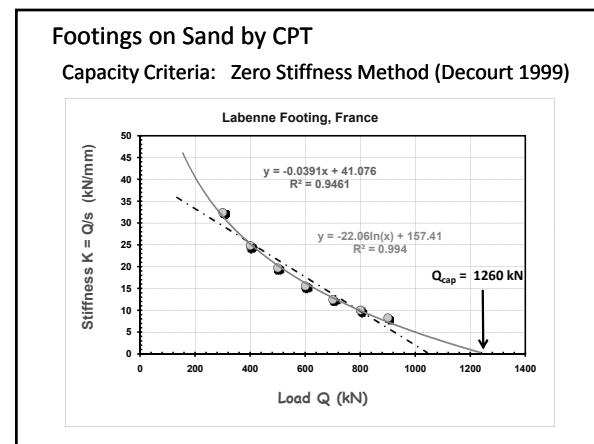
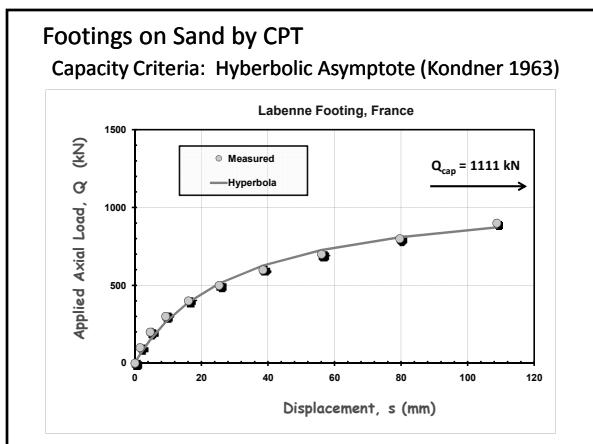
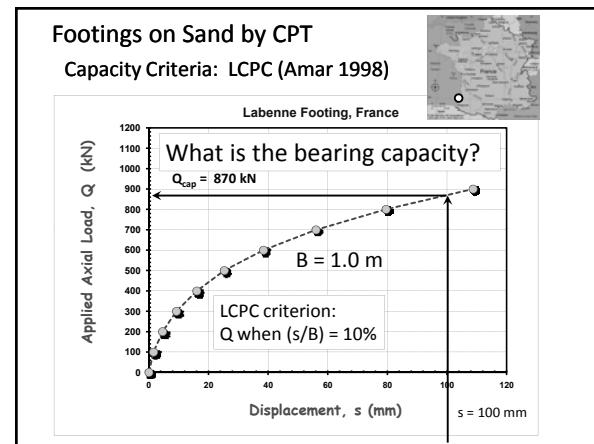
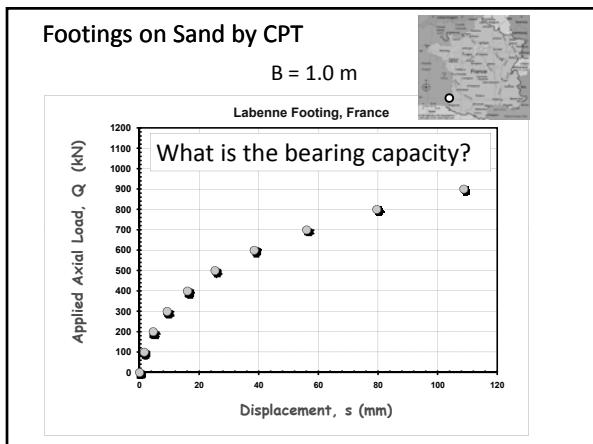
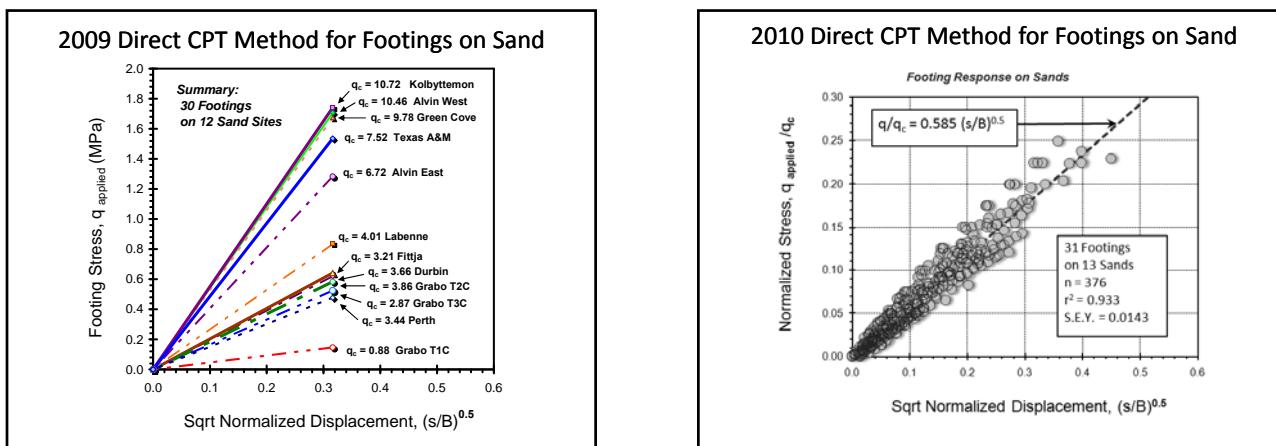


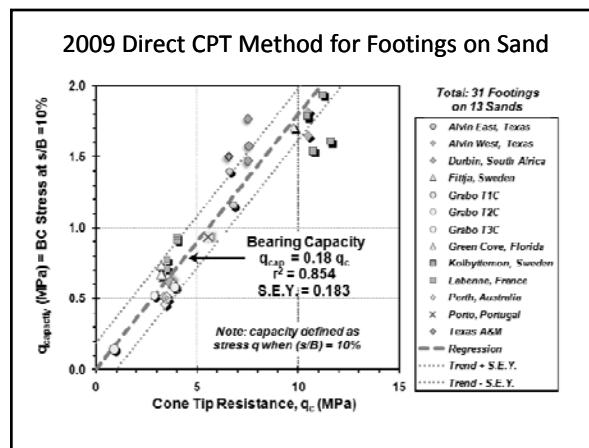
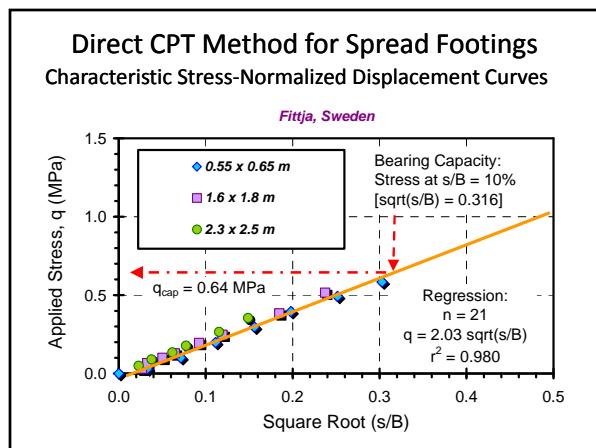
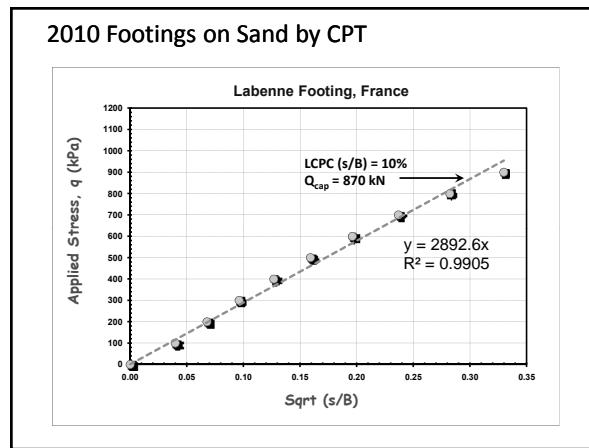
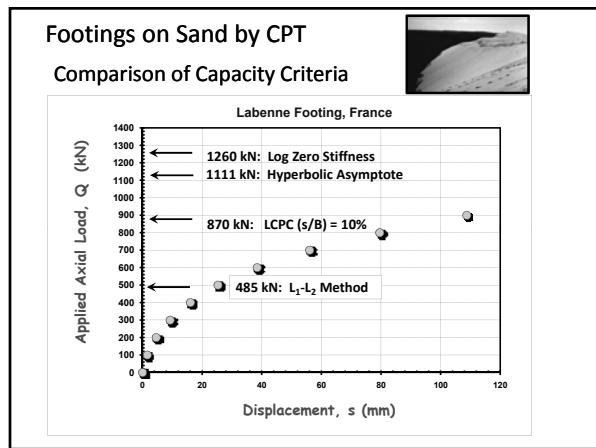
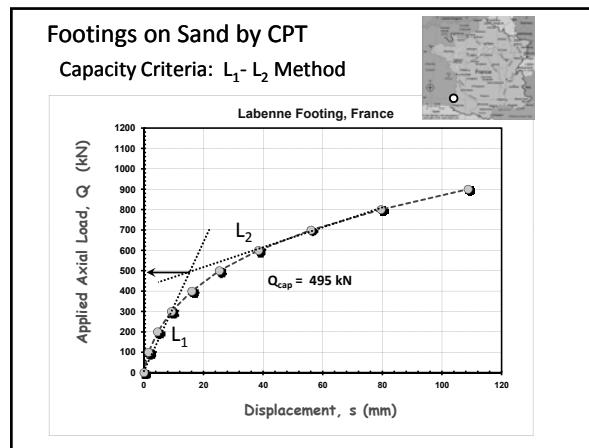
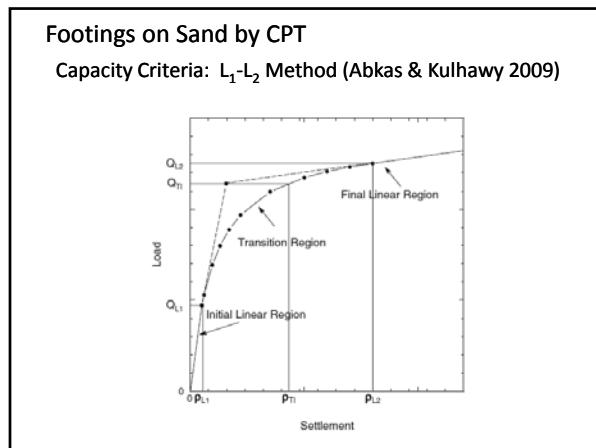
Texas A&M Sand Site National Geotech Experimentation Site











Direct CPT Footing Method - Kolby Site

CPT Direct Method for Characteristic Load-Displacement Response of Footings on Sand

Kolby Footing, Sweden (Bergdahl et al. 1986)

B (m) = 2.4 (equivalent square)

q_c (kPa) = 10000 (mean over 1B beneath base)

Characteristic Load-Displacement Curve

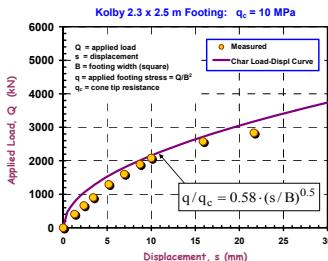
s/B

q/q_c

s (mm)

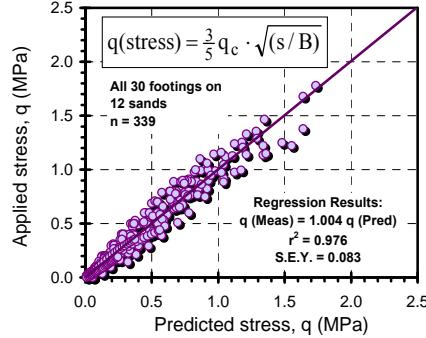
q (kPa)

Q (kN)



2010 Direct CPT Method for Footings on Sand

Footing Response on Sands



Rhymes with Orange - by Hilary Price



2009 Direct CPT Method for Footings on Silts

Silt Site	Location	Soil Conditions	Footings: Numbers, Shapes, and Sizes	Reference Source
-----------	----------	-----------------	--------------------------------------	------------------

Jossigny France Soft clayey silt 2 Square: B = 1 m Amar et al. (1998, ISC-1)

Tornhill Sweden Glacial Baltic till 3 Square: B = 0.5, 1, and 2 m Larsson (2001, SGI R-59)

Vagverket Sweden Stiff medium silt 3 Square: B = 0.5, 1, and 2 m Larsson (1997, SGI R-54)

Vattahammar Sweden Brn-Gry layered silt 3 Square: B = 0.5, 1, and 2 m Larsson (1997, SGI R-54)



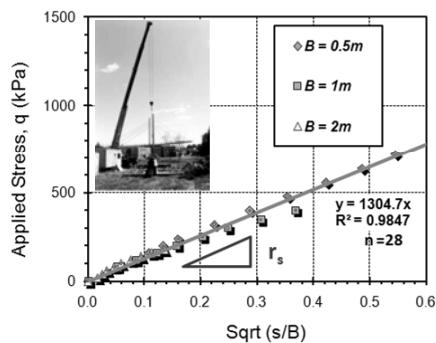
Footing Load Tests at Tornhill (Larsson, 2001)

Vagverket (Larsson, 1997)

Vagverket (Larsson, 1997)

2009 Direct CPT Method for Footings on Silts

Vagverket Footings



2009 Direct CPT Method for Footings on Silts

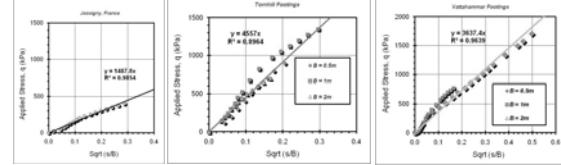
Silt Site	Emb. z _s (m)	GWT (m)	PI (%)	r_s = Slope	q_{cap} (MPa)	q_{inst} (MPa)	CPT I _c	Ratio r_s/q_{cap}
q vs $\text{sqrt}(s/B)$ (MPa)								

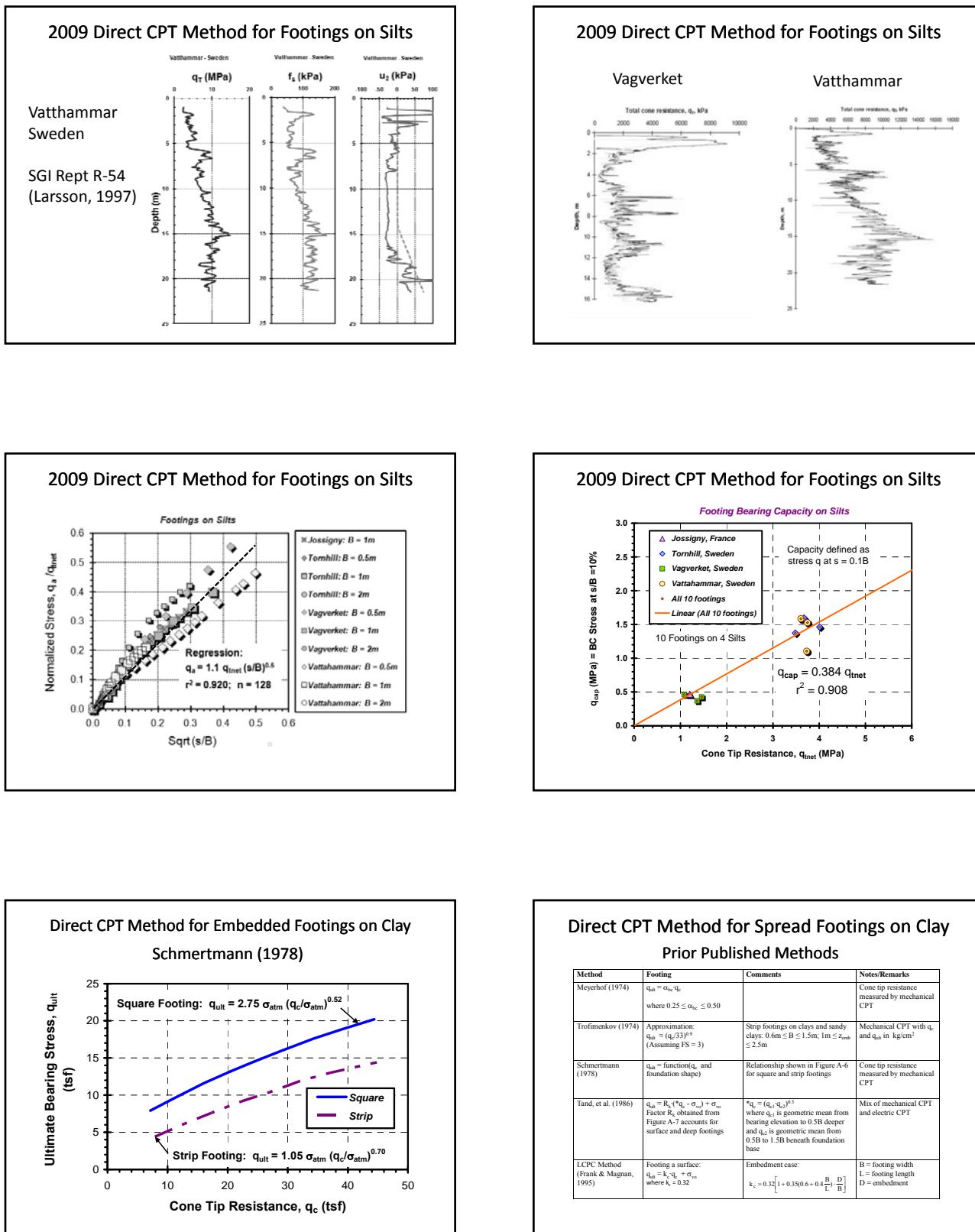
Jossigny 0 and 0.5 0.1 14 1.49 0.47 1.20 na 1.24

Tornhill 1.5 2 m 17 4.56 1.44 3.20 2.30 1.43

Vagverket 2 Artesian +2.3 9 1.30 0.41 1.01 2.14 1.29

Vattahammar 1.5 > 11 m 10 3.64 1.15 3.67 2.19 0.99





Direct CPT Method for Spread Footings

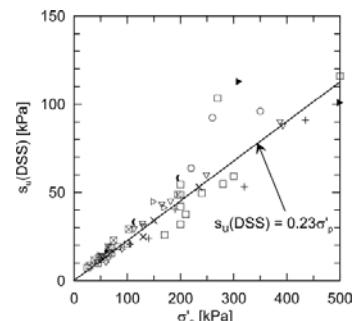
Tand, et al. (In-Situ '86, GSP 6)

Tand, K.E., Fenner, E.G. and Brajard, J.A. (1986). BC of footings on clay: CPT method. In: *In-Situ 86*, pp. 1017

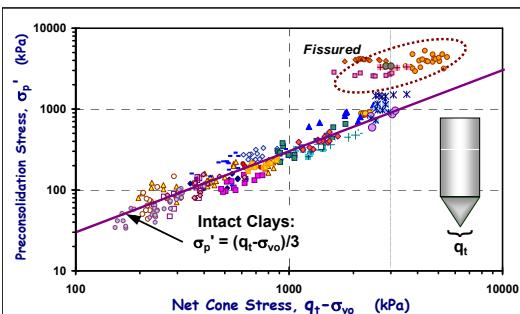
Number	Site and Location	Reference	Soil Conditions	Pf (kN)	Ext D (m)	z _e (m)	Type	q _{ult} (kPa)	q _{vo} (kPa)	q _{sp} (kPa)	q _{net} (kPa)	Penrometer
A	Texas City	Tand, et al. (1986), ASCE GSP 6	Jointed Beaumont clay	25	0.58	1.5	C	650	25.5	1130	1130	Electric CPT
B1	Texas City	Tand, et al. (1986), ASCE GSP 6	Jointed Beaumont clay	25	0.58	1.5	C	310	620	25.5	595	Electric CPT
B2	Texas City	Tand, et al. (1986), ASCE GSP 6	Jointed Beaumont clay	na	0.58	1.5	C	520	980	25.5	955	Electric CPT
C	Abilene, TX	Tand, et al. (1986), ASCE GSP 6	Jointed Beaumont clay	40	0.58	1.5	C	665	1600	25.5	1574	Electric CPT
D1	Houston, TX	Shahri et al. (1988), Univ. Houston	Jointed Beaumont clay	45	0.58	1.5	C	1165	2400	25.5	2400	Electric CPT
D2	Univ. Houston, TX	Shahri et al. (1988), Univ. Houston	Jointed Beaumont clay	45	0.76	6.4	C	1220	2700	108.8	2991	Electric CPT
E	Houston, TX	Shahri et al. (1988), Univ. Houston	Jointed Beaumont clay	45	2.2	2.2	C	780	2380	37.4	2343	Electric CPT
F	Rangsit, Thailand	Brand, et al. (1972) ASCE Earth- quake Resistant Design	Soft Bangkok clay	42	0.6 to 1.0	1.5	C	150	460	25.5	250	Electric CPT
G	Haga, Norway	Andersen & Stenhammar (1982), Vudbe et al. (1979) 6th Asian Reg. Conf.	Stiff OC clay	15	1.12	0	A	340	770	0	770	Electric CPT
I	Karapur, India	Axford, et al. (1979)	Stiff sandy clay	16	0.3 to 0.6	1	C	420	800	17	783	Mech. CPT
J	UK	Ward, et al. (1985), Geotechnique 1985, Vol. 32, No. 2, 1985	Fissured London clay	40	0.15	0	C	1020	2580	0	2580	Mech. CPT
J2	UK	Ward, et al. (1985), Geotechnique 1985, Vol. 32, No. 2, 1985	Fissured London clay	40	0.15	0	C	2440	6300	0	6300	Mech. CPT
K	Adelaide, Australia	Geotechnique	Firm silty clay	15	1.14	3	C	355	860	51	809	Mech. CPT
L1	Germany	Jelink et al. (1977) 9th ICSCFE	Hard lacustrine clay	22	0.88	10.2	C	3400	6500	173.4	8327	Mech. CPT
L2	Germany	Jelink et al. (1977) 9th ICSCFE	Hard lacustrine clay	22	1.8	10	C	3350	6200	170	6030	Mech. CPT
M1	London Clay, UK	Mariand-Quammen (1982) ESOPT	Stiff fissured clay	na	0.29	6	C	1200	2400	102	2298	Electric CPT
M2	London Clay, UK	Mariand-Quammen (1982) ESOPT	Stiff fissured clay	na	0.29	6	C	2000	2900	102	2794	Electric CPT
New	London, UK	London 2003 ECI Guidelines	Very soft clay "weak"	30	2.2	2.2	B	108	108	0	108	SCPTU
New	Shetland, Scotland	Jardine, et al. (1995, 2000), Geotechnique	Soft fissured clay	37	0.48	3	B	168	168	0	168	SCPTU
New	Shetland, Scotland	Schmid (1992), Wicht Min Symp	Soft fissurable clay	70	9.44	0	B	86	86	0	86	SCPTU
New	Covden, UK	Powell (2003) Char & Engng Prop	Stiff clay till	19	0.865	9	C	912	2105	242	1863	SCPTU

Undrained Shear Strength vs Preconsolidation Stress

DeGroot, Lunne, & Tjelta (2010, ISFOG-II)



Preconsolidation Stress of Clays from CPT (Kulhawy & Mayne, 1990)

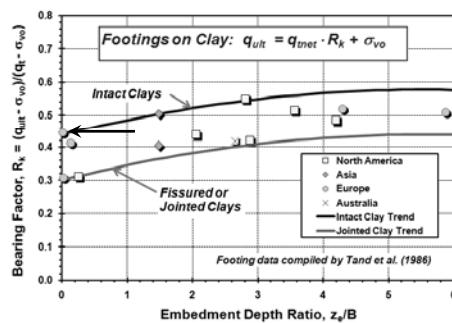


Footings on Soft Clay

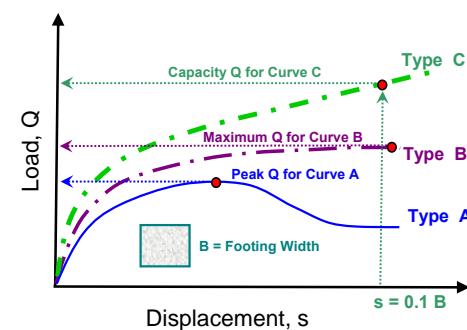
- Bearing Capacity Theory: $q_{ult} = *N_c \cdot s_u$
- Backcalculated Failure Case Histories (Bjerrum 1972; 1973) and Corrected Vane Data (Mesri 1975):
Mobilized shear strength: $s_u \approx 0.23 \sigma'_p$
- Approx. SCE-CSSM approach: $\sigma'_p = 0.33 (q_t - q_{vo})$
- Combine: $q_{ult} = (6.14)(0.23)(0.33) q_{tnet}$
- Relationship: $q_{ult} \approx 0.46 q_{tnet}$

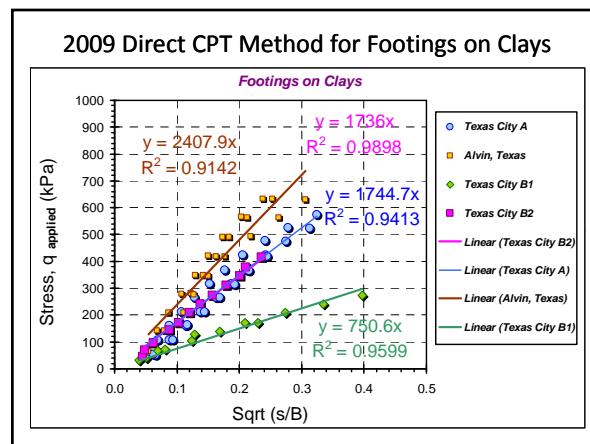
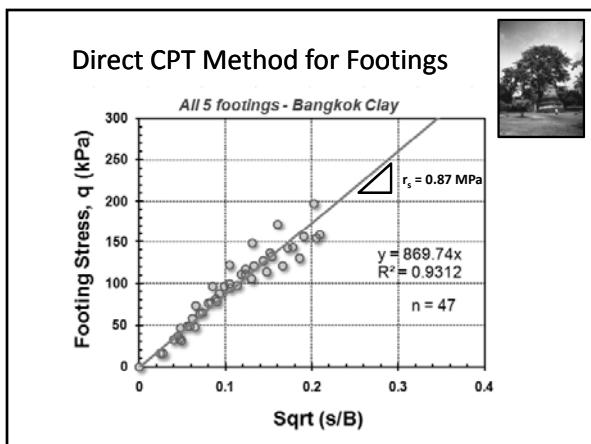
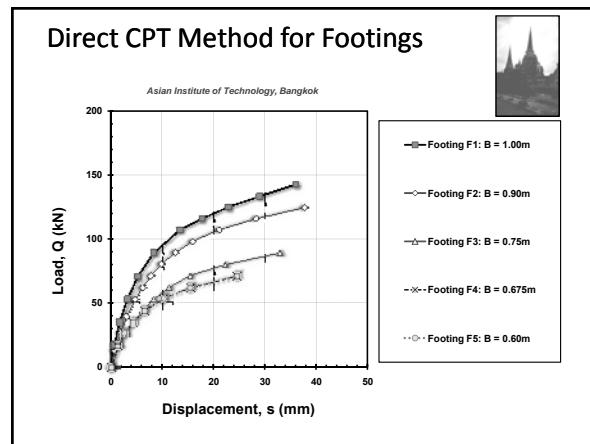
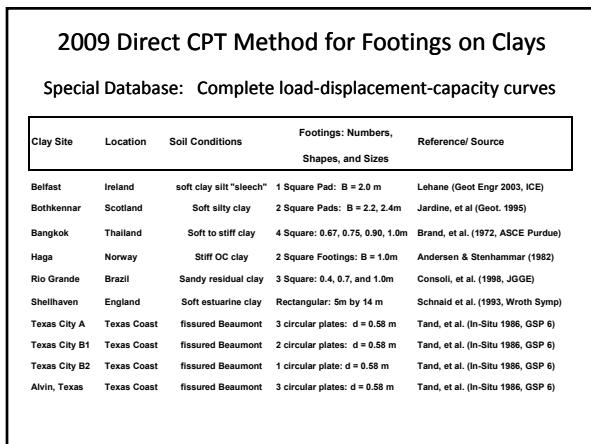
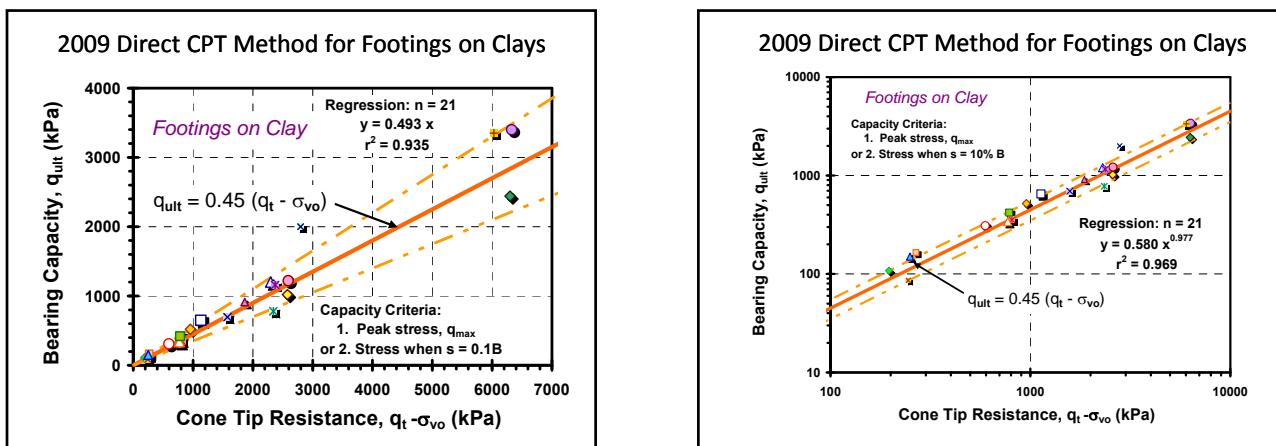
Direct CPT Method for Spread Footings

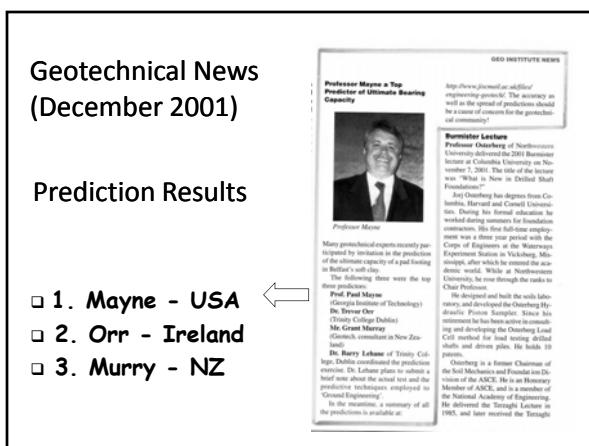
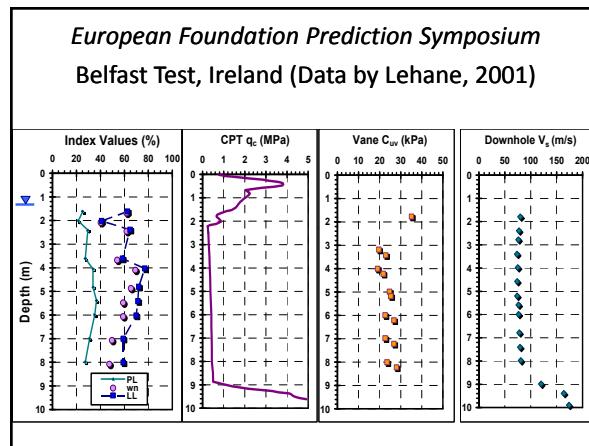
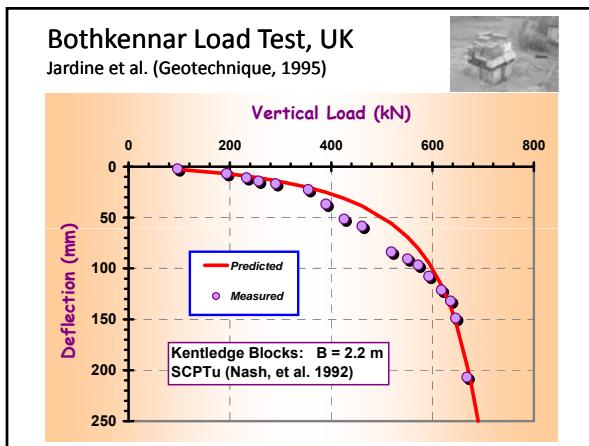
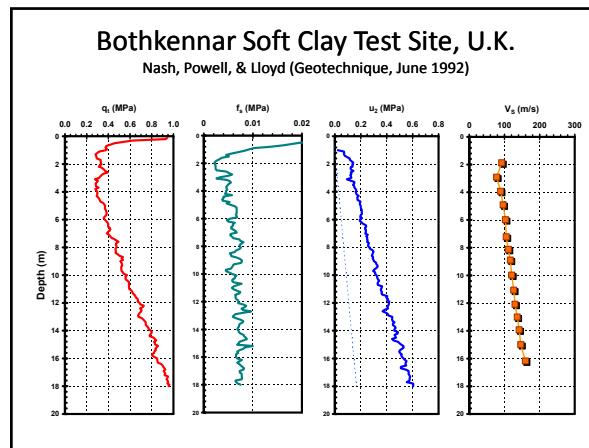
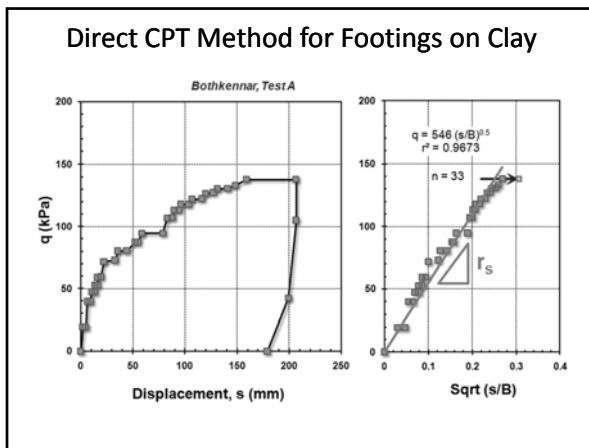
Tand, et al. (In-Situ '86, GSP 6)

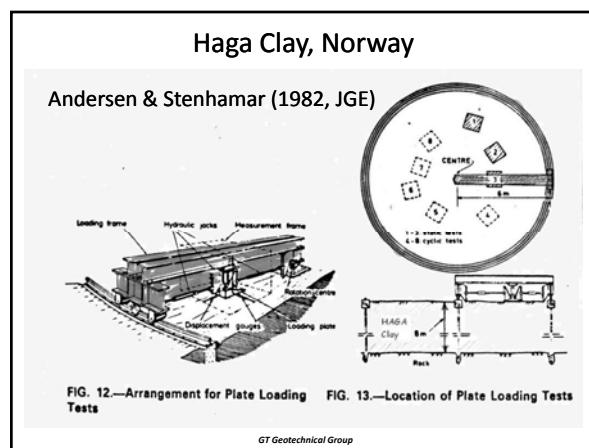
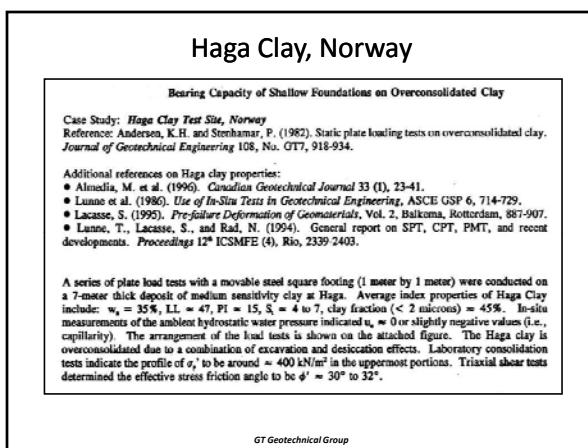
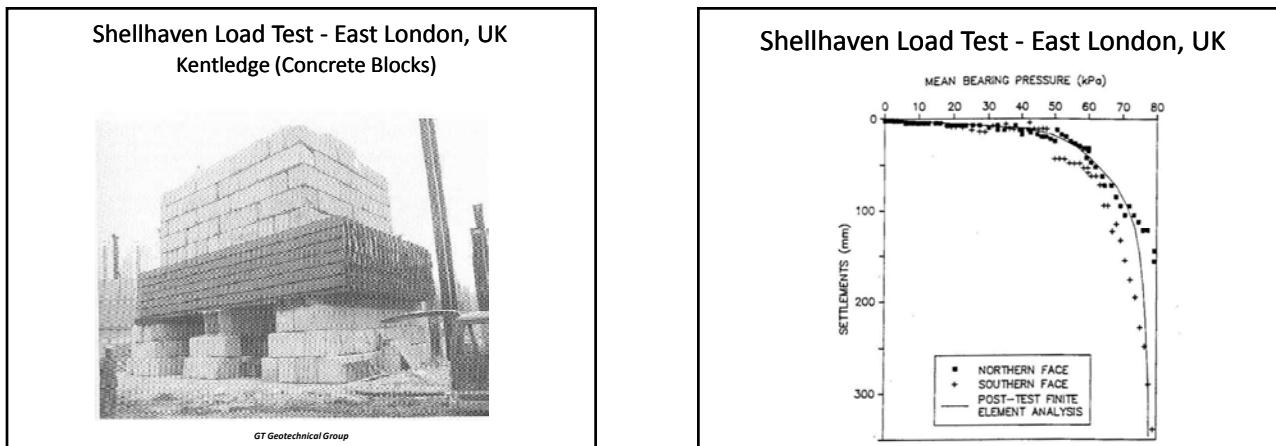
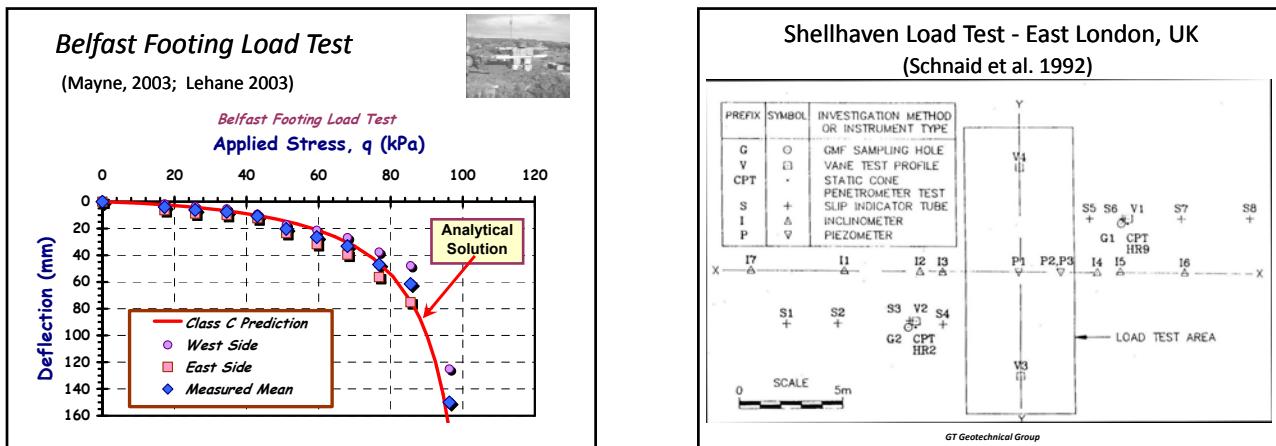


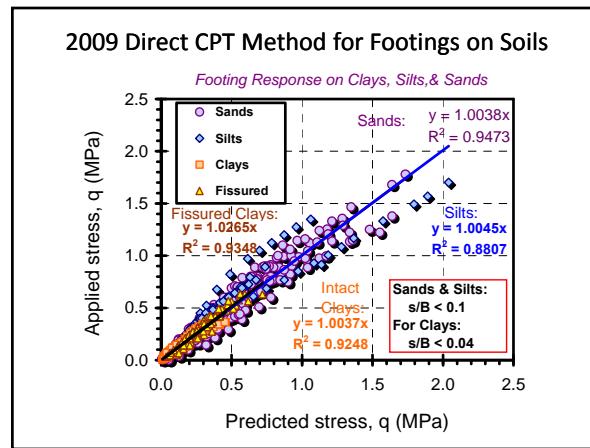
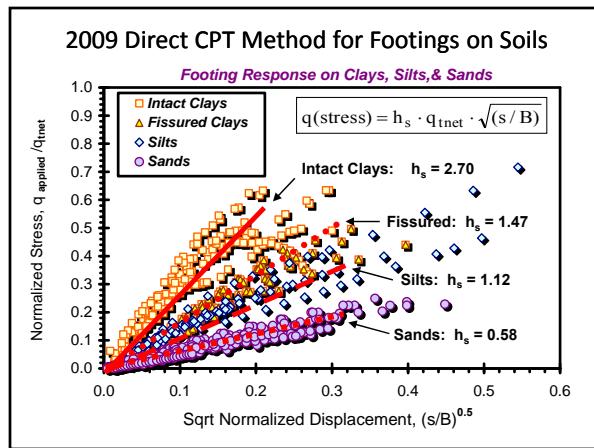
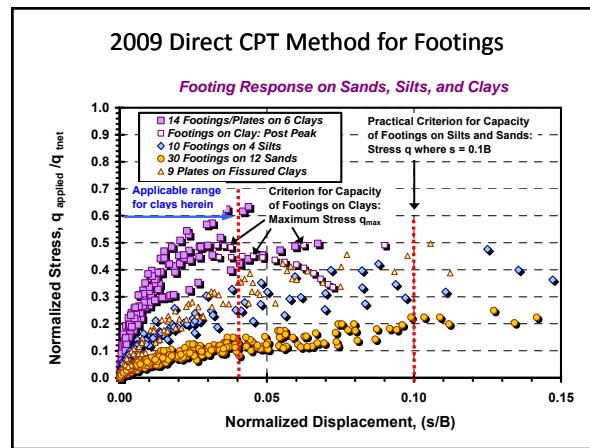
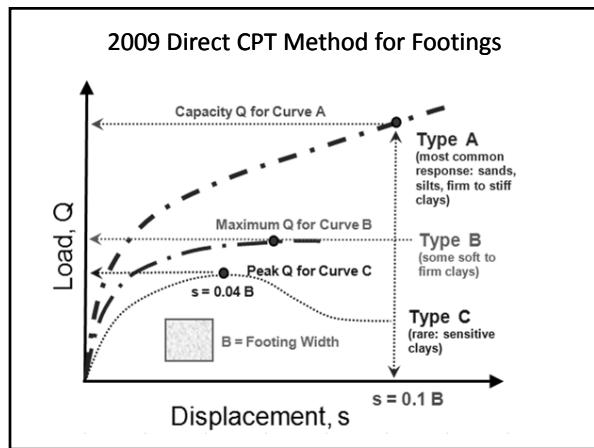
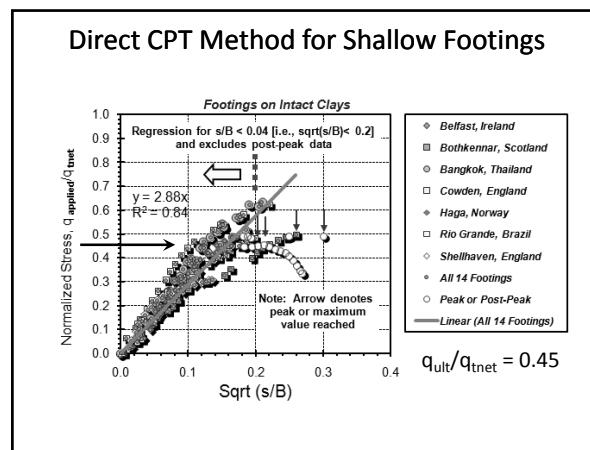
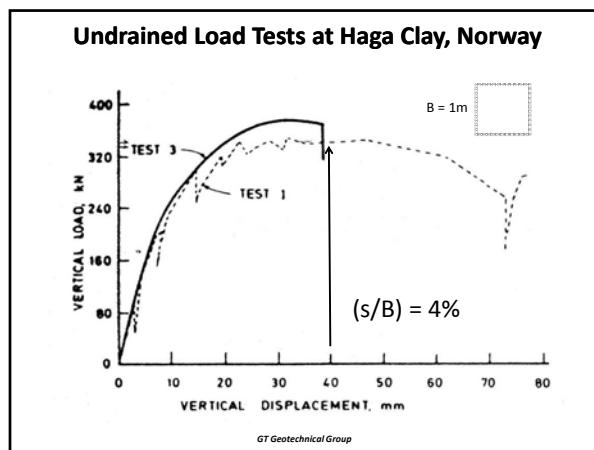
2009 Direct CPT Method for Footings on Clays Foundation Response Curves (after Kulhawy 2004)









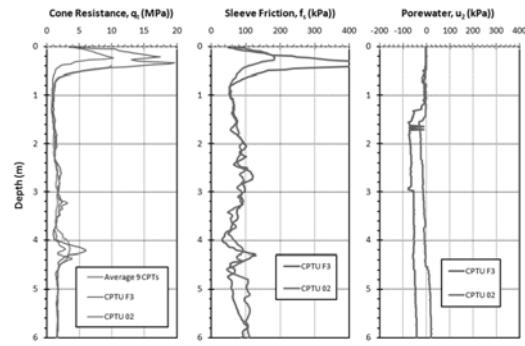


2010 Direct CPT Method for Footings on Soils

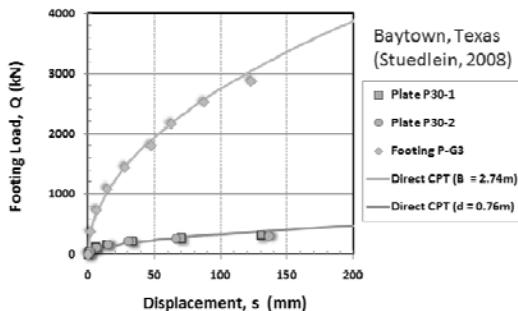


- New case study (Stuedlein, 2008)
- Baytown, Texas
- Stiff fissured Beaumont Clay
- 2 Plate Load Tests ($d = 0.76 \text{ m}$)
- 1 Large Square Footing ($B = 2.74 \text{ m}$)
- 9 CPTu soundings

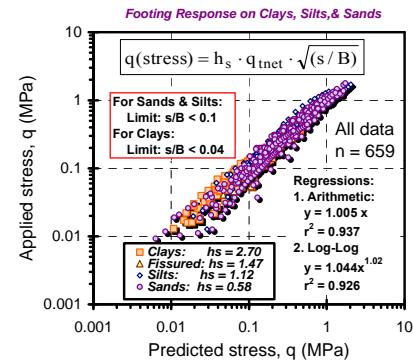
CPTUs at Baytown, Texas (Stuedlein, 2008)



2010 Direct CPT Method for Footings on Soils



2009 Direct CPT Method for Footings on Soils



Geostratification by CPTu

Soil Behavioral Type Interpretation of CPTu Soundings (Robertson, CGJ, 1990)

Define Normalized Piezocone Parameters:

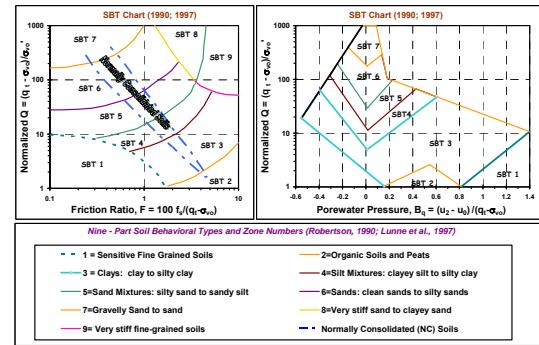
Normalized Tip Resistance: $Q = (q_t - \sigma_{vo})/\sigma_{vo}'$

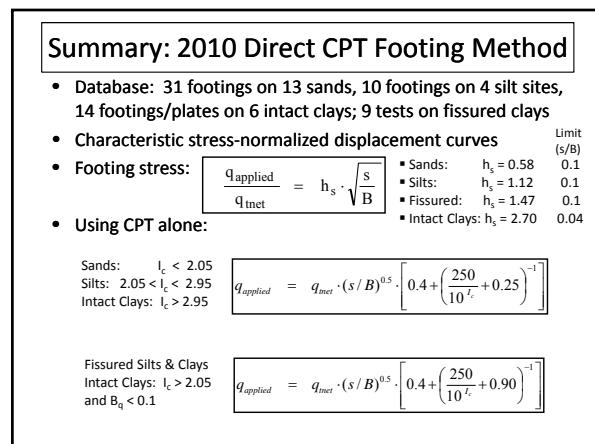
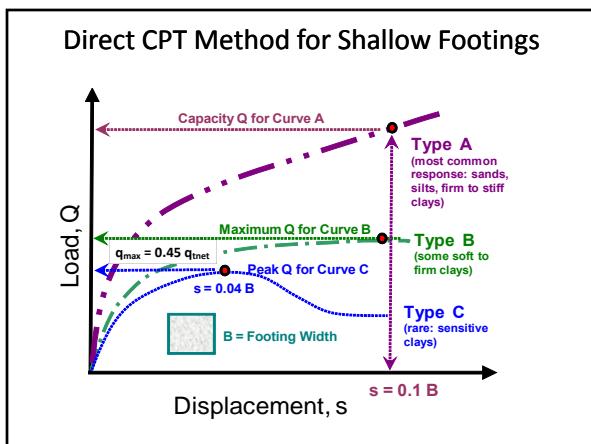
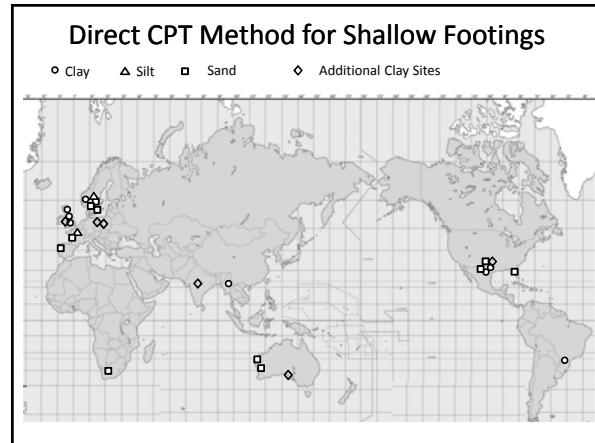
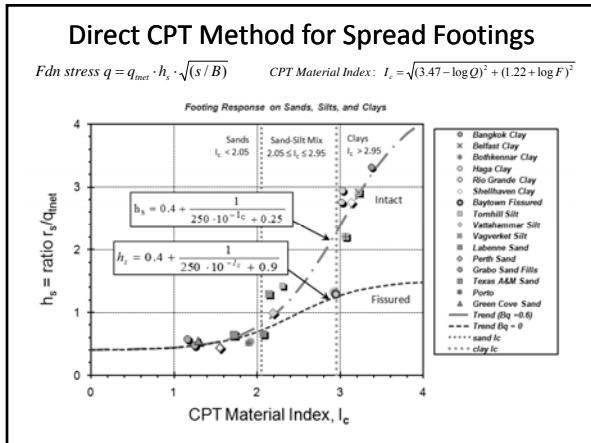
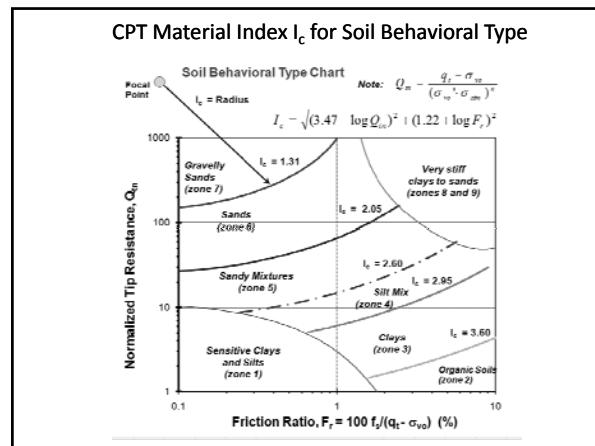
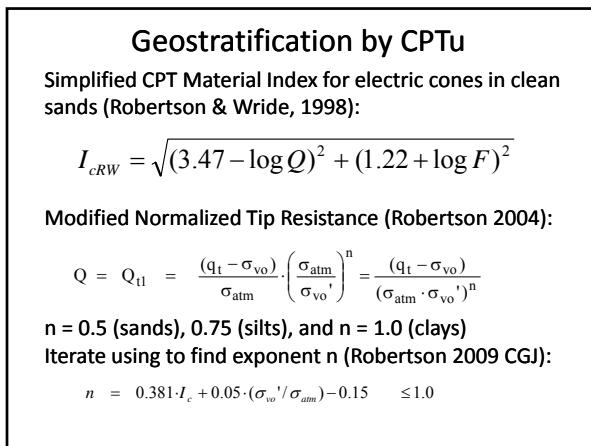
Normalized Sleeve Friction: $F = 100 f_s/(q_t - \sigma_{vo})$

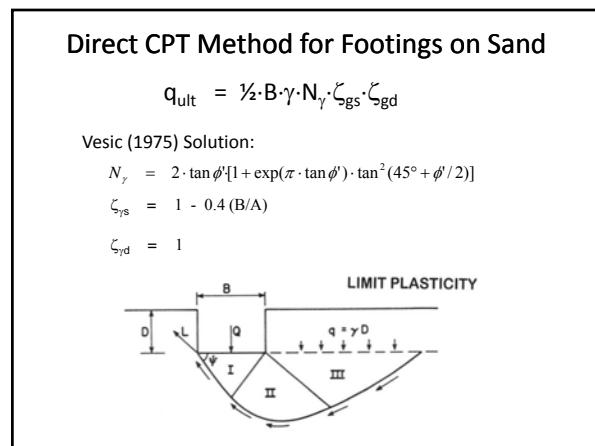
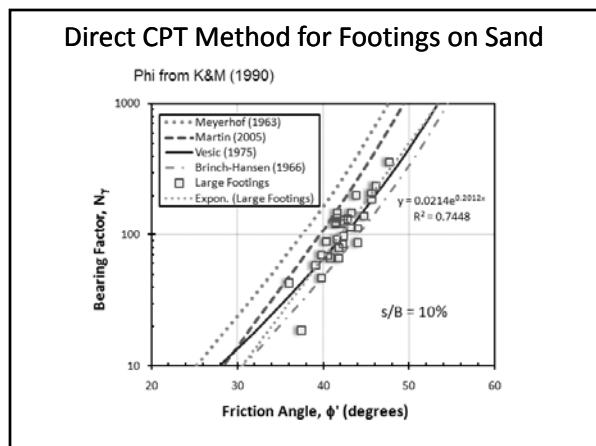
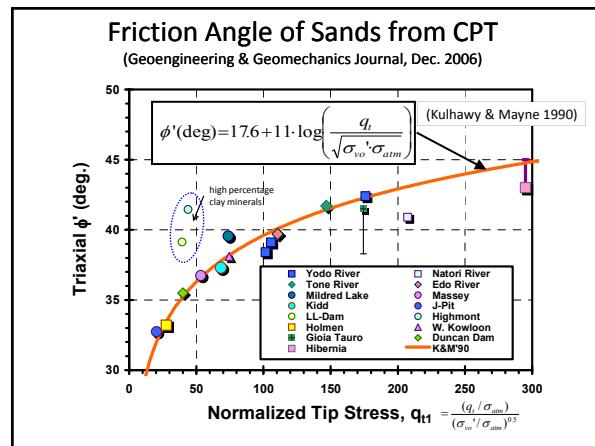
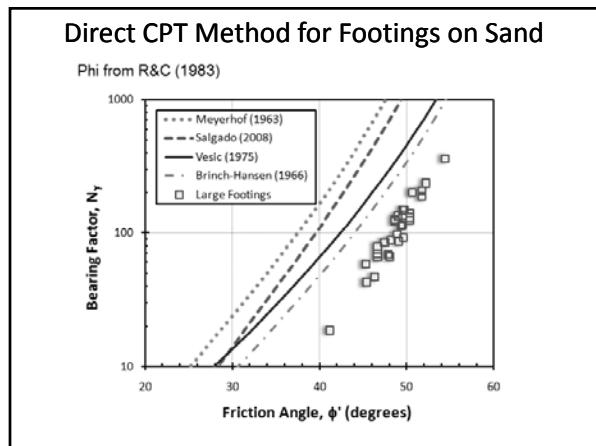
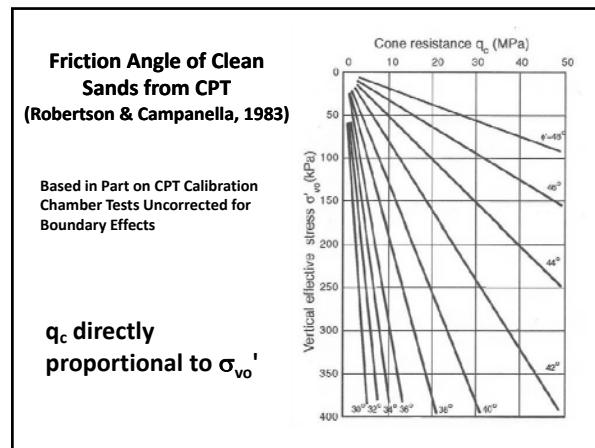
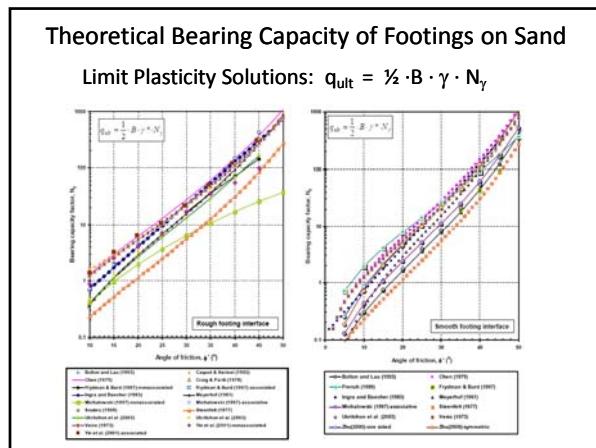
Normalized Porewater Pressure: $B_q = (u_2 - u_0)/(q_t - \sigma_{vo})$

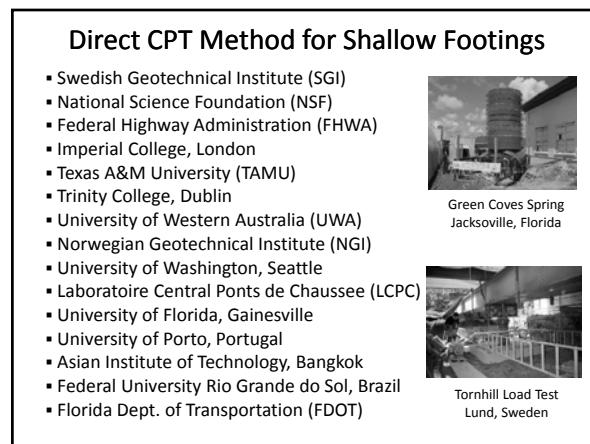
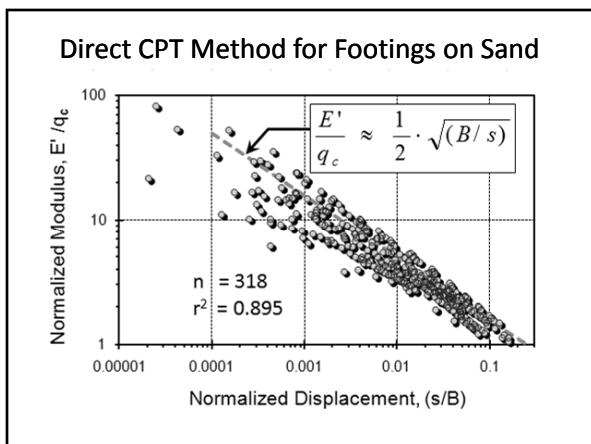
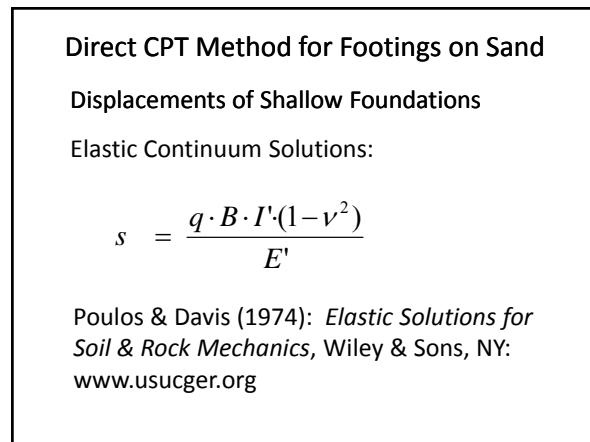
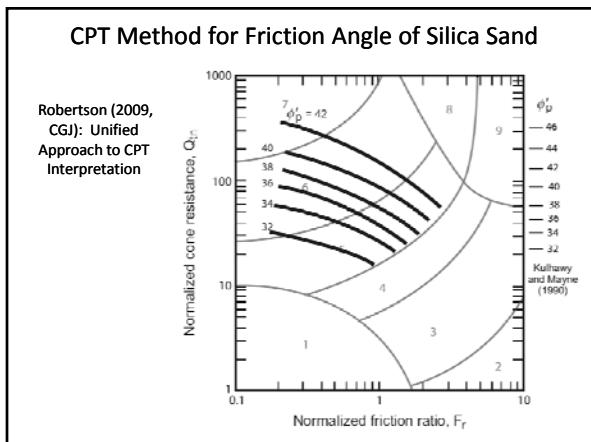
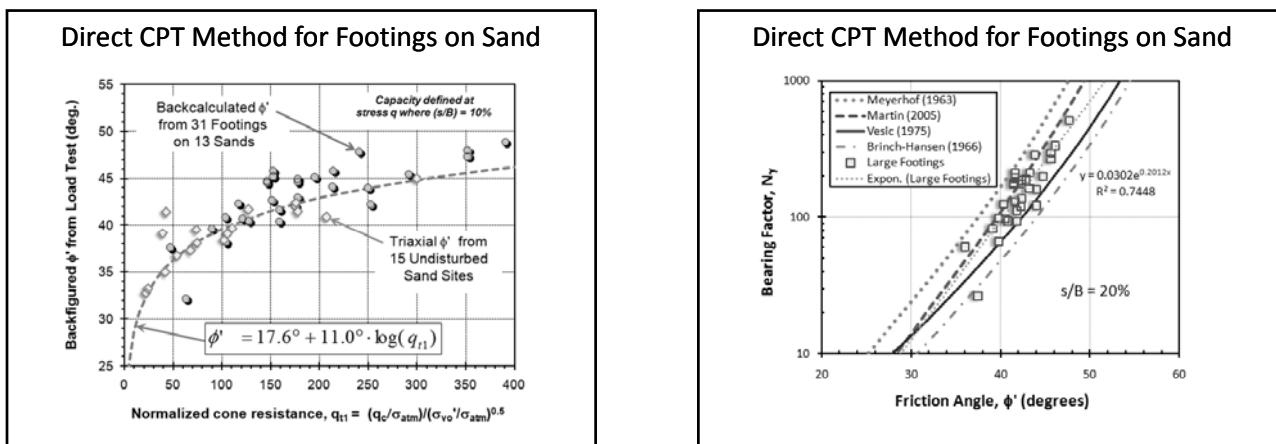
Geostratification by CPTu

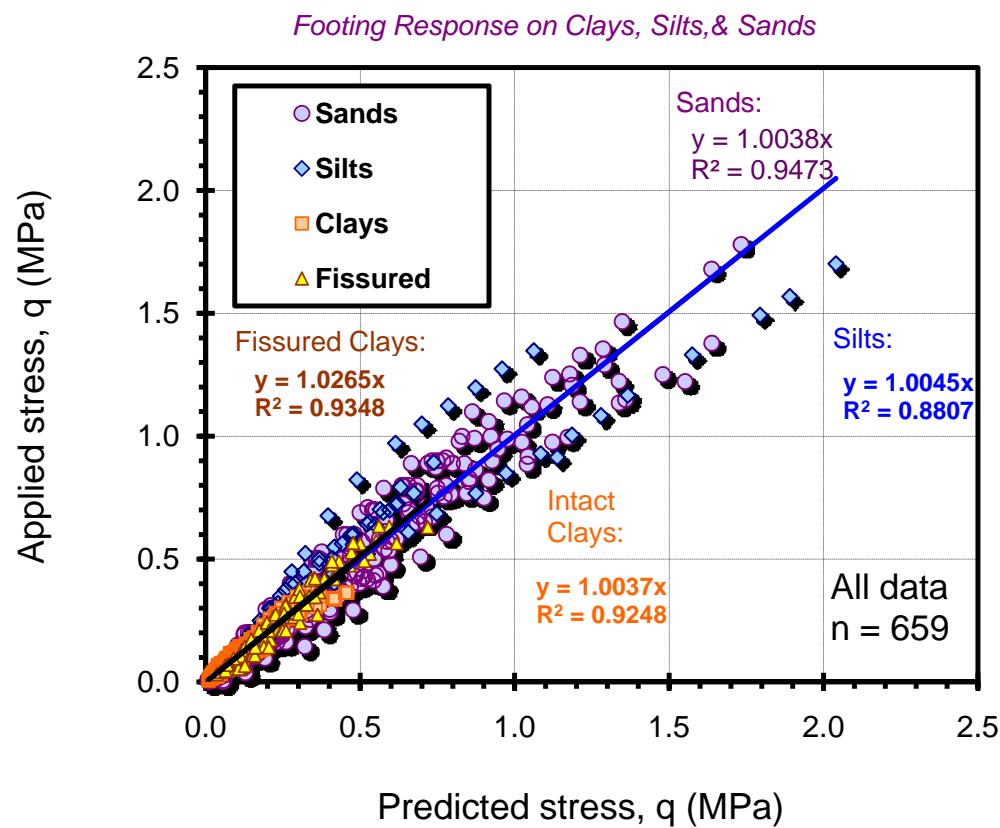
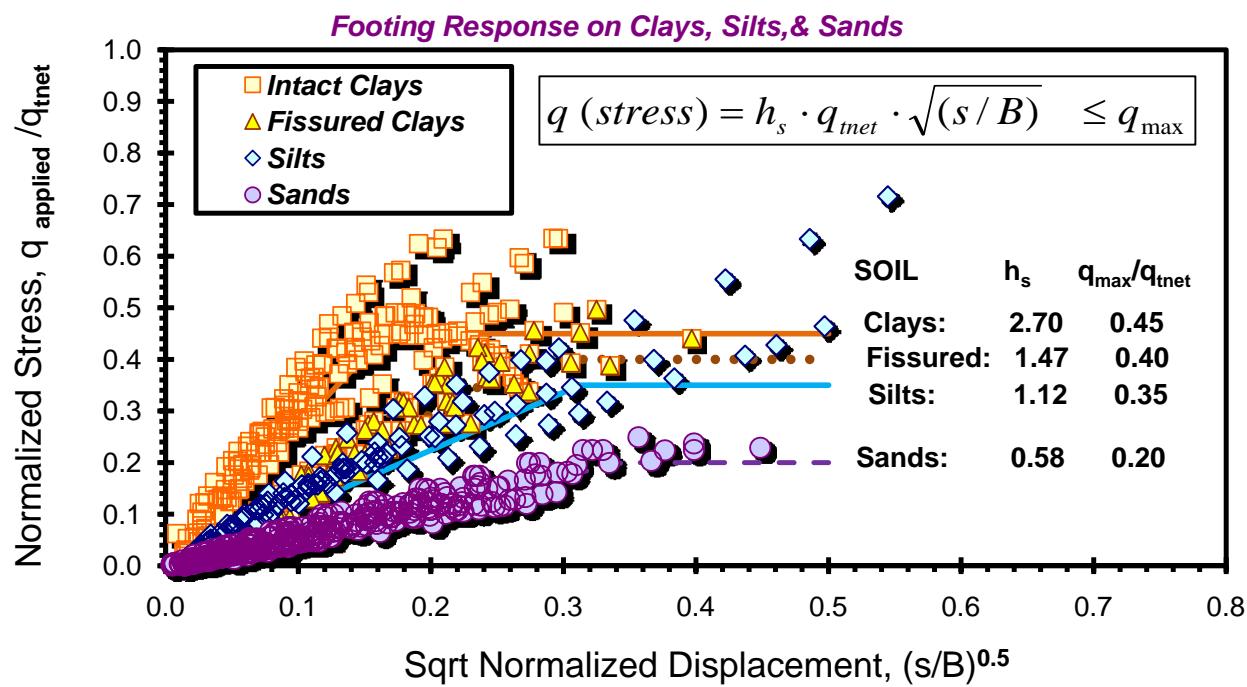
Nine-Zone Soil Behavioral Type (Robertson, 1990)











2009 Michael W. O'Neill Lecture

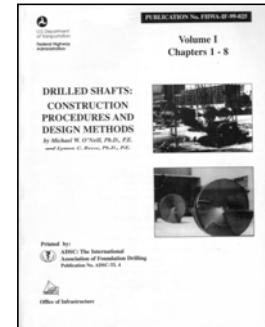
University of Houston

14th CIGMAT CONFERENCE
Center for Innovative Grouting, Materials, and Technology

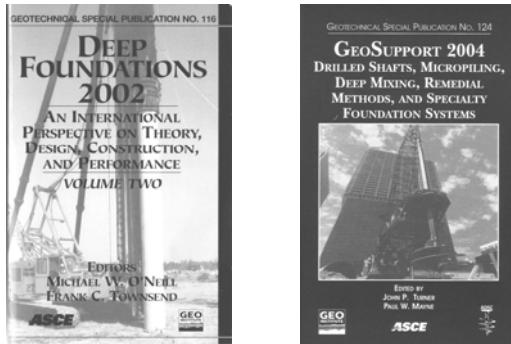
06 March 2009

Paul W. Mayne
Georgia Institute of Technology

Michael W. O'Neill (1940-2003)

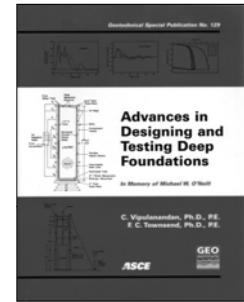


Michael W. O'Neill (1940-2003)



Michael W. O'Neill (1940-2003)

ASCE GSP 129 (2005)
Edited by C. Vipulanandan & F.C. Townsend

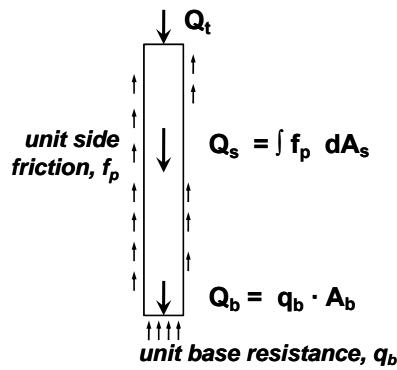


http://www.ce.ncsu.edu/usucger/Obituaries/O%27Neill_Michael.pdf

<http://www.vulcanhammer.net/wave/oneill.pdf>

<http://www.uh.edu/engineers/epi1822.htm>

Axial Pile Capacity: $Q_{\text{total}} = Q_{\text{sides}} + Q_{\text{base}}$



Axial Pile Capacity

- [82] $Q_{\text{total}} = Q_s + Q_b - W$
- [83] $Q_b = q_b A_b$
- [84a] *Undrained loading:* $q_b = *N_c \cdot s_u$
- [84b] *Drained loading:* $q_b = *N_q \cdot \sigma_{vo}'$

Axial Pile Capacity

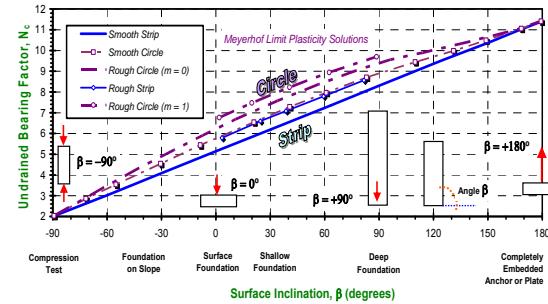
- Circular or square pile
- Undrained end bearing: $*N_c = 9.33$
- Drained end bearing (Vesic 1975): eqn [85]

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

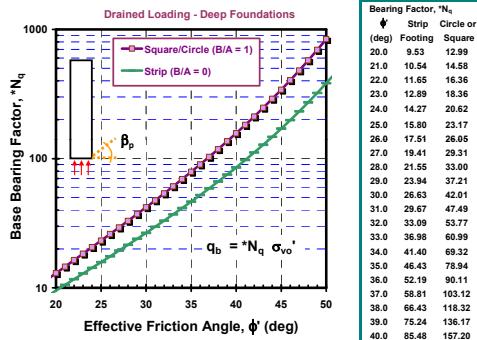
- Approx. eqn [86]:

$$*N_q \approx \frac{\exp\left(\frac{\phi'}{7.5}\right)}{1.3}$$

N_c = Undrained Bearing Capacity Factors (after Meyerhof, 1982)



N_q = drained bearing factor (Vesic 1975)



Beta Method for Pile Side Friction

$$\text{Eqn[86]: } f_p = C_M \cdot C_K \cdot K_0 \cdot \tan \phi' \cdot \sigma_{vo}'$$

Pile Installation Effects Modifier, C_K	Jeted Pile	$C_K = 0.5$
Drilled and Bored Piles		$C_K = 0.9$
Low-Displacement Driven Piles: (e.g., H-piles; open-ended pipe)		$C_K = 1.0$
High-Displacement Driven Piles (e.g., closed-ended pipe; precast)		$C_K = 1.1$
Pile Material Effects Modifier, C_M	Soil/Rough Concrete (drilled shafts)	$C_M = 1.0$
	Soil/Smooth Concrete (precast)	$C_M = 0.9$
	Soil/Timber (wood pilings)	$C_M = 0.8$
	Soil/Rough Steel (normal H- and pipe pilings)	$C_M = 0.7$
	Soil/Smooth Steel (cone penetrometer)	$C_M = 0.6$
	Soil/Stainless Steel (flat dilatometer)	$C_M = 0.5$

Rational Methods for Pile Capacity

- **End Bearing:** q_b = pile tip resistance
limit plasticity, cavity expansion theory, limit equilibrium
- **Side Resistance:** f_p = pile side friction
 - α method: $f_p = \alpha s_u$ and $\alpha = \text{fctn}(s_u)$
 - β method: $f_p = \beta \sigma_{vo}'$ and $\beta \approx K_0 \tan \phi'$
 - λ method (offshore)
 - effective stress methods
 - numerical simulations

α -Methods for Pile Side Resistance

- Tomlinson (1957, 4th ICSMFE)
- Wroth (1972, ASCE Purdue Conf)
- Vesic (1977, NCHRP 42, TRB)
- Semple (1980, ICE, London)
- American Petroleum Inst. (1981)
- Dennis & Olson (1985, OTC)
- Kulhawy & Jackson (1989, ASCE GSP 22)
- Nowacki et al. (1993, ICE, London)
- Phoon & Kulhawy (1993, GSP 38)
- Randolph (2003 Rankine, Geotechnique)

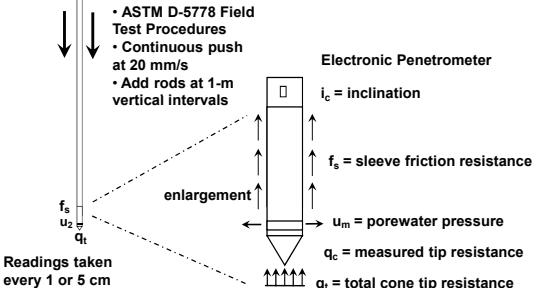
β -Methods for Pile Side Resistance

- Burland (1972, Geotechnique)
- Flaate & Selnas (1977, 9th ICSMFE, Tokyo)
- Kulhawy et al. (1983, EPRI EL-2870)
- Konrad & Roy (1987, Geotechnique)
- Poulos (1989 Rankine, Geotechnique)
- Kulhawy & Jackson (1989, GSP 22)
- Karlsrud & Nadim (1990, OTC, Houston)
- Karlsrud (1999, ASCE GSP 88, Austin)
- O'Neill (2001 Terzaghi, ASCE JGGE)

Cone rig with hydraulic pushing system

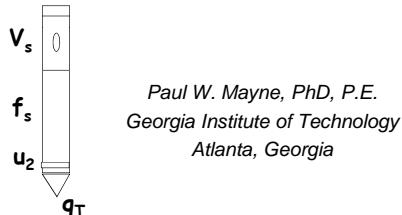


Cone Penetration Test (CPT)



2009 Michael W. O'Neill Lecture

Evaluating Axial Pile Response by Cone Penetrometer Tests



AXIAL PILE CAPACITY FROM CONE PENETROMETER

Method One "Direct" CPT Method (Scaled Pile)

$$f_p = f_{ctn} \text{ (soil type, pile type, } q_t \text{, or } f_s \text{ and } \Delta u \text{)}$$

$$q_b = f_{ctn} \text{ (soil type, } q_t \text{-} u_b \text{, and degree of movement, } s/B \text{)}$$

$$Q_{Total} = Q_s + Q_b - W_p$$

$$Q_{side} = \sum (f_p dA_s)$$

$$Q_{base} = q_b A_b$$

Method Two: Rational or "Indirect" Method

$$\text{OCR, } s_u, K_o, \gamma_l, D_R, \phi'$$

$$f_p = c_m c_k K_o \sigma_{vo}' \tan \phi'$$

$$\text{Drained: } q_b = N_q \sigma_{vo}'$$

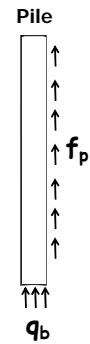
$$\text{Undrained: } q_b = N_c s_u$$

$$q_b = \text{unit end bearing}$$

LCPC Pile End Bearing from CPT

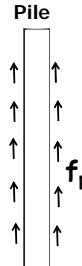
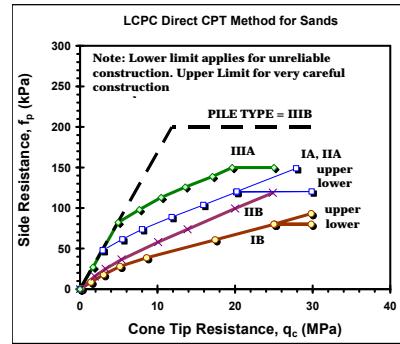
(Bustamante & Gianeselli, 1982; Frank & Magnan, 1995)

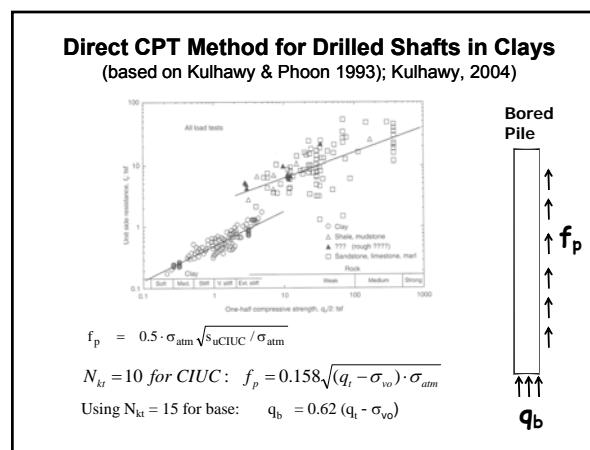
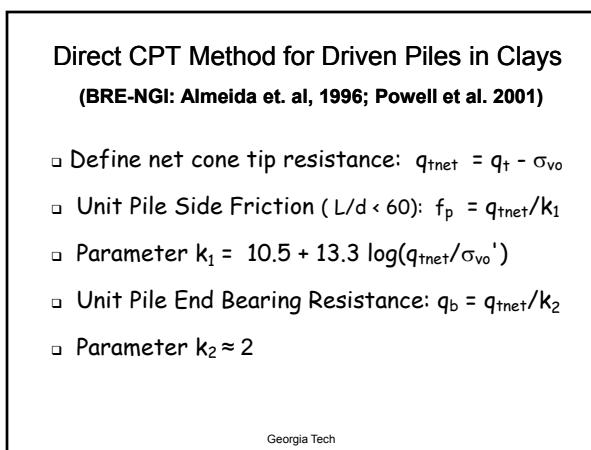
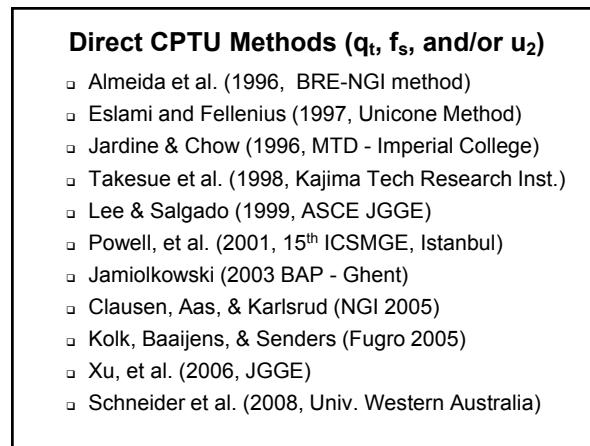
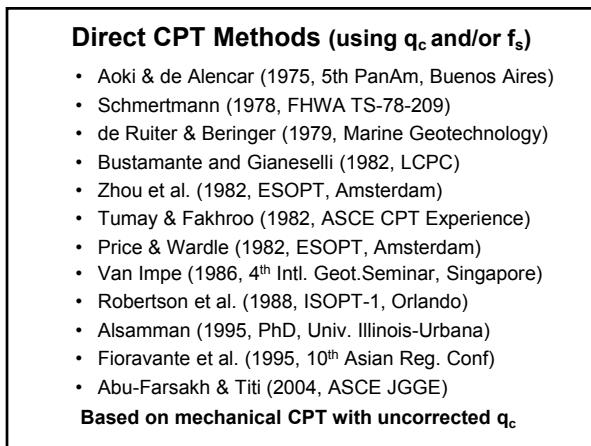
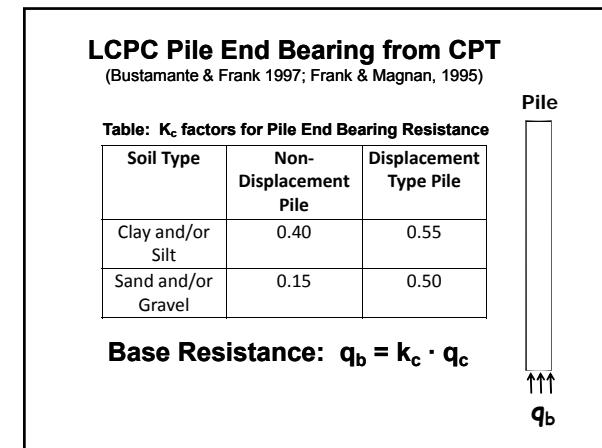
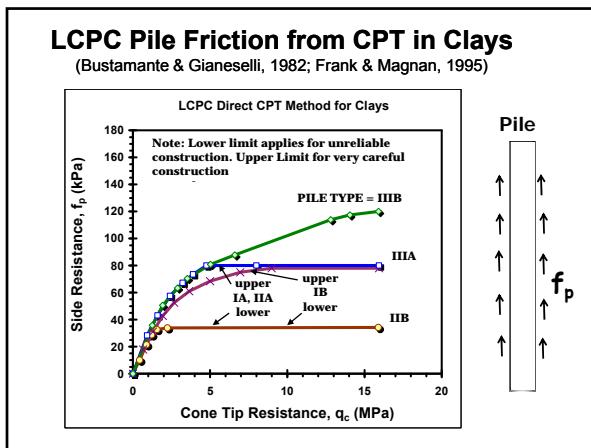
Pile Category	Type of Pile
IA	Plain bored piles, Mud bored piles, Hollow auger bored piles, Case screwed piles, Type I micropiles, Piers, Barrettes
IB	Cased bored piles, Driven cast piles
IIA	Driven precast piles, Prestressed tubular piles, Jacked concrete piles
IIB	Driven steel piles, Jacked steel piles
III A	Driven grouted piles, Rammed piles
III B	High pressure grouted piles ($d > 0.25m$), Type II micropiles



LCPC Pile Friction from CPT in Sands

(Bustamante & Gianeselli, 1982; Frank & Magnan, 1995)





Fugro Direct CPT Method for Driven Closed-End Pile in Sands
(Clausen et al. 2005; Schneider et al. 2008)

$$f_p = 2.1 \cdot (D_R - 0.1)^{1.7} (\sigma_{vo}' / \sigma_{atm})^{0.25} (z/L) \cdot \sigma_{atm} \cdot F_{tip} \cdot F_{load} \cdot F_{mat} \geq 0.1 \cdot (\sigma_{vo}' / \sigma_{atm})$$

- $D_R = 0.4 \ln\{\ q_c/[22(\sigma_{vo}' \cdot \sigma_{atm})^{0.5}] \} \}$ = estimated relative density (as a decimal)
- z = depth
- L = pile length
- F_{tip} = tip factor (= 1.0 for open-ended; 1.6 for closed-ended)
- F_{load} = load factor (= 1.0 for tension and 1.3 for compression)
- F_{mat} = material factor (= 1.0 for steel and 1.2 for concrete).
- End bearing resistance for a driven closed-ended pile is:

$$q_b = \frac{0.8 \cdot q_c}{(1 + D_R^2)}$$

Driven Piles in Quartz-Silica Sands
(Schneider, et al. 2008, ASCE JGGE Sept)

- Four recent methods (2005)**
- Driven piles in sand**
- Open-ended and closed-ended**
- Evaluate plugging**
- Statistical reliability measures**

Method	Design equation
Fugro-05*	$\sigma_{tip} = 0.1 (B_0) \left[\frac{\sigma_{vo}'}{\sigma_{atm}} \right]^{0.25} \left[\frac{z}{L} \right]^{0.25}$
ICP-05	$\sigma_{tip} = 0.1 (B_0) \left[\frac{\sigma_{vo}'}{\sigma_{atm}} \right]^{0.25} (Lz)^{-0.8} \left[\frac{A}{B^2} \right]^{0.25}$
NGB-05	$\sigma_{tip} = 0.022 (B_0) \left[\frac{\sigma_{vo}'}{\sigma_{atm}} \right]^{0.25} \left[\frac{z}{L} \right]^{-0.25} + A \sigma_{tip}^{0.25} \ln h_2$ h=0.8 for open-ended piles in tension and 1.0 for all other cases h=0.8 for piles in tension and 1.0 for piles in compression
UWA-05	$\sigma_{tip} = 0.22 (B_0) \left[\frac{\sigma_{vo}'}{\sigma_{atm}} \right]^{0.25} \left[\frac{z}{L} \right]^{-0.25} + A \sigma_{tip}^{0.25} \ln h_2$ $B_0 = 0.4 \ln(z/L) + 22$ $F_{tip} = 1.0$ for driven open ended and 1.6 for driven closed $F_{mat} = 1.0$ for tension and 1.3 for compression $F_{load} = 1.0$ for sand and 1.2 for concrete $A = \text{Area of effective area ratio} = 1 - B(B_0/D)^2$ $A/\sigma_{tip} \text{ ratio} = 1 - (D/B)$ $h_2 = 0.8$ if f_{tip}/f_{load} is not measured, average BIR = $\text{avg}(f_{tip}/f_{load}) \times 1.5^{0.25}$ f_{tip}/f_{load} ratio of tension to compression capacity (equal to 1 for compression and 0.8 for tension)

Direct CPT Method for Drilled Shafts in Sands
(Fioravante et al. 1995; Jamiolkowski 2003)

KTRI Pile Friction Method from CPTu
(Takesue, et al. 1998)

Unicone Method

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

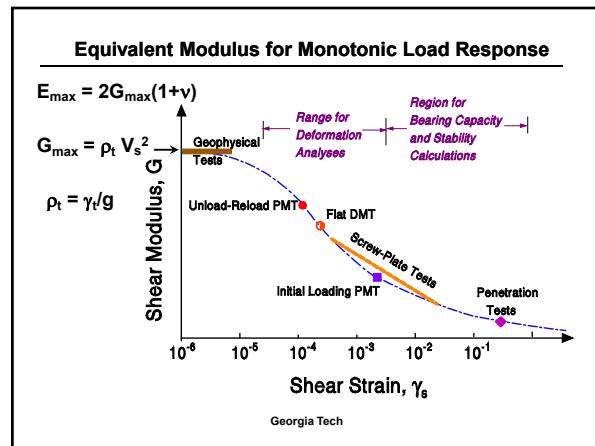
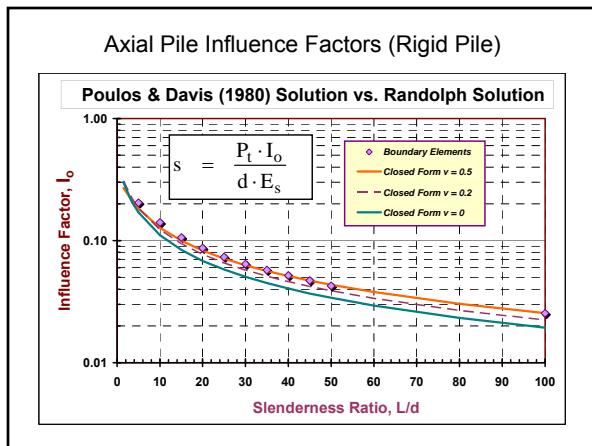
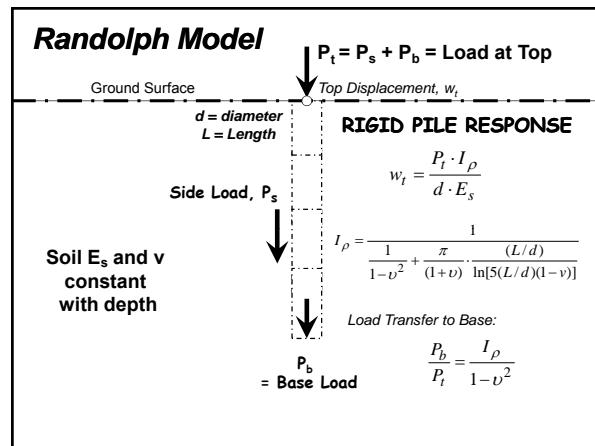
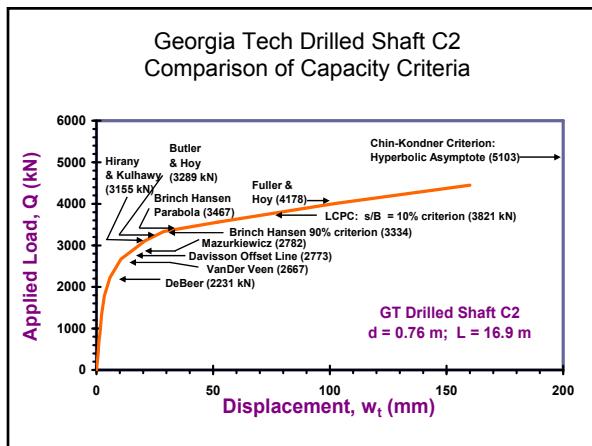
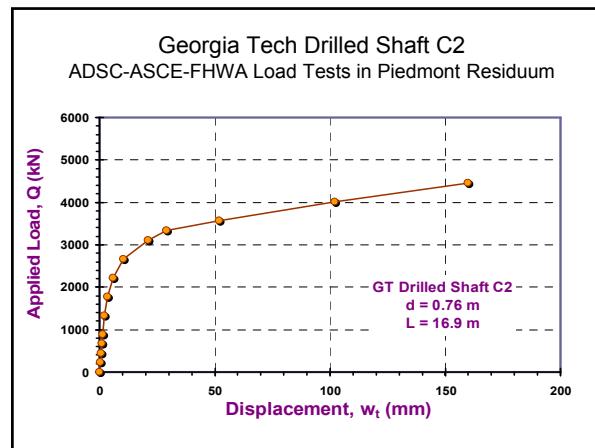
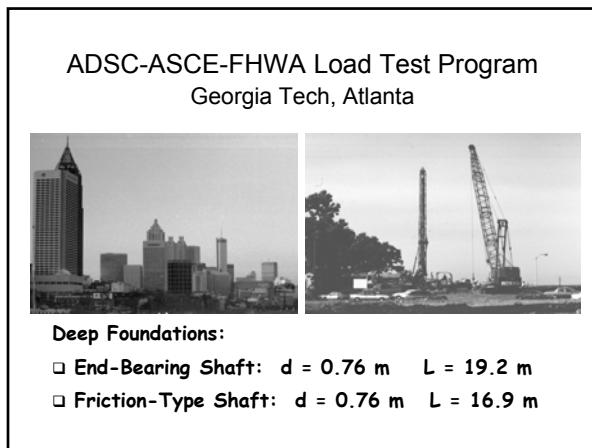
- Determine effective cone resistance: $q_E = q_t - u_2$
- Plot q_E vs f_s for soil type
- Side Resistance: $f_p = C_s \cdot q_E$
- Tip Resistance: $B < 0.4m: q_b = q_E$
 $B > 0.4m: q_b = q_E/(3B)$
- $B = \text{pile width (m)}$

Soil Type	C _s Value
1. Very soft sensitive soils	0.080
2. Soft Clay	0.050
3. Stiff clay to silty clay	0.025
4. Silt-Sand Mix	0.010
5. Sands	0.004

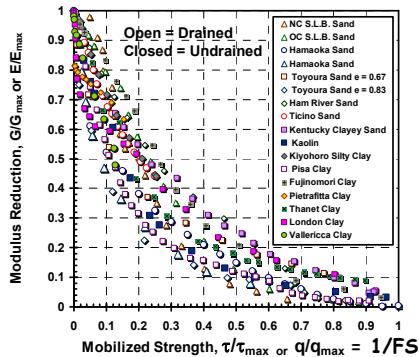
NCHRP Synthesis 368: Cone Penetration Test

Chapter 8 on Pile Foundations

- www.trb.org
- webforum.com/tc16
- geosystems.ce.gatech.edu



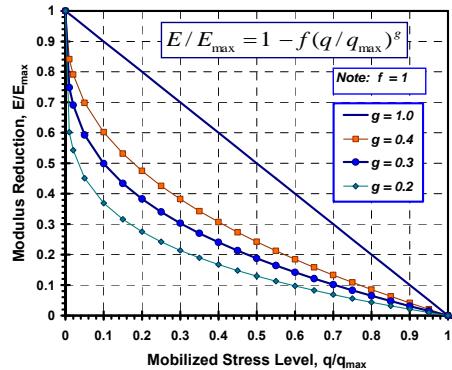
Modulus Reduction from TS and TX Data



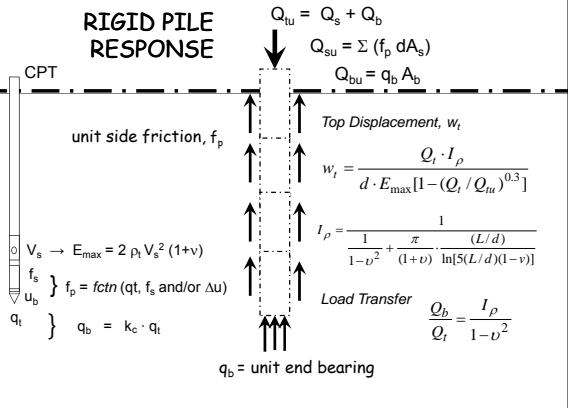
Equivalent Modulus for Monotonic Loading (Static) Response

- Initial Stiffness from Shear Wave Velocity (V_s)
 - Shear Modulus: $G_{max} = \rho V_s^2$
 - Young's Modulus: $E_{max} = 2 G_{max}(1+\nu)$
 - $\nu = 0.20$ at small strains
- Modified Hyperbola for Modulus Reduction (Fahey & Carter, 1993; Fahey 1998; Mayne 2007):
 $E/E_{max} = 1 - f(q/q_{ult})^g$
- Factor of Safety, $FS = q_{ult}/q$
- For uncemented, unstructured soils: $f = 1$; $g \approx 0.3$

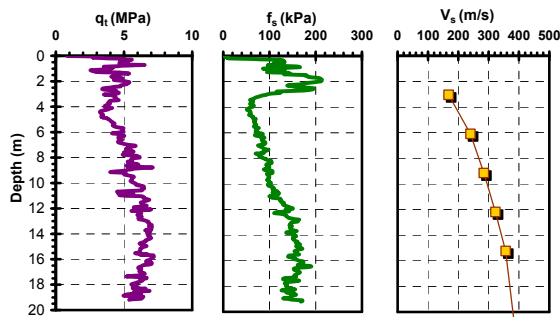
Modulus Reduction Scheme (Fahey & Carter 1993)



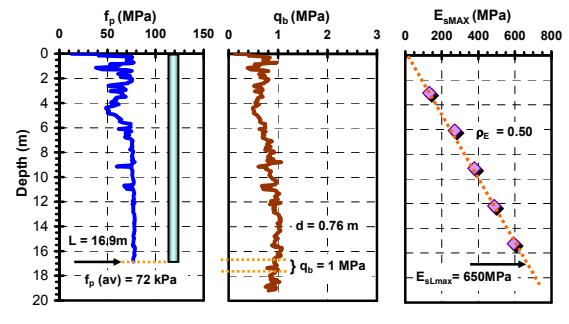
RIGID PILE RESPONSE

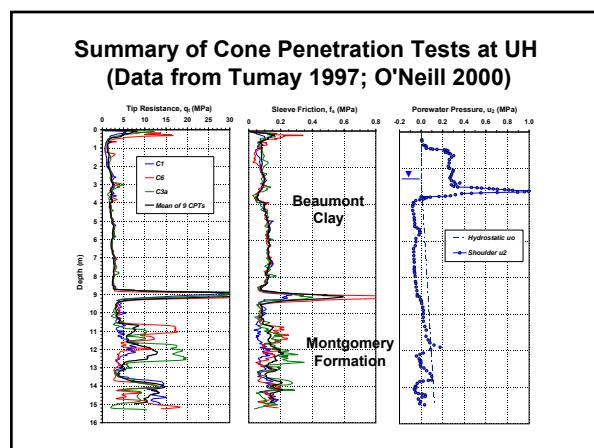
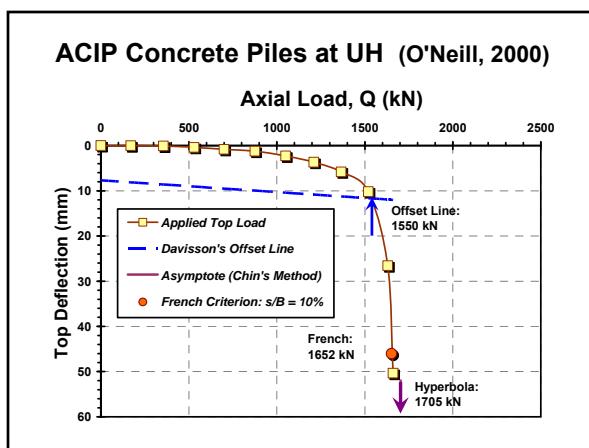
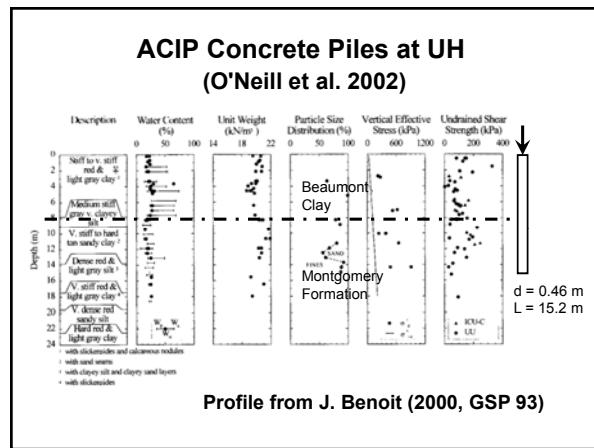
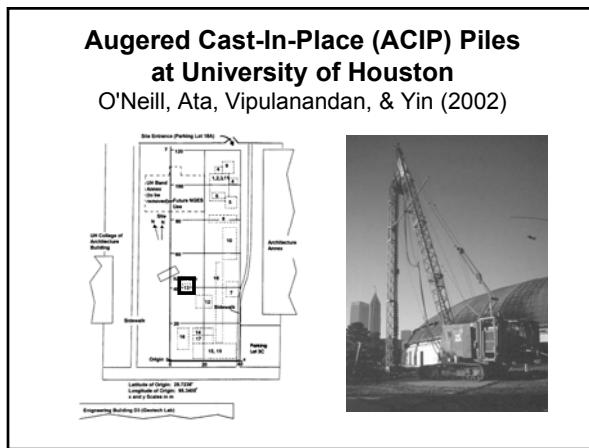
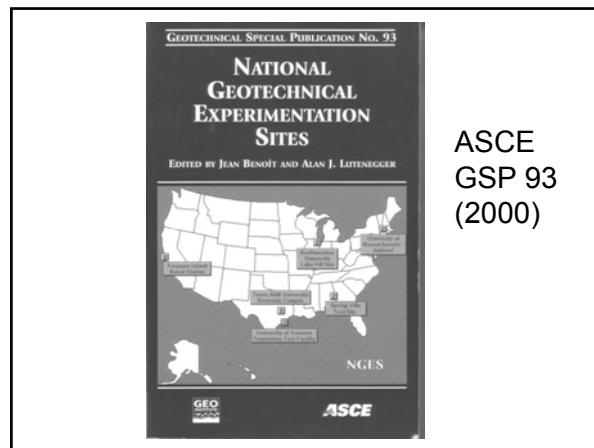
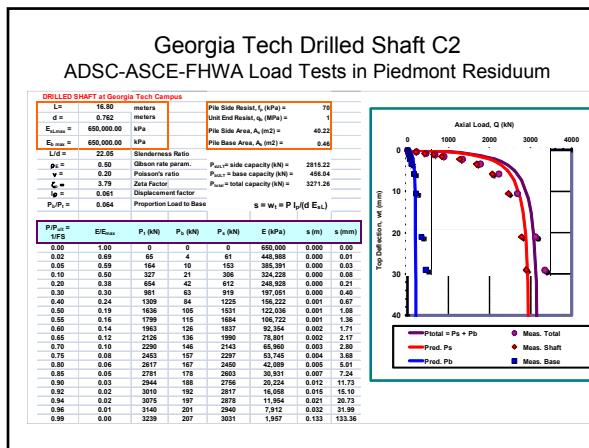


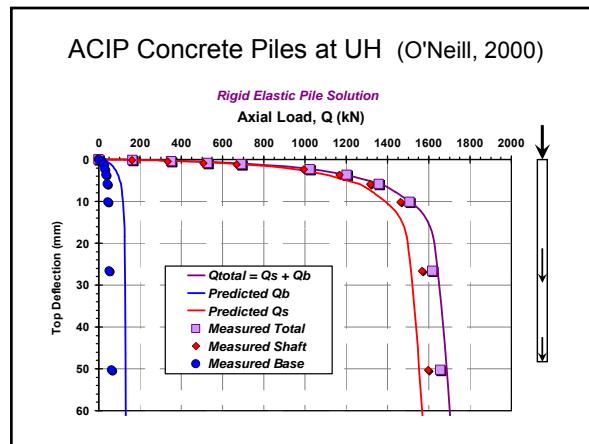
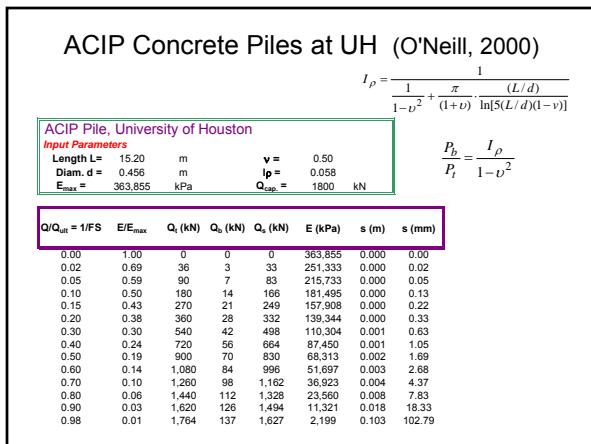
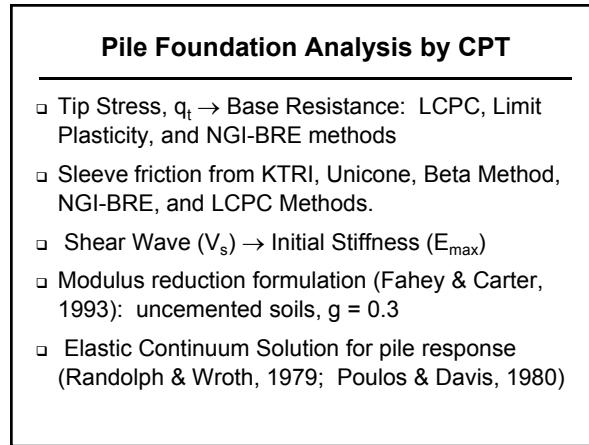
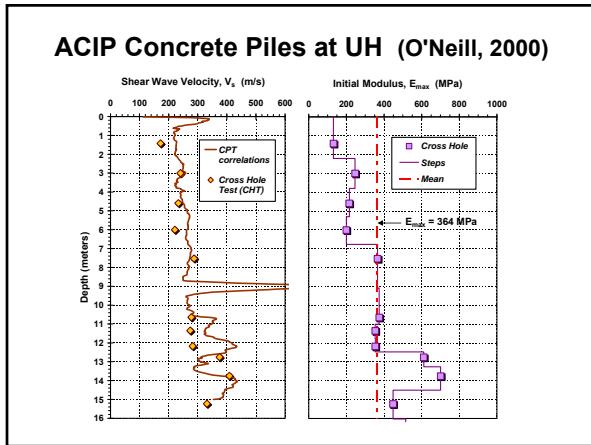
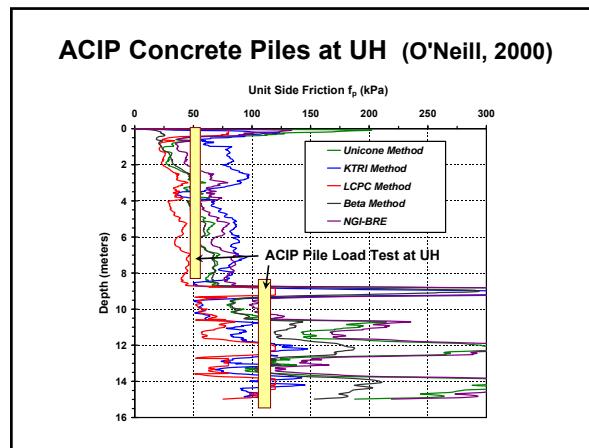
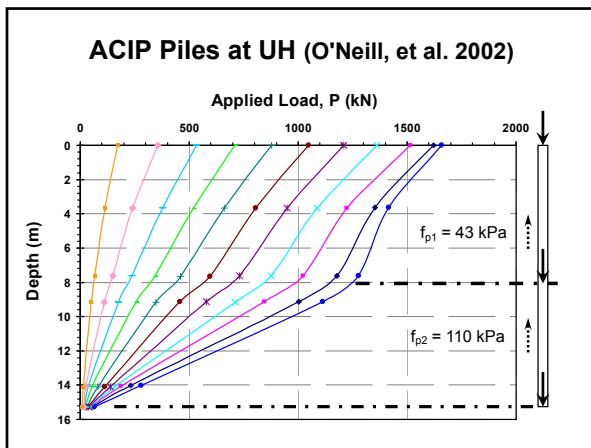
Georgia Tech Drilled Shaft C2 ADSC-ASCE-FHWA Load Tests in Piedmont Residuum

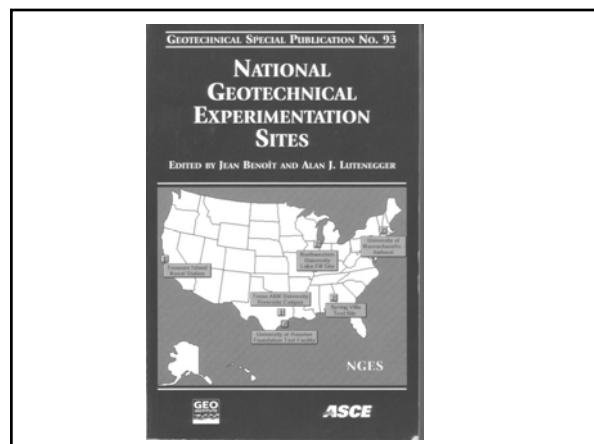
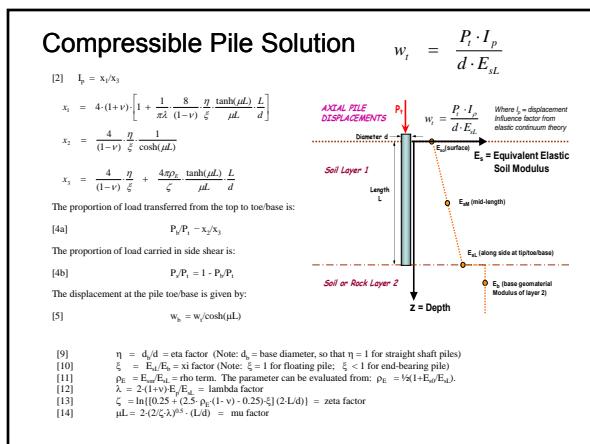
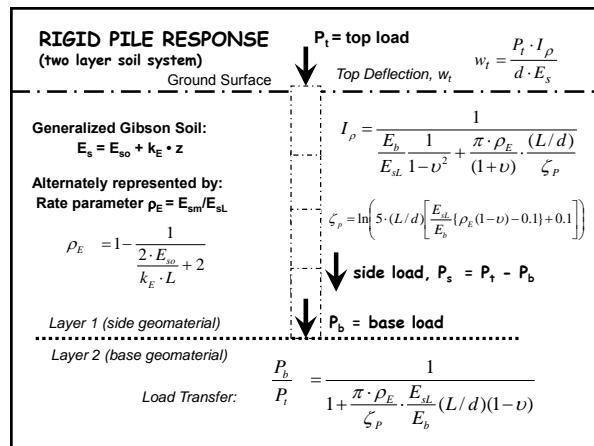
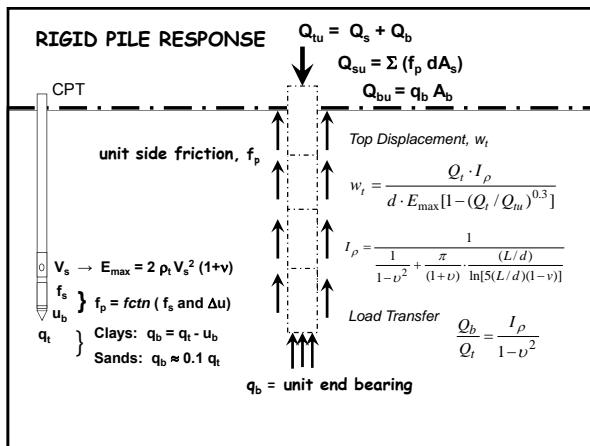
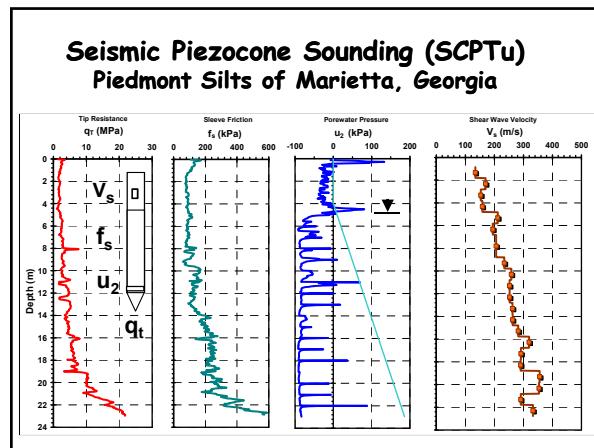
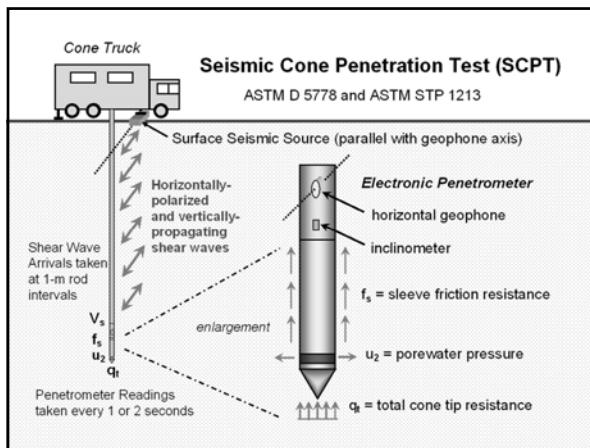


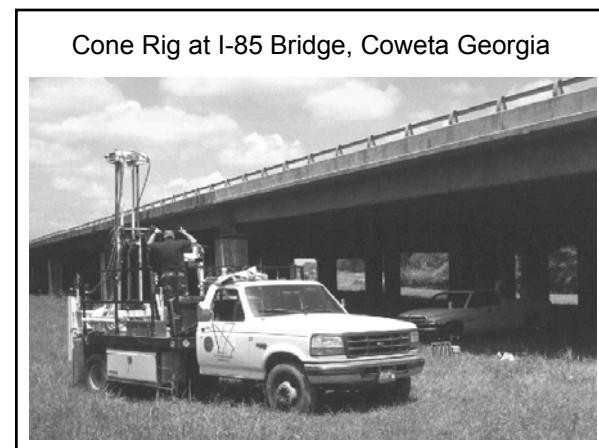
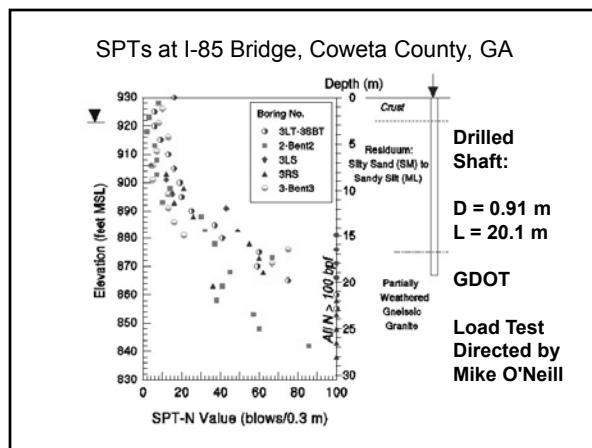
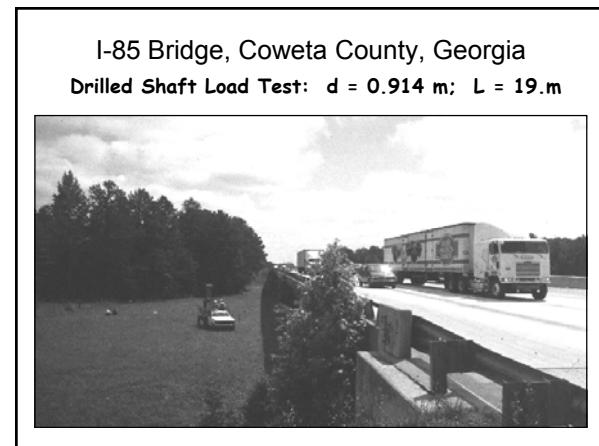
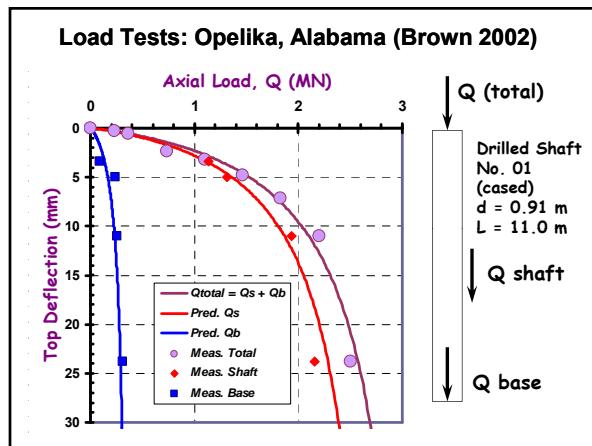
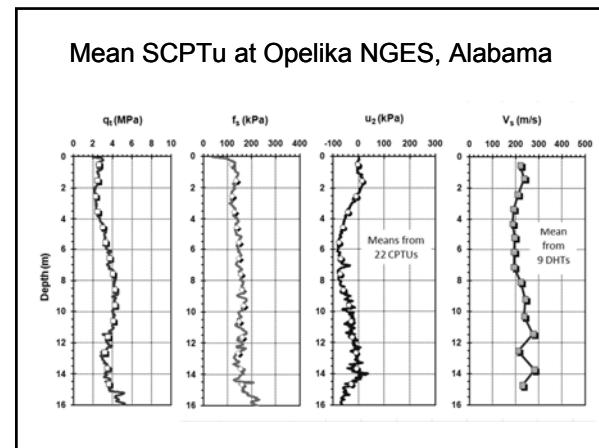
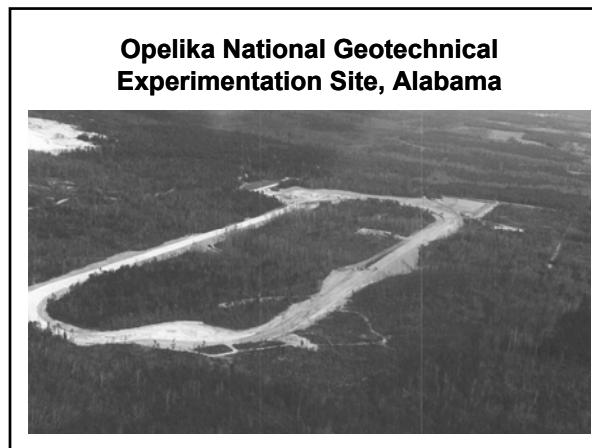
Georgia Tech Drilled Shaft C2 ADSC-ASCE-FHWA Load Tests in Piedmont Residuum

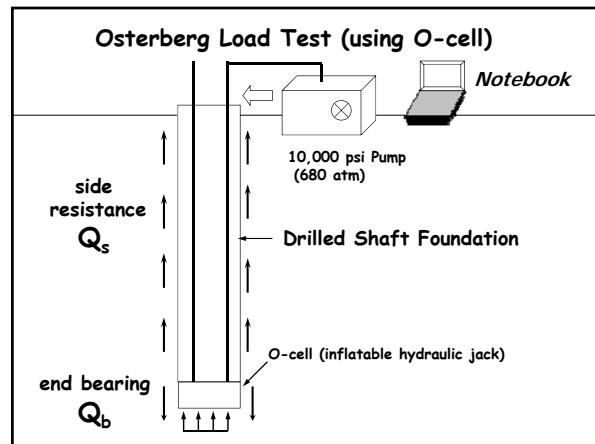
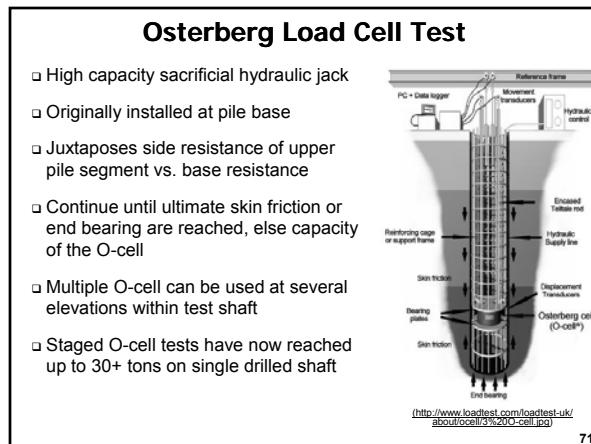
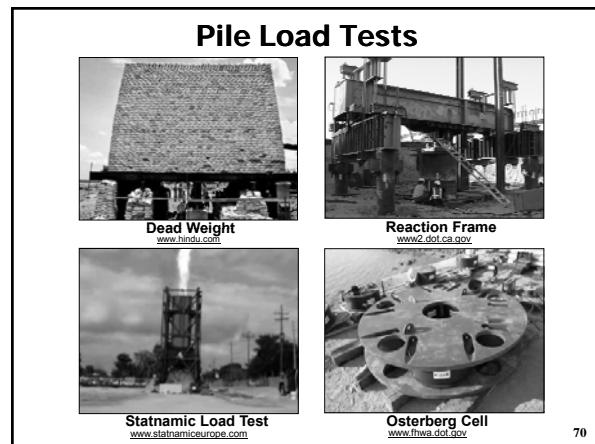
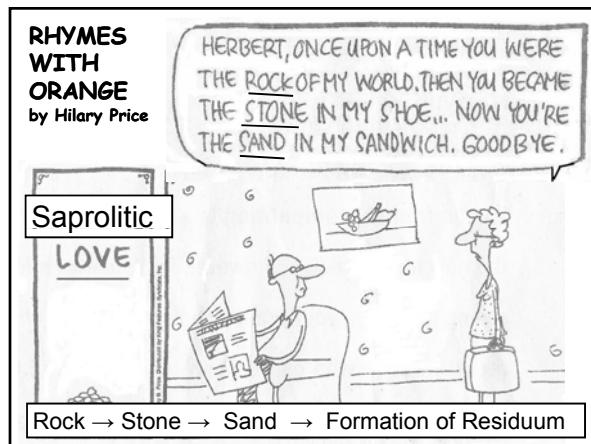
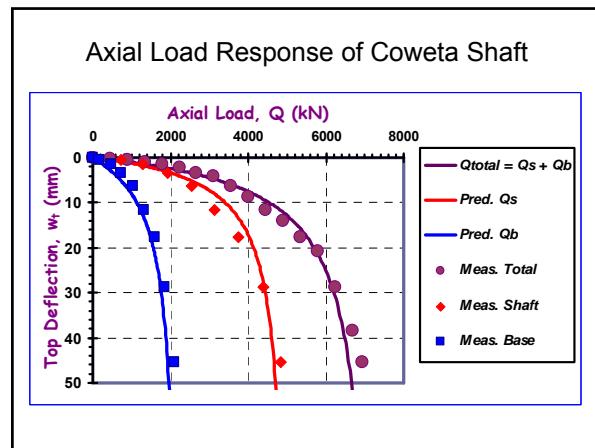
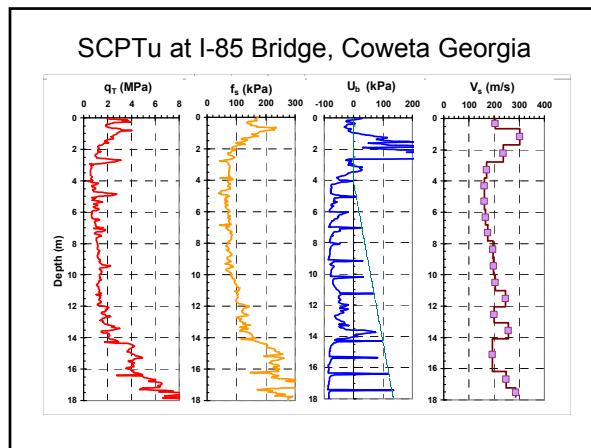


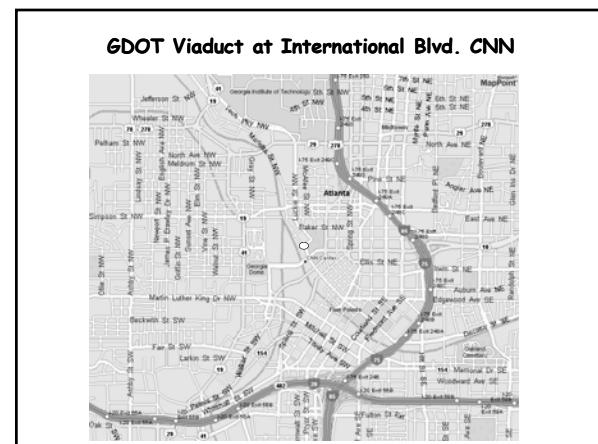
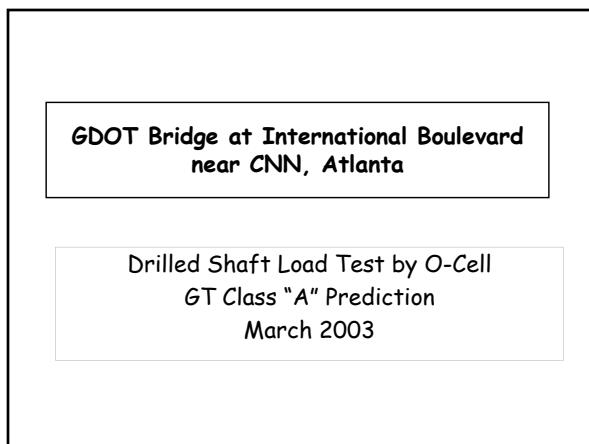
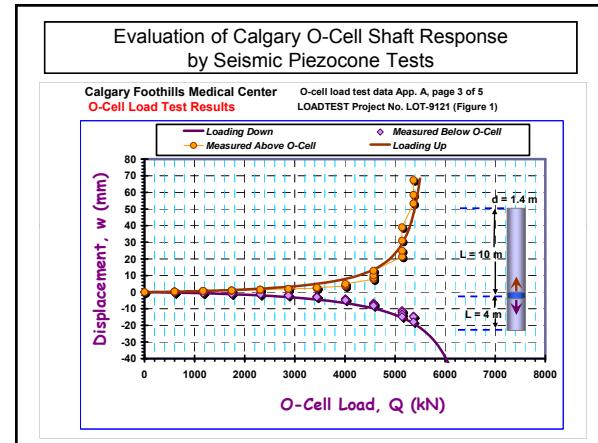
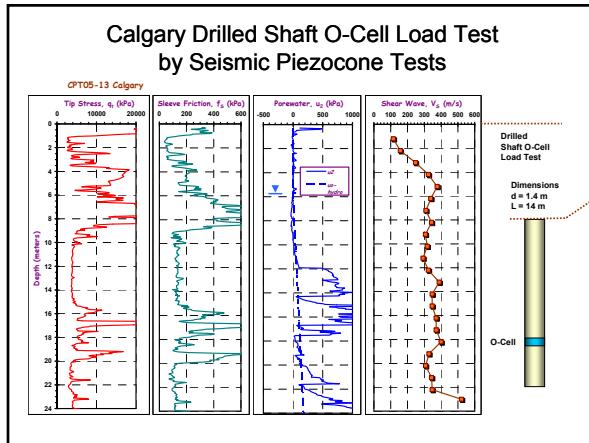
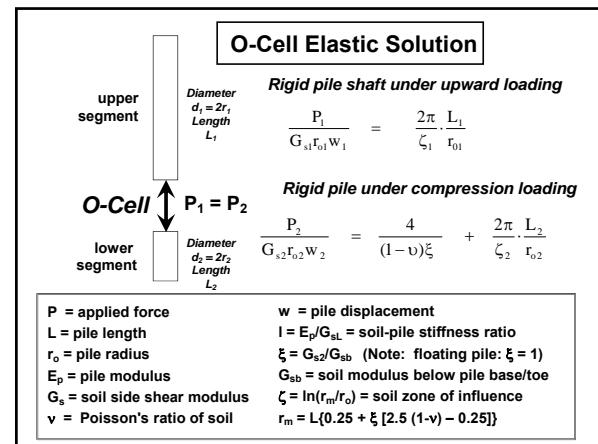


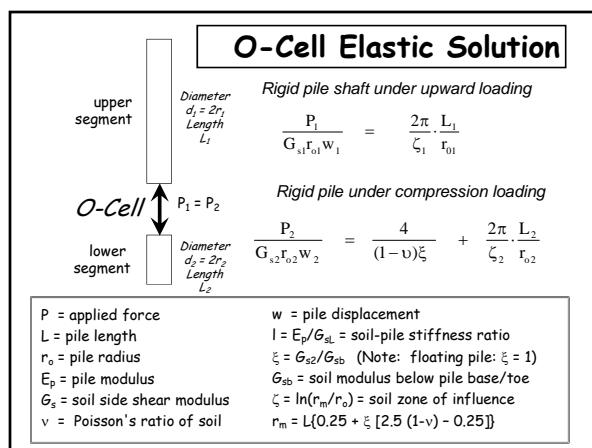
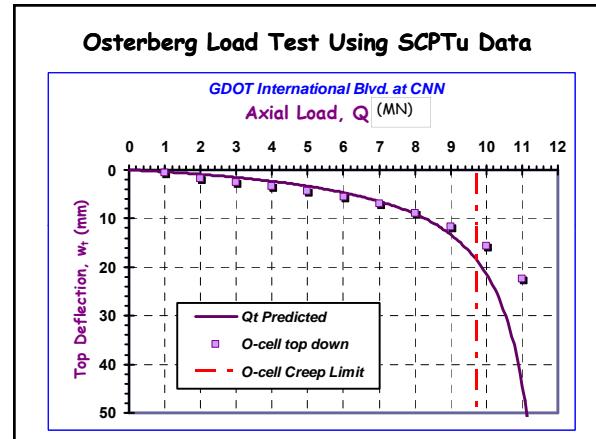
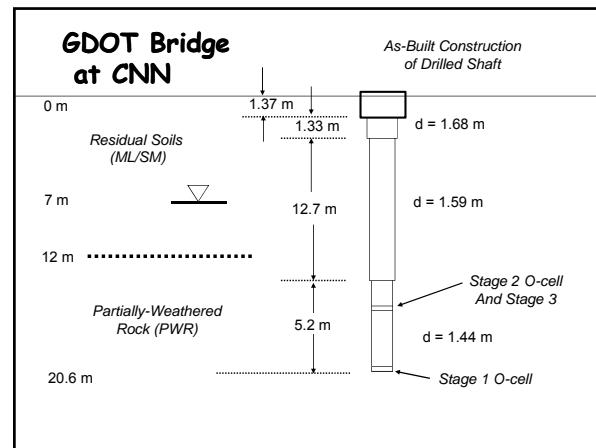
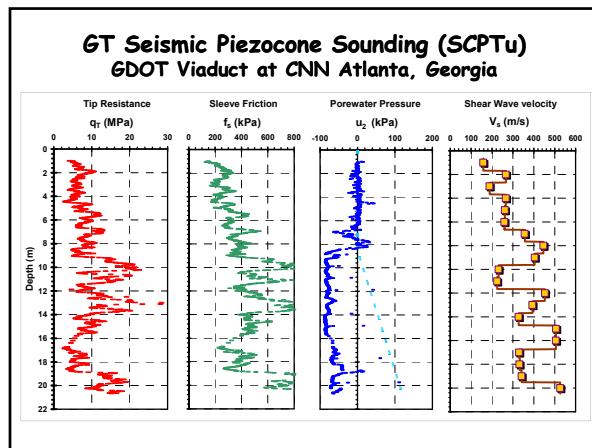








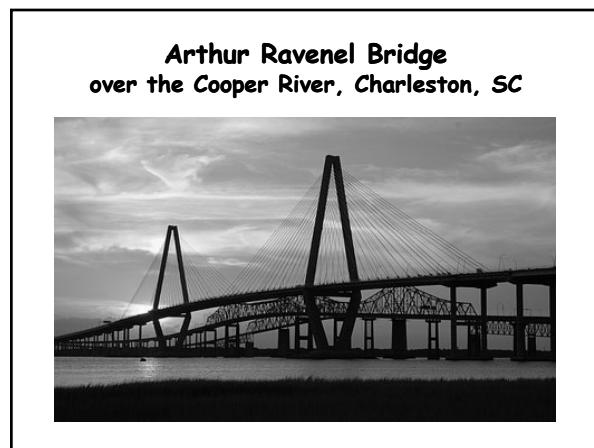
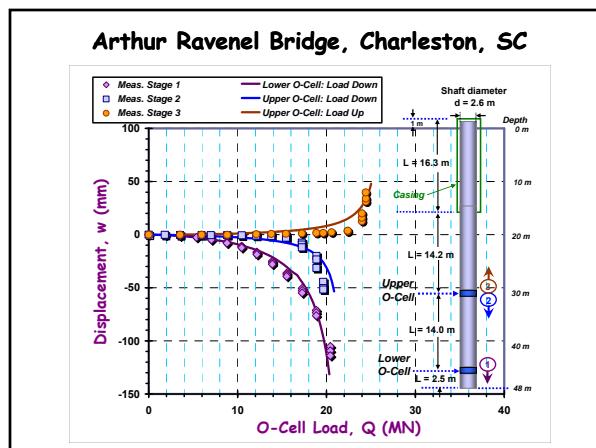
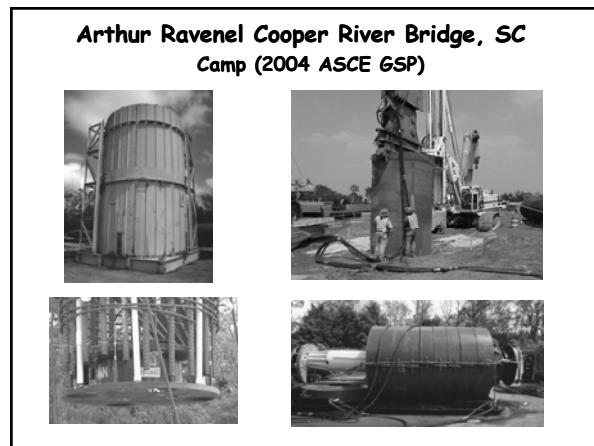
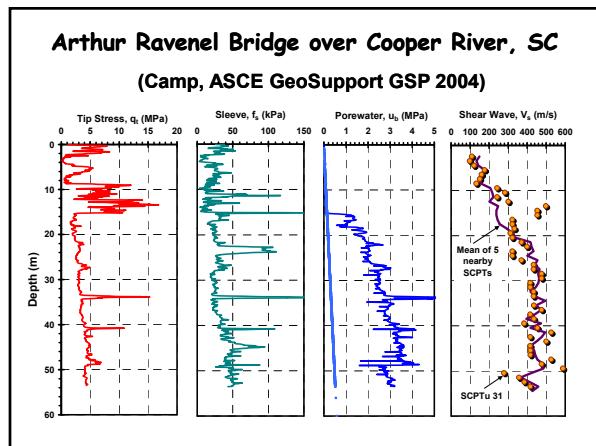
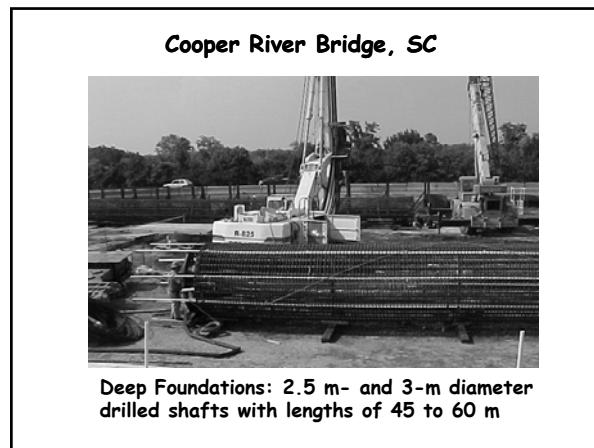
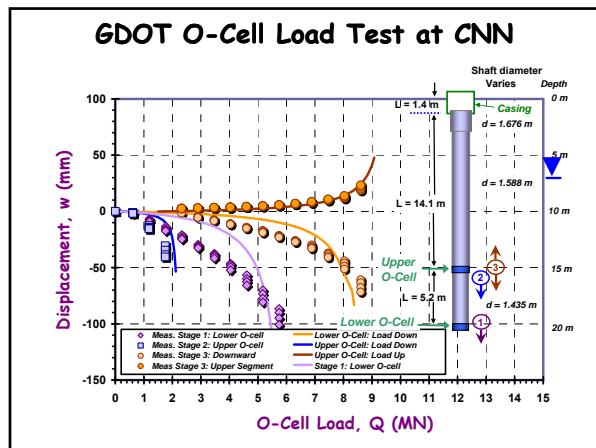


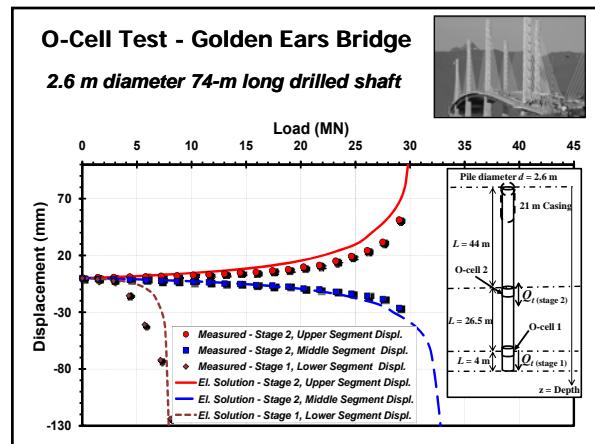
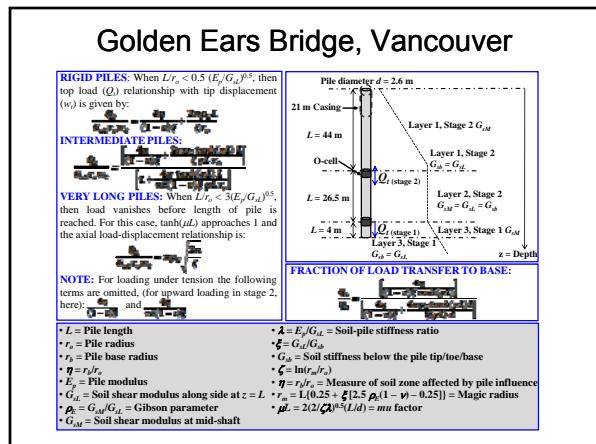
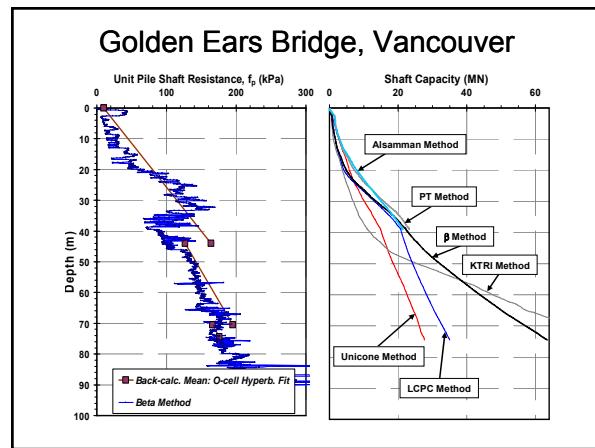
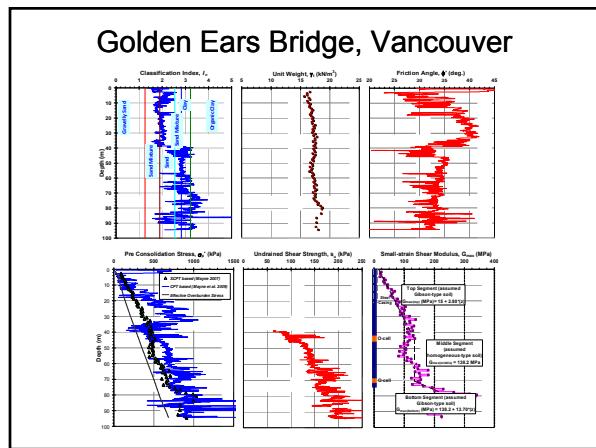
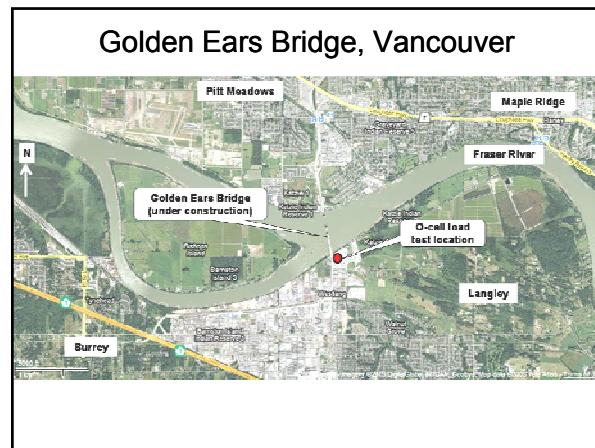
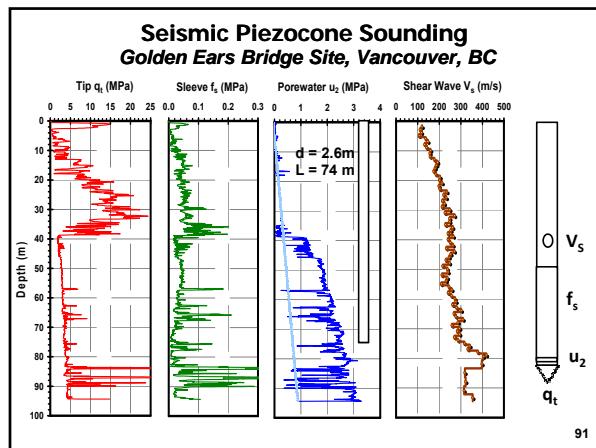


RIGID BASE RESPONSE (Compression)

(Poulos, 1988; Randolph & Wroth, 1978, 1979)
SEPARATE REDUCTIONS FOR SIDE and BASE COMPONENTS
GDOT I-40 CNN International Blvd Viaduct, Atlanta, GA
STAGE 1: Upward O-cell Pulling Down
Mid-Section plus 1/2 Base

Pile Diameter, d (m)	1.44	(E_h) = Modulus pile tip	23.58						
Pile Length, L (m)	5.23	(E_h) = Bearing Stratum							
Pile Modulus, E_p (kPa)	35000000	(E_m) = Mid-depth Modulus							
Poisson's Ratio, ν	0.25	(E_b) = Mod. at Surface							
Soil Modulus, E_s (kPa)	480000	L/d Ratio	3.645						
Base Modulus, E_b (kPa)	1392000	Ratio P_b/P_t	0.166						
Rho = Ratio (E_s/E_b) s_z , L	2735.00	Zeta	1.828						
Shear Capacity, Q_s (kN)	6105.6	$K = E_b/d$	7.29						
Base Capacity, Q_b (kN)	6105.6	Ratio P_s/P_t	0.834						
Total Capacity, Q_t (kN)	8843.6	$\tan \delta$	0.500						
Compress. Param. 0									
X = Ratio (E_h/E_b)	0.344827598	Ratio Q_{base}/Q_{comp}	0.71233						
Els = Base Effect		Side Area (m ²)	23.58						
dam d (inches)	56.49882	Area Pile (m ²)	1.62						
DAVISSON OFFSET LINE CRITERION		Pile Modulus, $E_p = \rho \cdot d^2/2$	0.72						
Offset, a inches	0.6202825 inches	Ratio L/e	7.29						
$P_t = A_d E_p L/a$	3944622.2 N/mm	RIGID L/e < $0.5(E_b/d)^{0.5}$	3.88						
for 10 mm change, $Q =$	3944622.2 N/mm	LONG L/e > $2.5(E_b/d)^{0.5}$	23.50						
	16.7693169 0	Max above soil surface kPa	200000						
	16.0325643 10400	Gmax (kPa)	580000						
		q end (kPa)	3777						
		End (kPa)	116						
		BASE ELEV Vs (m/s)	526						
		Unit wt. (KN/m ³)	20.54						
		CPT qt at base	15900 kPa						
Increment	0.18	Expon g [#]	0.3						
Displacement (mm)	w (mm)	w (m)	Pb (kN)	Pb/Ptu	Gbase	Pgide	Pgide/Pb		
0.0000	0.0000	0.0000	0.0000	0.0000	580000.0000	0.0000	20000.0000	1.1618	
0.0000	0.0000	0.0000	0.0042	0.0000	580000.0000	0.0040	0.0000	20000.0000	
0.0000	0.0000	0.0000	0.0049	0.0000	57197.4198	0.0049	0.0000	19623.0000	1.1602





European Pile Foundation Systems

www.geoforum.com

Website maintained by Rainer Massarsch Sweden

Pile Info

Pile Installation Process

Pile Classification System

Pile Info

Swedish Piling Technology Award

Geo Events

Geo Directory

Geo Market Guide

Consultants

Contractors

New and Emerging Pile Types

Atlas Pile

Dewaal Pile

Olivier Pile

Fundex Pile

New and Emerging Pile Types (Paniagua, 2009)

Pile / Technique	Country	Developer – Contractor ^a
<i>First Generation</i>		
Atlas	Belgium	Franki
De Waal	USA	Morris Shea
Funda VB	Germany	Jebens
Fundex	USA	American Pile Driving
<i>Second Generation</i>		
Screw injection	Netherlands	Fundierungstechnik
SVB	Germany	Jebens
SVV	Germany	Jebens
Tubex	USA	American Pile Driving
<i>Third Generation</i>		
APGD	USA	Berkel
Displacement	Germany	Bauer
Screw pile	UK	Cementation
Soil displacement	UK	Cementation
Discrepile	Italy	Trevi
Omega	Belgium	Socofonda
T.	France	Soleilanche-Bachy
TSD	UK	May Gurney

German Direct CPT Method: Piles in Sand

Kempfert & Becker (2010, ASCE GSP 205)

German Direct CPT Method

Kempfert & Becker (2010, ASCE GSP 205)

Sands

Clays

Mayne and Niazi (2009 Michael W. O'Neill Lecture)

DFI JOURNAL

DEEP FOUNDATIONS INSTITUTE

Vol 2, No 1 May 2009

PAPERS

- Evaluating Axial Elastic Pile Response from Cone Penetration Tests (The 2009 Michael W. O'Neill Lecture) – Paul L. Mayne, Michael W. O'Neill, and Michael J. O'Neill [1]
- Test Pile Program To Determine Axial Capacity And Pile Skin Friction Parameters – W. Robert Thompson, H. P. Z. Usmani, P. E., Steven Saxe, P. E. [2]
- Reduction of Local Shear around Cylindrical Bridge Pile Foundations Using Prepressive Piles – Dr. R. B. Bhattacharya [3]
- Soil Model in jet Grout Columns – Timothy D. Stark, Paul J. Koenig, and John C. Weller [4]
- Effect of Soil Properties on the Lateral Load Capacity of Grouted Shafts – A Practical Insight and an Analytical Approach – Dr. Olegor Begishev [5]
- Design Considerations for Piles in Soft Glacial Till – Kenneth Giese [6]

Deep Foundations Institute is the Industry Association of Individuals and Organizations Dedicated to Quality and Integrity in the Design and Construction of Deep Foundations.

Mayne and Woeller (2008)
British Geotechnical Association
(BGA) International Conference
on Foundations (ICOF)

Mayne and Woeller (2010)
ASCE GSP 205
GeoShanghai

Mayne and Woeller (2009)
ASCE GSP 186
International Foundations
Conference & Expo Exhibit
(IFCEE'09, Orlando)

Niazi, Mayne, and
Woeller (2010)
CPT'10

2010 PDCA-DFI Seminar
Baltimore, Maryland

**Evaluating Axial Pile
Response from
Cone Penetrometer Tests**

12 March 2010
Paul W. Mayne
Georgia Institute of Technology

Elastic Pile Solutions (Randolph 1989)

RIGID PILES: Rigid when $L/r_o \leq 0.3 (\frac{E_s}{G_s})^{1/2}$ Then, the top load (P_t) relationship with top displacement (w_t) is given by:

$$\frac{P_t}{G_s r_o w_t} = \frac{4\eta}{(1-\nu)\zeta^2} + \frac{2\pi\rho_g L}{\zeta} \frac{L}{r_o}$$

INTERMEDIATE PILES:

$$\frac{P_t}{G_s r_o w_t} = \frac{4\eta}{(1-\nu)\zeta^2} - \frac{2\pi\rho_g \tanh(\mu L)}{\zeta} \frac{\mu L}{r_o}$$

VERY LONG PILES: when $L/r_o \geq 3 (\frac{E_s}{G_s})^{1/2}$ then load vanishes before length of pile reached. Then for this case $\tan(\mu L)$ approaches 1 and the axial load-displacement relationship is:

$$\frac{P_t}{G_s r_o w_t} = \pi\rho_g \sqrt{\frac{2\lambda}{\zeta}}$$

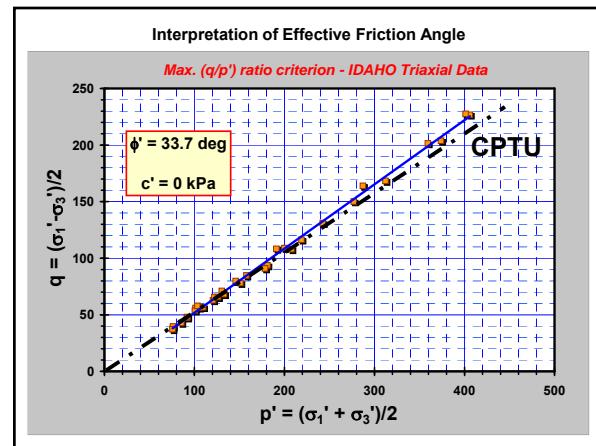
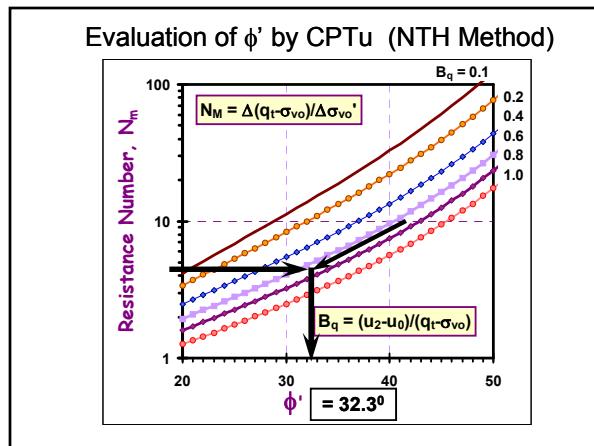
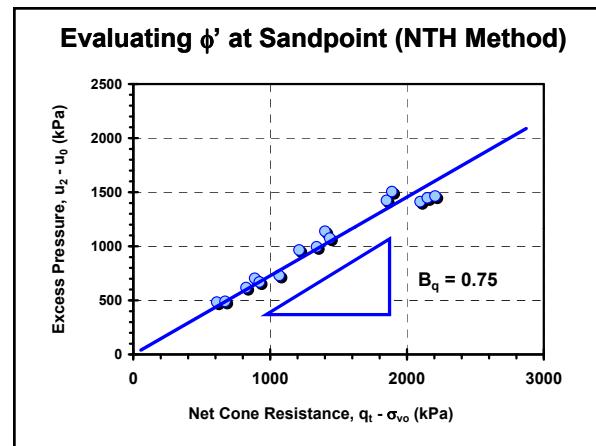
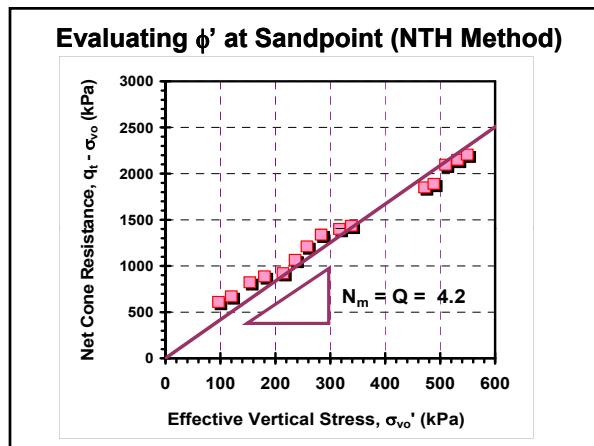
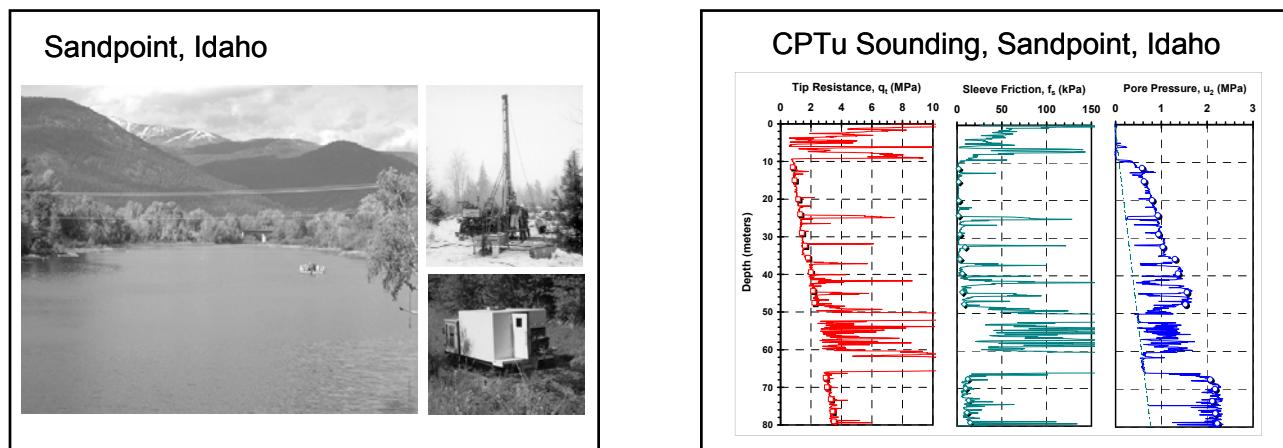
L = pile length
 r_o = pile radius
 ζ = radius of pile base
 ν = v_0 = eta term
 E_s = pile modulus
 G_s = soil shear modulus at $z=1$
 ρ_g = soil shear modulus parameters
 $\lambda = E_s/G_s$ = soil-pile stiffness ratio
 $\lambda = G_s/G_a$ = zeta term
 G_a = soil stiffness below pile tip the base
 $\zeta = \ln(r_o/z)$ = soil zone influence
 $L_p = L/(0.25 + \zeta(2.5 \rho_g(1-\nu) - 0.25))$ = "magic radius"

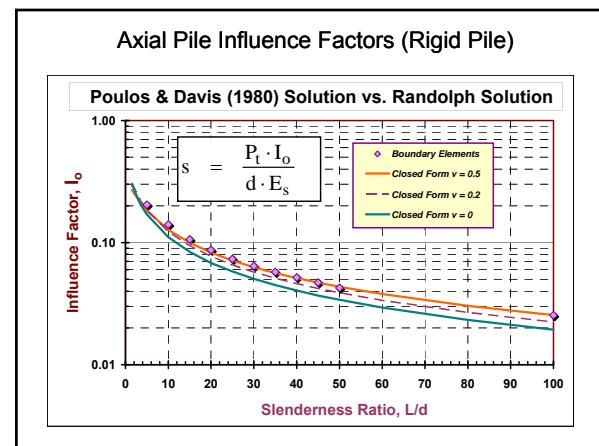
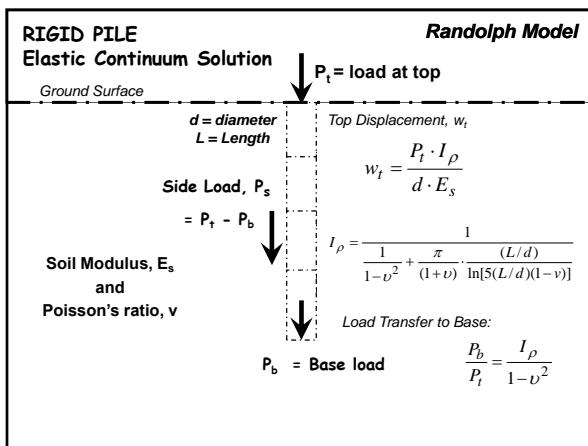
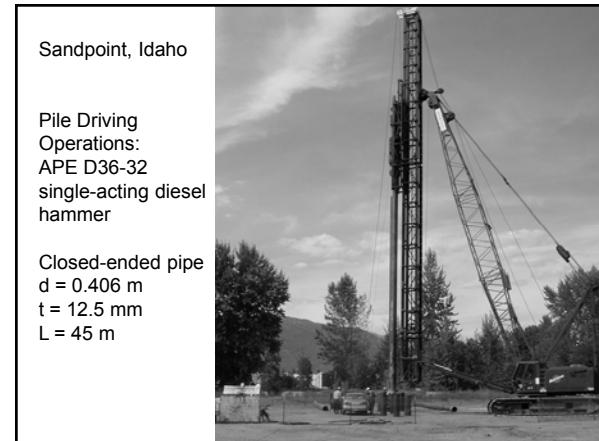
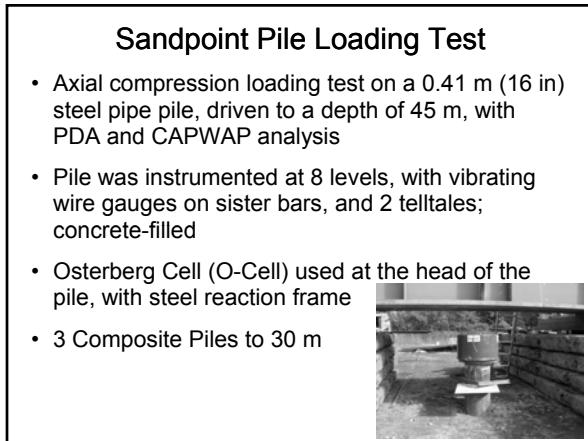
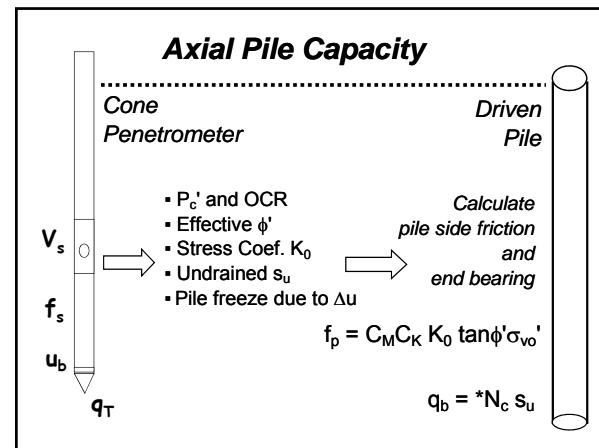
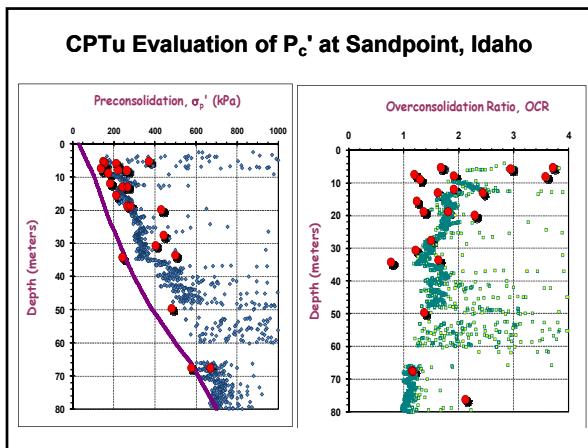
Driven Piles, Sandpoint, Idaho

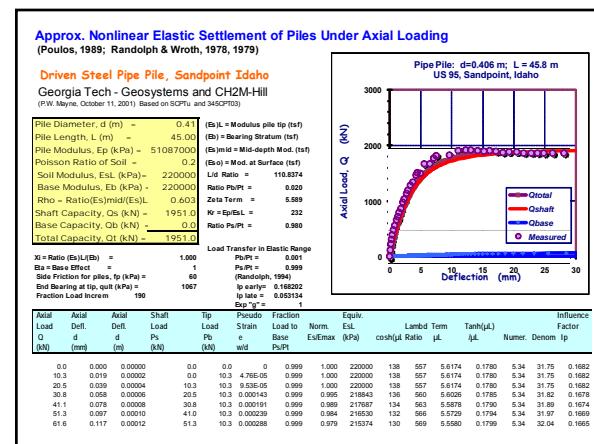
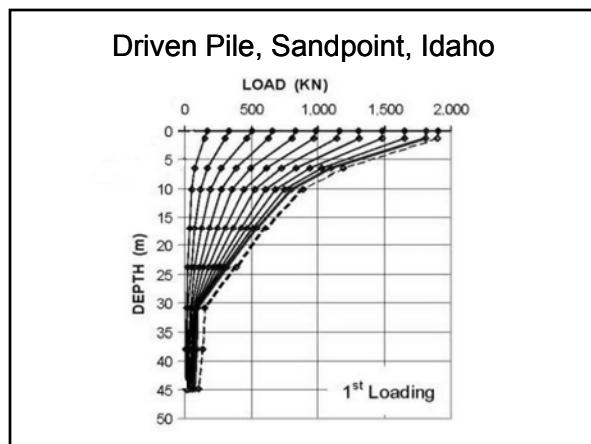
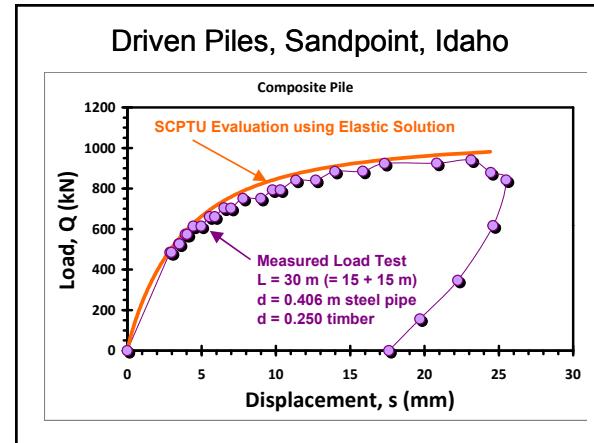
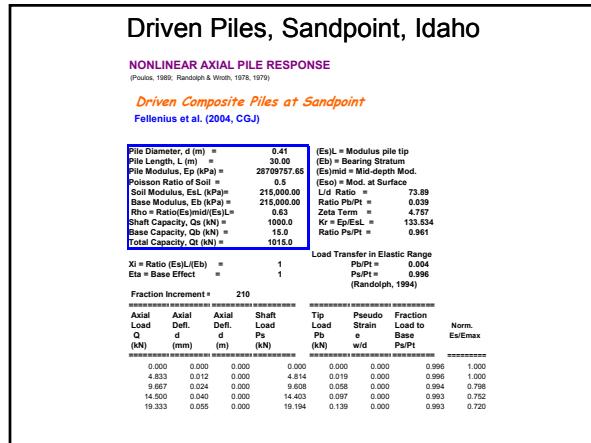
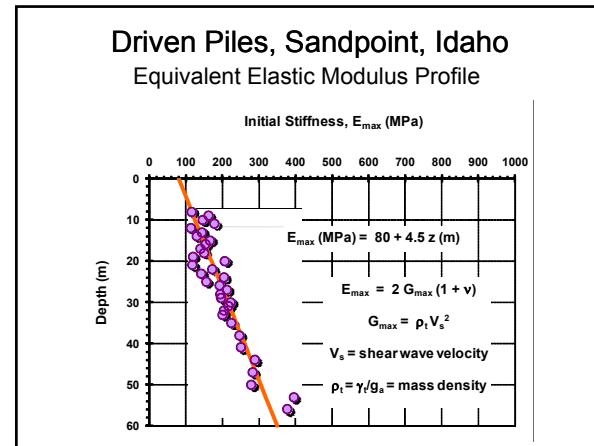
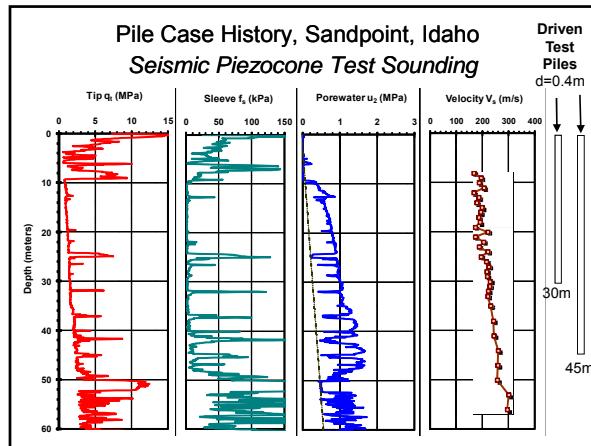
- Sand Creek Byway Project
- Data and load test information courtesy of Dean Harris, CH2M-Hill
- 14th Annual PDCA Conference - Idaho 06-08 May 2010

Sandpoint, Idaho

- Route 95 bridge for IDOT
- Deep deposit of silty clay (> 100 m) with sand lenses & layers
- 80-m deep piezocone test by ConeTec (upper 60 m as SCPTu)
- Driven composite piles (15 m steel pipe over 15 m timber)
- Very long driven 45-m driven pipe pile (Fellenius, et al. 2004, CGJ)

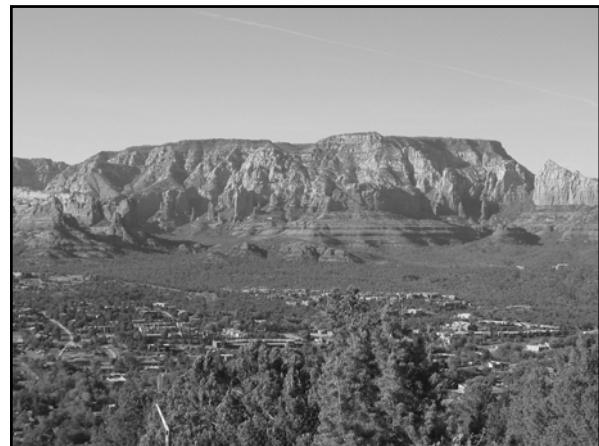






Evaluating Axial Pile Response by CPT

- Elastic continuum solution (Randolph 2003)
- Fundamental initial ground stiffness from shear wave velocity: $G_{\max} = \rho_t V_s^2$
- CPT readings for direct pile capacity evaluation
- Nonlinearity of modulus with FS using algorithm by Fahey (1998)
- Axial load-displacement response and load transfer distributions with depth
- Case studies: Texas, Georgia, Alabama, Alberta, British Columbia, Idaho, and South Carolina



REFERENCES

- ASTM D 3441, "Test Method for Performing Mechanical Cone Penetrometer Testing of Soils", Volume 04.08, American Society for Testing and Materials, West Conshohocken, Pennsylvania, 2000.
- ASTM D 5778, "Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils", Volume 04.08, American Society for Testing and Materials, West Conshohocken, Pennsylvania, Dec. 2007.
- Baldi, G., R. Bellotti, V.N. Ghionna, M. Jamiolkowski, and D.C.F. LoPresti, "Modulus of Sands From CPTs and DMTs. *Proceedings, 12th International Conference on Soil Mechanics & Foundation Engineering*, Vol. 1, Rio de Janeiro, 1989, Balkema, Rotterdam, pp. 165-170.
- Been, K., J.H.A. Crooks, D.E Becker, and M.G. Jefferies, "The Cone Penetration Test in Sands: State Parameter", *Geotechnique*, Vol. 36, No. 2, 1986, pp. 239-249.
- Been, K., B.E. Lingnau, J.H.A. Crooks, and B. Leach, "Cone Penetration Test Calibration for Erksak Beaufort Sea Sand", *Canadian Geotechnical Journal*, Vol. 24, No. 4, 1987, pp. 601-610.
- Begemann, H.K., "The Friction Jacket Cone as an Aide in Determining the Soil Profile", *Proceedings, 6th Intl. Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, Montreal, 1965, pp. 17-20.
- Boulanger, R.W. and I.M. Idriss, "State Normalization of Penetration Resistance and the Effect of Overburden Stress on Liquefaction Resistance", *Proceedings, 11th Conference on Soil Dynamics & Earthquake Engineering*, Berkeley, California, 2004, pp. 484-491.
- Briaud, J-L. and J. Miran, *The Cone Penetrometer Test*, Report No. FHWA-SA-91-043, Federal Highway Administration, Washington, D.C., February 1992, 161 pp.
- Broms, B.B. and N. Flodin, "History of Soil Penetration Testing", *Proceedings of the First International Symposium on Penetration Testing*, Vol. 1, Orlando (*Penetration Testing 1988*, Balkema, Rotterdam), March 20-24, 1988, pp. 157-220.
- Burland, J.B., "Small Is Beautiful: The Stiffness of Soils at Small Strains", *Canadian Geotechnical Journal*, Vol. 26, No. 4, 1989, pp. 499-516.
- Burns, S.E. and P.W. Mayne, "Monotonic and Dilatory Pore Pressure Decay During Piezocone Tests", *Canadian Geotechnical Journal*, Vol. 35, No. 6, 1998, pp. 1063-1073.
- Burns, S.E. and P.W. Mayne, "Analytical Cavity Expansion-Critical State Model for Piezocone Dissipation in Fine-Grained Soils", *Soils & Foundations*, Vol. 42, No. 2, 2002, pp. 131-137.
- Burns, S.E. and P.W. Mayne, "Interpretation of Seismic Piezocone Results for the Evaluation of Hydraulic Conductivity in Clays", *ASTM Geotechnical Testing Journal*, Vol. 25, No. 3, 2002, pp. 333-340.
- Bustamante, M. and L. Gianeselli, "Pile Bearing Capacity Prediction by Means of Static Penetrometer", *Proceedings, European Symposium on Penetration Testing*, Vol. 2, Amsterdam, 1982, pp. 493-500.
- Bustamante, M. and Frank, R., "Design of Axially Loaded Piles - French Practice", *Design of Axially Loaded Piles - European Practice*, (Proc. ERTC3, Brussels), Balkema, Rotterdam, pp. 161-175.
- Campanella, R.G., J.P. Sully, J.W. Greig, and G. Jolly, "Research and Development of a Lateral Stress Piezocone", *Transportation Research Record*, No. 1278, National Academy Press, 1990, pp. 215-224.
- Campanella, R.G., "Field Methods for Dynamic Geotechnical Testing". *Dynamic Geotechnical Testing II* (Special Tech Publication 1213), ASTM, West Conshohocken, PA, 1994, pp. 3-23.

Campanella, R.G., P.K. Robertson, and D. Gillespie, "Seismic Cone Penetration Test", *Use of In-Situ Tests in Geotechnical Engineering* (GSP 6), ASCE, Reston, Virginia, 1986, pp. 116-130.

Campanella, R.G. and P.K. Robertson, "Current Status of the Piezocone Test", *Proceedings of the First International Symposium on Penetration Testing*, Vol. 1, Orlando (*Penetration Testing 1988*, Balkema, Rotterdam), March 20-24, 1988, pp. 93-116.

Campanella, R.G. and I. Weemees, "Development and Use of an Electrical Resistivity Cone for Groundwater Contaminant Studies", *Canadian Geotechnical Journal*, Vol. 27, No. 5, 1990, pp. 557-567.

Campanella, R.G., H. Kristiansen, C. Daniel, and M.P. Davies, "Site Characterization of Soil Deposits Using Recent Advances in Piezocone Technology", *Geotechnical Site Characterization*, Vol. 2, (Proc. ISC-1, Atlanta), Balkema, Rotterdam, 1998, pp. 995-1000.

Casey, T.J. and P.W. Mayne, "Development of an Electrically-Driven Automatic Downhole Seismic Source", *Soil Dynamics and Earthquake Engineering*, Vol. 22, 2002, pp. 951-957.

Chen, B.S-Y. and P.W. Mayne, *Profiling the Overconsolidation Ratio of Clays by Piezocone Tests*, Report No. GIT-CEE GEO-94-1 to National Science Foundation by Geosystems Engineering Group, Georgia Institute of Technology, Atlanta, 1994, 280 pp. Download: www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html

Chen, B.S-Y. and P.W. Mayne, "Statistical Relationships between Piezocone Measurements and Stress History of Clays", *Canadian Geotechnical Journal*, Vol. 33, No. 3, 1996, pp. 488-498.

Dahlberg, R., "Penetration, Pressuremeter, and Screw-Plate Tests in a Preloaded Natural Sand Deposit", *Proceedings of the European Symposium on Penetration Testing*, Vol. 2.2, Stockholm, 1974: pp. 68-87.

DeBeer, E.E., E. Goelen, W.J. Heynen and K. Joustra, "Cone Penetration Test: International Reference Test Procedure", *Penetration Testing 1988*, Vol. 1 (Proc. ISOPT-1, Orlando), Balkema, Rotterdam, 1988, pp. 27-52.

DeGroot, D.J. and A.J. Lutenegger, "Geology and Engineering Properties of Connecticut Valley Varved Clay", *Characterization and Engineering Properties of Natural Soils*, Vol. 1, Swets & Zeitlinger, Lisse, 2003, pp. 695-724.

DeGroot, D.J. and R. Sandven, "General Report: Laboratory and Field Comparisons", *Geotechnical & Geophysical Site Characterization*, Vol. 2, (ISC-2, Porto), Millpress, Rotterdam, 2004, pp. 1775-1789,

DeJong, J.T. and J.D. Frost, "A Multi-Friction Sleeve Attachment for the Cone Penetrometer", *ASTM Geotechnical Testing Journal*, Vol. 25, No. 2, 2002, pp. 111-127.

Demers, D., and S. Leroueil, "Evaluation of Preconsolidation Pressure and the Overconsolidation Ratio from Piezocone Tests of Clay Deposits in Quebec", *Canadian Geotechnical Journal*, Vol. 39, No. 1, 2002, pp. 174-192.

DeRuiter, J., "Electric Penetrometer for Site Investigations", *Journal of the Soil Mechanics and Foundations Division* (ASCE), Vol. 97, No. SM2, 1971, pp. 457-472.

DeRuiter, J., "Current Penetrometer Practice", *Cone Penetration Testing and Experience*, (Proc. ASCE National Convention, St. Louis), American Society of Civil Engineers, Reston, Virginia, 1981, pp. 1-48.

DeRuiter, J., "The Static Cone Penetration Test", *Proceedings of the Second European Symposium on Penetration Testing*, Vol. 2 (Amsterdam), May 24-27, 1982, Balkema, Rotterdam, pp. 389-405.

Elmgren, K., "Slot-Type Pore Pressure CPTu Filters", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 9-12.

Elsworth, D., "Analysis of Piezocone Data using Dislocation Based Methods", *Journal of Geotechnical Engineering*, Vol. 119, No. 10, 1993, pp. 1601-1623.

Eslami, A. and B.H. Fellenius, "Toe Bearing Capacity of Piles from CPT Data", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 453-460.

Eslami, A. and B.H. Fellenius, "Pile Capacity by Direct CPT and CPTu Methods Applied to 102 Case Histories", *Canadian Geotechnical Journal*, Vol. 34, No. 6, 1997, pp. 880-898.

Eslami, A., "Bearing Capacity of Shallow and Deep Foundations from CPT Resistance", *Proceedings, GeoCongress*, (Atlanta), ASCE, Reston, Virginia, Feb. 26-Mar. 1, 2006, 6 pp.

Fahey, M. and J.P. Carter, "A Finite Element Study of the Pressuremeter in Sand Using a Nonlinear Elastic Plastic Model", *Canadian Geotechnical Journal*, Vol. 30, No. 2, 1993, pp. 348-362.

Fahey, M., P.K. Robertson, and A.A. Soliman, "Towards a Rational Method of Predicting Settlements of Spread Footings on Sand", *Vertical and Horizontal Deformations of Foundations and Embankments*, Vol. 1, GSP 40, ASCE, Reston, Virginia, 1994, pp. 598-611.

Farrington, S.P., "Development of a Wireline CPT System for Multiple Tool Usage", *Proceedings, Industry Partnerships for Environmental Science and Technology*, Department of Engineering, Oct. 17-19, 2000, 40 pp.

Farrington, S.P. and J.D. Shinn, "Hybrid Penetration for Geotechnical Site Investigation", *Proceedings, GeoCongress 2006*, Atlanta, published by ASCE, Reston, Virginia, 5 pp.

Finke, K.A. and P.W. Mayne, "Piezocone Response in Piedmont Residual Geomaterials", *Behavioral Characteristics of Residual Soils*, (GSP 92), ASCE, Reston, Virginia, 1999, pp. 1-11.

Fioravante, V. et al. (1995). Load carrying capacity of large diameter bored piles in sand and gravel. *Proceedings, 10th Asian Regional Conference on Soil Mechanics & Foundation Engineering*.

Fioravante, V., M. Jamiołkowski, V.N. Ghionna, and S. Pedroni, "Stiffness of Carbonatic Quiou Sand from CPT", *Geotechnical Site Characterization*, Vol. 2, Balkema, Rotterdam, 1998, pp. 1039-1049.

Frank, R. and J-P. Magnan, "Cone Penetration Testing in France: National Report", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 3, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 147-156.

Ghionna, V.N. and M. Jamiołkowski, "A Critical Appraisal of Calibration Chamber Testing of Sands", *Calibration Chamber Testing* (Proc. ISOCCT-1, Potsdam), Elsevier, New York, 1991, pp. 13-40.

Gorman, C.T., V.P. Drnevich, and T.C. Hopkins, "Measurement of In-Situ Shear Strength", *In-Situ Measurement of Soil Properties*, Vol. II, ASCE, Reston, Virginia, 1975, pp. 139-140.

Gotman, A., "Tapered Pile Bearing Capacity Calculation by Static Sounding Data", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 467-472.

Gupta, R.C. and J.L. Davidson, "Piezoprobe Determined Coefficient of Consolidation", *Soils & Foundations*, Vol. 26, No. 3, 1986, pp. 12-22.

Hebeler, G.L., J.D. Frost, and J.D. Shinn, "A Framework for Using Textured Friction Sleeves at Sites Traditionally Problematic for CPT", *Geotechnical and Geophysical Site Characterization*, Vol. 1 (Proc. ISC-2, Porto), Millpress, Rotterdam, 2004, pp. 693-699.

Hegazy, Y.A., *Delineating Geostratigraphy by Cluster Analysis of Piezocone Data*, PhD dissertation, School of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, GA, June 1998, 464 pp.

Hegazy, Y.A. and P.W. Mayne, "Statistical Correlations Between V_s and CPT Data for Different Soil Types". *Proceedings, Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society, Linköping, 1995, pp. 173-178.

Hight, D. and S. Leroueil, "Characterization of Soils for Engineering Purposes", *Characterization and Engineering Properties of Natural Soils*, Vol. 1, Swets and Zeitlinger, Lisse, 2003, pp. 255-360.

Houlsby, G.T. and C.I. Teh, "Analysis of the Piezocone in Clay", *Penetration Testing 1988*, Vol. 2, Balkema, Rotterdam, 1988, pp. 777-783.

Howie, J.A., C. Daniel, A.A. Asalemi, and R.G. Campanella, "Combinations of In-Situ Tests for Control of Ground Modification in Silts and Sands", *Innovations and Applications in Geotechnical Site Characterization*, (GSP No.97), ASCE, Reston, Virginia, 2000, pp. 181-198.

Hryciw, R.D., A.M. Ghalib, and S.A. Raschke, "In-Situ Soil Characterization Using Vision Cone Penetrometer", *Geotechnical Site Characterization*, Vol. 2, (Proc. ISC-1, Atlanta), Balkema, Rotterdam, 1998, pp. 1081-1086.

Hryciw, R.D. and S. Shin, "Thin Layer and Interface Characterization by VisCPT", *Geotechnical & Geophysical Site Characterization*, Vol. 1, (Proc. ISC-2, Porto), Millpress, Rotterdam, 2004, pp. 701-706.

Jamiolkowski, M., C.C. Ladd, J.T. Germaine, and R. Lancellotta, "New Developments in Field and Lab Testing of Soils", *Proceedings, 11th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, San Francisco, August 12-16, 1985, pp. 57-154.

Jamiolkowski, M. and P.K. Robertson, "Future Trends for Penetration Testing", *Penetration Testing in the UK*, Thomas Telford, London, 1988, pp. 321-342.

Jamiolkowski, M., V.N. Ghionna, R. Lancellotta, and E. Pasqualini, "New Correlations of Penetration Tests for Design Practice", *Penetration Testing 1988*, Vol. 1, (Proc. ISOPT-1, Orlando), Balkema, Rotterdam, 1988, pp. 263-296.

Jamiolkowski, M., "Opening Address: CPT'95", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 1, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 7-15.

Jamiolkowski, M., "Where Are We Going", *Pre-Failure Deformation Characteristics of Geomaterials*, Vol. 2 (Proc. Torino '99), Swets & Zeitlinger, Lisse, 2001, pp. 1251-1262.

Jamiolkowski, M. and M.C. Pepe, "Vertical Yield Stress of Pisa Clay from Piezocone Tests", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 10, 2001, pp. 893-897.

Jamiolkowski, M., D.C.F. LoPresti, and M. Manassero, "Evaluation of Relative Density and Shear Strength of Sands from Cone Penetration Test and Flat Dilatometer Test", *Soil Behavior and Soft Ground Construction* (GSP 119), ASCE, Reston, Virginia, 2001, pp. 201-238.

Jamiolkowski, M., "Soil Parameters Relevant to Bored Pile Design from Laboratory and In-Situ Tests", *Deep Foundations on Bored and Auger Piles*, Millpress, Rotterdam, 2003, pp. 83-100.

Jardine, R.J., D.M. Potts, A.B. Fourie, and J.B. Burland, "Studies of the Influence of Non-Linear Stress-Strain Characteristics in Soil-Structure Interaction", *Geotechnique* Vol. 36, No. 3, 1986, pp. 377-396.

Jardine, R.J., J.R. Standing, and N. Kovacevic, "Lessons Learned From Full Scale Observations and the Practical Application of Advanced Testing & Modeling", *Deformation Characteristics of Geomaterials*, Vol. 2, Taylor & Francis Group, London, 2005, pp. 201-245.

Jardine, R.J., F. Chow, R. Overy, and J. Standing, *ICP Design Methods for Driven Piles in Sands and Clays*, Thomas Telford Ltd., London, 2005, 105 pp.

Jefferies, M.G. and M.P. Davies, "Use of CPTu to Estimate Equivalent SPT N60", *ASTM Geotechnical Testing Journal*, Vol. 16, No. 4, December 1993, pp. 458-468.

Juran, I. and M.T. Tumay, "Soil Stratification Using the Dual-Element Pore-Pressure Piezocone Test", *Transportation Research Record* 1235, 1989, pp. 68-78.

Karlsrud, K., C.J.F. Clausen, and P.M. Aas, "Bearing Capacity of Driven Piles in Clay: the NGI Approach", *Frontiers in Offshore Geotechnics* (Proc. ISFOG, Perth), Taylor & Francis Group, London, 2005, pp. 775-782.

Keaveny, J.M. and J.K. Mitchell, "Strength of Fine-Grained Soils Using the Piezocone", *Use of In-Situ Tests in Geotechnical Engineering* (GSP 6), ASCE, Reston/VA, 1986, pp. 668-699.

Kolk, H.J., A.E. Baaijens, and M. Senders, "Design Criteria for Pipe Piles in Silica Sands", *Frontiers in Offshore Geotechnics* (Proc. ISFOG, Perth), Taylor & Francis Group, London, 2005, pp. 711-716.

Konrad, J-M. and K.T. Law, "Undrained Shear Strength From Piezocone Tests", *Canadian Geotechnical Journal*, Vol. 24, No. 3, 1987, pp. 392-405.

Kulhawy, F.H., C.H. Trautmann, J.F. Beech, T.D. O'Rourke, and W. McGuire, *Transmission Line Structure Foundations for Uplift-Compression Loading*, Report EL-2870. Electric Power Research Institute, Palo Alto, 1983, 412 pp.

Kulhawy, F.H. and P.W. Mayne, *Manual on Estimating Soil Properties for Foundation Design*. Report EPRI EL-6800, Electric Power Research Institute, Palo Alto, 1990, 306 pp.

Ladd, C.C., "Stability Evaluation During Staged Construction", The 22nd Terzaghi Lecture, *Journal of Geotechnical Engineering* 117, No. 4, 1991, pp. 540-615.

Ladd, C.C. and D.J. DeGroot, "Recommended Practice for Soft Ground Site Characterization", *Soil & Rock America 2003*, (Proc. 12th Pan American Conf., MIT), Verlag Glückauf, Essen, 2003, pp. 3-57.

Larsson, R., "Use of a Thin Slot as Filter in Piezocone Tests", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 35-40.

Larsson, R. and M. Mulabdić, *Piezocene Tests in Clay*, Report No. 42, Swedish Geotechnical Institute, Linköping, 1991, 240 pp.

Lee, L.T., P.G. Malone, and G.E. Robitaille, "Grouting on Retraction of Cone Penetrometer", *Geotechnical Site Characterization*, Vol. 1, (Proc. ISC-1, Atlanta), Balkema, Rotterdam, 1998, pp. 641-643.

Lee, J.H. and R. Salgado, "Determination of Pile Base Resistance in Sands", *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 125, No. 8, 1999, pp. 673-683.

Lehane, B. and E. Cosgrove, "Applying Triaxial Compression Stiffness Data to Settlement Prediction of Shallow Foundations", *Geotechnical Engineering*, Vol. 142, Inst. Civil Engrs, Oct. 2000, pp. 191-200.

Lehane, B.M., J.A. Schneider, and X. Xu, *A Review of Design Methods for Offshore Driven Piles in Siliceous Sand*, Report GEO: 05358, University of Western Australia, Perth, Sept. 2005, 105 pp.

Leroueil, S., G. Bouclin, F. Tavenas, L. Beregeron, and P. LaRochelle, "Permeability Anisotropy of Natural Clays as a Function of Strain", *Canadian Geotechnical Journal*, Vol. 27 (5), Oct. 1990. pp. 568-579.

Leroueil, S. and M. Jamiolkowski, "Exploration of Soft Soil and Determination of Design Parameters", *Proceedings, GeoCoast'91*, Yokohama, Vol. 2, Port & Harbour Research Institute, 1991, pp. 969-998.

Leroueil, S., G. Martel, D. Demers, D. Virely, and P. LaRochelle, "Practical Use of the Piezocone in Eastern Canada", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 515-522.

Leroueil, S. and D. Hight, "Behavior and Properties of Natural Soils and Soft Rocks", *Characterization and Engineering Properties of Natural Soils*, Vol. 1, Swets and Zeitlinger, Lisse, 2003, pp. 29-254.

Lunne, T., T. Eidsmoen, T., D. Gillespie, and J.D. Howland, "Laboratory and Field Evaluation of Cone Penetrometers", *Use of In-Situ Tests in Geotechnical Engineering* (GSP 6), ASCE, Reston, Virginia, 1986, pp. 714-729.

Lunne, T., S. Lacasse, S. and R.S. Rad, "General Report: SPT, CPT, PMT, and Recent Developments in In-Situ Testing", *Proceedings, 12th Intl. Conf. Soil Mechanics & Foundation Engineering*, Vol. 3, Rio de Janeiro, 1994, pp. 2339-2403.

Lunne, T. and J. Keaveny, "Technical Report on Solution of Practical Problems Using CPT", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 3, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 119-138.

Lunne, T., J.J.M. Powell, and P.K. Robertson, "Use of Piezocone Tests in Non-Textbook Materials", *Advances in Site Investigation Practice*, Thomas Telford Ltd, London, 1996, pp. 438-451.

Lunne, T., P.K. Robertson, and J.J.M. Powell, *Cone Penetration Testing in Geotechnical Practice*. Blackie Academic, EF Spon/Routledge Publishers, New York, 1997, 312 pp.

Lunne, T., "In-Situ Testing in Offshore Geotechnical Investigations", *Proceedings, International Conference on In-Situ Measurement of Soil Properties and Case Histories*, Bali, 2001, pp. 61-81.

Lunne, T., M. Long, and C.F. Forsberg, "Characterization and Engineering Properties of Holmen, Drammen Sand", *Characterisation and Engineering Properties of Natural Soils*, Vol. 2 (Proc. Singapore), Swets & Zeitlinger, Lisse, 2003, pp. 1121-1148.

Lunne, T., M.F. Randolph, S.F. Chung, K.H. Andersen, and M. Sjursen, "Comparison of Cone and T-Bar Factors in Two Onshore and One Offshore Clay Sediments", *Frontiers in Offshore Geotechnics* (Proc. ISFOG, Perth), Taylor & Francis Group, London, 2005, pp. 981-989.

Lutenegger, A.J., M.G. Kabir, and S.R. Saye, "Use of Penetration Tests to Predict Wick Drain Performance in Soft Clay", *Penetration Testing 1988*, Vol. 2, Balkema, Rotterdam, 1988, pp. 843-848.

Lutenegger, A.J. and D.J. DeGroot, "Techniques for Sealing Cone Penetrometer Holes", *Canadian Geotechnical Journal*, Vol. 32, No. 5, 1995, pp. 880-891.

Lutenegger, A.J., D.J. DeGroot, C. Mirza, and M. Bozozuk, "Recommended Guidelines for Sealing Geotechnical Exploratory Holes", *Report 378*, National Cooperative Highway Research Program, Transportation Research Board, National Academy Press, Washington, D.C., 1995, 52 pp.

Mayne, P.W., "CPT-Based Prediction of Footing Response. *Measured & Predicted Behavior of Five Spread Footings on Sand* (GSP 41)", ASCE, Reston, Virginia, 1994, pp. 214-218.

Mayne, P.W., "Stress-Strain-Strength-Flow Parameters from Enhanced In-Situ Tests", *Proceedings, International Conference on In-Situ Measurement of Soil Properties and Case Histories*, Bali, Indonesia, 2001, pp. 27-48.

Mayne, P.W., "Class-A Footing Response Prediction from Seismic Cone Tests", *Deformation Characteristics of Geomaterials*, Vol. 1 (Proc. IS-Lyon), Swets & Zeitlinger, Lisse, 2003, pp. 883-888.

Mayne, P.W., "Integrated Ground Behavior: In-Situ and Lab Tests", *Deformation Characteristics of Geomaterials*, Vol. 2 (Proc. Lyon), Taylor & Francis, London, 2005, pp. 155-177.

Mayne, P.W., "The 2nd James K. Mitchell Lecture: Undisturbed Sand Strength from Seismic Cone Tests", *Geomechanics & Geoengineering*, Vol. 1, No. 4, Taylor & Francis Group, London, 2006.

Mayne, P.W., "In-Situ Test Calibrations for Evaluating Soil Parameters", Overview Paper, *Characterization & Engineering Properties of Natural Soils*, Vol. 3 (Proc. Singapore Workshop), Taylor & Francis Group, London, 2006.

Mayne, P.W. (2007). *NCHRP Synthesis 368: Cone Penetration Testing*, Transportation Research Board, National Academies Press, Washington, DC. 118 pages.

Mayne, P.W. and Campanella, R.G., "Versatile Site Characterization by Seismic Piezocone", *Proceedings, 16th International Conference on Soil Mechanics and Geotechnical Engineering*, Vol. 2 (Osaka), Millpress, Rotterdam, 2005, pp. 721-724.

Mayne, P.W., B.R. Christopher, and J. DeJong, *Subsurface Investigations: Geotechnical Site Characterization*, Pub. No. FHWA NHI-01-031, National Highway Institute, Arlington, Virginia, 2002, 300 pp. Download from: <http://www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html>

Mayne, P.W. and A. Elhakim, "Axial Pile Response Evaluation by Geophysical Piezocone Tests", *Proceedings, Ninth International Conference on Piling and Deep Foundations*, Nice, France, June 3-5, 2002, pp. 543-550.

Mayne, P.W. and G.J. Rix, "Correlations Between Shear Wave Velocity and Cone Tip Resistance in Clays", *Soils & Foundations*, Vol. 35, No. 2, 1995, pp. 107-110.

Mayne, P.W. and H.E. Stewart, "Pore Pressure Response of K_0 -Consolidated Clays", *Journal of Geotechnical Engineering*, Vol. 114, No. 11, 1988, pp. 1340-1346.

Mayne, P.W., F.H. Kulhawy, and J.N. Kay, "Observations on the Development of Porewater Pressures During Piezocone Testing in Clays", *Canadian Geotechnical Journal*, Vol. 27, No. 4, 1990, pp. 418-428.

Mayne, P.W., J.K. Mitchell, J.A Auxt, and R. Yilmaz, "U.S. National Report on CPT", *Proc. Intl. Symposium on Cone Penetration Testing*, Vol. 1, Swedish Geotechnical Society, Report 3:95, Linköping, 1995, pp. 263-276.

Mayne, P.W., P.K. Robertson, and T. Lunne, "Clay Stress History Evaluated from Seismic Piezocone Tests", *Geotechnical Site Characterization*, Vol. 2, Balkema, Rotterdam, 1998, pp. 1113-1118.

Mayne, P.W. and H.G. Poulos, "Approximate Displacement Influence Factors for Elastic Shallow Foundations", *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 125, No. 6, 1999, pp. 453-460.

Mayne, P.W. and J.A. Schneider, "Evaluating Axial Drilled Shaft Response by Seismic Cone", *Foundations & Ground Improvement*, GSP 113, ASCE, Reston, Virginia, 2001, pp. 655-669.

Mayne, P.W. and G. Zavala, "Axial Shaft Response from Seismic Piezocone Tests". *GeoSupport 2004*, GSP 124, Joint ADSC-ASCE GeoInstitute Conference, Orlando, 2004, pp. 429-440.

McGillivray, A.V., T. Casey, P.W. Mayne, and J.A. Schneider, "An Electro-Vibrocone for Site-Specific Evaluation of Soil Liquefaction Potential", *Innovations and Applications in Geotechnical Site Characterization*, (GSP No.97), ASCE, Reston, Virginia, 2000, pp. 106-117.

Mesri, G. and M. Abdel-Ghaffar, "Cohesion Intercept in Effective Stress Stability Analysis", *Journal of Geotechnical Engineering*, Vol. 119, No. 8, 1993, pp. 1229-1249.

Mimura, M., "Characteristics of Some Japanese Natural Sands – Data from Undisturbed Frozen Samples", *Characterisation and Engineering Properties of Natural Soils*, Vol. 2 (Proc. Singapore), Swets & Zeitlinger, Lisse, 2003, pp.1149-1168.

Mitchell, J.K. and W.S. Gardner, "In-Situ Measurement of Volume Change Characteristics", *In-Situ Measurement of Soil Properties*, Vol. II (Proc. Raleigh Conf.), ASCE, Reston, VA, 1975, pp. 279-345.

Mitchell, J.K. and T. Lunne, "Cone Resistance as a Measure of Sand Strength", *Journal of Geotechnical Engineering*, Vol. 104 (GT7), 1978, pp. 995-1012.

Mitchell, J.K. and Z.V. Solymar, "Time-Dependent Strength Gain in Freshly Deposited or Densified Sand", *Journal of Geotechnical Engineering*, Vol. 110, No. 11, 1984, pp. 1559-1576.

Mlynarek, Z., E. Welling, and W. Tschuschke, "Conductivity Piezocone Penetration Test for Evaluation of Soil Contamination", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 233-237.

Moss, R.E.S., R.B. Seed, R.E. Kayen, J.P. Stewart, and A. Der Kiureghian, "CPT-Based Probabilistic Assessment of Seismic Soil Liquefaction Initiation", *Report No. PEER-2003/xx*, Pacific Earthquake Engineering Research, September 2003, 70 pp.

Moss, R.E.S., Seed, R.B. and Olsen, R.S. 2006. Normalizing the CPT for overburden stress. *Journal of Geotechnical & Geoenvironmental Engrg.* 132 (3): pp. 378-387.

Moss, R.E.S., R.B. Seed, R.E. Kayen, J.P. Stewart, A. Der Kiureghian, and K.O. Cetin, "CPT-Based Probabilistic and Deterministic Assessment of In-Situ Seismic Soil Liquefaction Potential", *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 132 (8), Aug. 2006, pp. 1032-1051.

Mulabdić, M., S. Eskilson, and R. Larsson, "Calibration of Piezocones for Investigations in Soft Soils and Demands for Accuracy of the Equipment", *Report Varia 270*, Swedish Geotechnical Institute, Linköping, 62 pp.

Nash, D.F.T., J.J.M. Powell and I.M. Lloyd, "Initial Investigation of the soft clay test site at Bothkennar", *Geotechnique* 42 (2), June 1992: pp. 163-183.

Olsen, R.S. and J.K. Mitchell, "CPT Stress Normalization and Prediction of Soil Classification", *Proceedings International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society, Linköping, 1995, pp. 257-262.

Parez, L. and R. Faureil, "Le Piézocone. Améliorations Apportées à la Reconnaissance de Sols", *Revue Française de Géotech.*, Vol. 44, 1988, pp. 13-27.

Peuchen, J., "Commercial CPT Profiling in Soft Rocks and Hard Soils", *Geotechnical Site Characterization*, Vol. 2, (Proc. ISC-1, Atlanta), Balkema, Rotterdam, 1998, pp. 1131-1137.

Poulos, H.G. and E.H. Davis, *Elastic Solutions for Soil and Rock Mechanics*, John Wiley & Sons, New York, 1974, 441 pp.

Poulos, H.G. and E.H. Davis, *Pile Foundation Analysis and Design*, John Wiley & Sons, New York, 1980, 397 pp.

Poulos, H.G., "Pile Behavior: Theory and Applications" (Rankine Lecture), *Geotechnique*, Vol. 39, No. 3, 1989, pp. 363-415.

Powell, J.J.M, R.S.T. Quarterman, and T. Lunne, "Interpretation and Use of the Piezocone Test in UK Clays", *Penetration Testing in the UK*, Thomas Telford, London, 1988, pp. 151-156.

Powell, J.J.M. and R.S.T. Quarterman, "The Interpretation of Cone Penetration Tests in Clays with Particular Reference to Rate Effects", *Penetration Testing 1988*, Vol. 2, (Orlando), Balkema, Rotterdam, 1988, pp. 903-909.

Powell, J.J.M., T. Lunne, and R. Frank, R., "Semi-Empirical Design for Axial Pile Capacity in Clays", *Proceedings, 15th International Conference on Soil Mechanics and Geotechnical Engineering*, Vol. 2, (Istanbul), Balkema, Rotterdam, 2001, pp. 991-994.

Randolph, M.F. and C.P. Wroth, "Analysis of Deformation of Vertically-Loaded Piles", *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 104, No. GT12, 1978, pp. 1465-1488.

Randolph, M.F. and C.P. Wroth, "A Simple Approach to Pile Design and the Evaluation of Pile Tests", *Behavior of Deep Foundations*, STP 670, ASTM, West Conshohocken, PA, 1979, pp. 484-499.

Randolph, M.F., "Characterization of Soft Sediments for Offshore Applications", *Geotechnical & Geophysical Site Characterization*, Vol. 1 (Proc. ISC-2, Porto), Millpress, Rotterdam, 2004, pp. 209-232.

Randolph, M.F., M. Cassidy, S. Gourvenec, and C. Erbrich, "Challenges of Offshore Geotechnical Engineering", *Proceedings, 16th International Conference on Soil Mechanics and Geotechnical Engineering*, Vol. 1, Osaka, pp. 123-176.

Robertson, P.K., "In-Situ Testing and Its Application to Foundation Engineering", *Canadian Geotechnical Journal*, Vol. 23, No. 4, 1986, pp. 573-594.

Robertson, P.K., "Soil Classification Using the Cone Penetration Test", *Canadian Geotechnical Journal*, Vol. 27, No. 1, 1990, pp. 151-158.

Robertson, P.K., Sixty Years of the CPT - How Far Have We Come?", *Proceedings, International Conference on In-Situ Measurement of Soil Properties and Case Histories*, Bali, Indonesia, May 21-24, 2001, pp. 1- 16.

Robertson, P.K. and R.G. Campanella, "Interpretation of Cone Penetration Tests: Sands", *Canadian Geotechnical Journal*, Vol. 20, No. 4, 1983, pp. 719-733.

Robertson, P.K., R.G. Campanella, and A. Wightman, "SPT-CPT Correlations", *Journal of the Geotechnical Engineering Division* (ASCE), Vol. 108, No. GT 11, 1983, pp. 1449-1459.

Robertson, P.K. and R.G. Campanella, *Guidelines for Use & Interpretation of the Electronic Cone Penetration Test*, Hogentogler & Company, Inc., Gaithersburg, MD, 1984, 154 pp.

Robertson, P.K., R.G. Campanella, D. Gillespie, and J. Greig, "Use of Piezometer Cone Data", *Use of In-Situ Tests in Geotechnical Engineering*, GSP 6, ASCE, Reston, Virginia, 1986, pp. 1263-1280.

Robertson, P.K., R.G. Campanella, M.P. Davies, and A. Sy, "Axial Capacity of Driven Piles in Deltaic Soils Using CPT", *Penetration Testing 1988*, Vol. 2, Balkema, Rotterdam, 1988, pp. 919-928.

Robertson, P.K., J.P. Sully, D.J. Woeller, T. Lunne, J.J.M. Powell, and D.G. Gillespie, "Estimating Coefficient of Consolidation from Piezocone Tests", *Canadian Geotechnical Journal*, Vol. 39, No. 4, 1992, pp. 539-550.

Robertson, P.K., T. Lunne, and J.J.M. Powell, "GeoEnvironmental Applications of Penetration Testing", *Geotechnical Site Characterization*, Vol. 1, (Proc. ISC-1, Atlanta), Balkema, Rotterdam, 1998, pp. 35-48.

Robertson, P.K. and C.E. Wride, "Evaluating Cyclic Liquefaction Potential Using the Cone Penetration Test". *Canadian Geotechnical Journal*, Vol. 35, No. 3, 1998, pp. 442-459.

Robertson, P.K., C.E. Wride, et al., "The CANLEX Project: Summary & Conclusions", *Canadian Geotechnical Journal*, Vol. 37, No. 3, 2000, pp. 563-591.

Salgado, R., R.W. Boulanger, and J.K. Mitchell, "Lateral Stress Effects on CPT Liquefaction Correlations", *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 123, No. 8, 1997, pp. 726-735.

Salgado, R., J.K. Mitchell, and M. Jamiolkowski, "Cavity Expansion and Penetration Resistance in Sand", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 123, No. 4, 1997, pp. 344-354.

Salgado, R., J.K. Mitchell, and M. Jamiolkowski, "Calibration Chamber Size Effects on Penetration Resistance in Sand", *J. Geotechnical and Geoenvironmental Engineering*, Vol. 124 (9), 1998, 878-888.

Schmertmann, J.H., *Guidelines for Cone Penetration Test: Performance and Design*. Report FHWA-TS-78-209, Federal Highway Administration, Washington, D.C., 1978, 146 pp.

Schnaid, F., "Geocharacterization and Engineering Properties of Natural Soils by In-Situ Tests", *Proceedings, 16th International Conference on Soil Mechanics and Geotechnical Engineering*, Vol. 1 (Osaka), September 12-16, 2005, Millpress, Rotterdam, pp. 3-45.

Schnaid, F., B.M. Lehane, and M. Fahey, "In-Situ Test Characterization of Unusual Geomaterials", *Geotechnical & Geophysical Site Characterization*, Vol. 1 (Proc. ISC-2, Porto), Millpress, Rotterdam, 2004, pp. 49-74.

Schnaid, F., G.C. Gills, J.M. Soares, and Z. Nyirenda, "Predictions of the Coefficient of Consolidation from Piezocone Tests", *Canadian Geotechnical Journal*, Vol. 34, No. 2, 1997, pp. 315-327.

Schneider, J.A. and P.W. Mayne, "Ground Improvement Assessment Using SCPTUs and Crosshole Data," *Innovations & Applications in Geotechnical Site Characterization* (GSP 97), ASCE, Reston, Virginia, 2000, pp. 169-180.

Senneset, K., "Penetration Testing in Norway", *Proceedings of the European Symposium on Penetration Testing*, Vol. 1, Swedish Geotechnical Society, Stockholm, June 5-7, 1974, pp. 85-95.

Senneset, K., R. Sandven, T. Lunne, T. By, and T. Amundsen, "Piezocone Tests in Silty Soils", *Penetration Testing 1988*, Vol. 2, Balkema, Rotterdam, 1988, pp. 955-974.

Senneset, K., R. Sandven, and N. Janbu, "Evaluation of Soil Parameters from Piezocone Tests", *Transportation Research Record 1235*, National Academy Press, Washington, D.C, 1989, pp. 24-37.

Shinn, J.D. and W.L. Bratton, "Innovations with CPT for Environmental Site Characterization", *Proceedings, International Symposium on Cone Penetration Testing*, Vol. 2, Swedish Geotechnical Society Report 3:95, October 4-5, 1995, Linköping, pp. 93-98.

Sills, G.C., M.S.S. Almeida, and F. Danzinger, "Coefficient of Consolidation from Piezocone Dissipation Tests in a Very Soft Clay", *Penetration Testing 1988*, Vol. 2, Balkema, Rotterdam, 1988, pp. 967-974.

Skomedal, E. and J.M. Bayne, "Interpretation of Pore Pressure Measurements from Advanced Cone Penetration Testing", *Penetration Testing in the UK*, Thomas Telford, London, 1988, pp. 279-283.

Sully, J.P. and R.G. Campanella, "Evaluation of Field CPTU Dissipation Data in Overconsolidated Fine-Grained Soils", *Proceedings, 13th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 1, New Delhi, 1994, pp. 201-204.

Tanaka, H. and M. Tanaka, "Key Factors Governing Sample Quality", *Characterization of Soft Marine Clays*, T. Tsuchida, and A. Nakase, Eds, A.A. Balkema, Rotterdam, 1999, pp. 57-81.

Tatsuoka, F. and S. Shibuya, *Deformation Characteristics of Soils and Rocks from Field and Laboratory Tests*, Report of the Institute of Industrial Science, Vol. 37 (1), Univ. of Tokyo, 1992, 136 pp.

Teh, C.I. and G.T. Housby, "An Analytical Study of the Cone Penetration Test in Clay", *Geotechnique* Vol. 41, No. 1, 1991, pp. 17-34.

Trak, B., P. LaRochelle, F. Tavenas, S. Leroueil, and M. Roy, "A New Approach to the Stability Analysis of Embankments on Sensitive Clays", *Canadian Geotechnical Journal*, Vol. 17, No. 4, 1980, pp. 526-544.

Tumay, M.T., R.L. Boggess, and Y. Acar, "Subsurface Investigations with Piezocone Penetrometers", *Cone Penetration Testing and Experience*, (Proc. ASCE National Convention, St. Louis), American Society of Civil Engineers, Reston, Virginia, 1981, pp. 325-342.

Tumay, M.T., "In-Situ Testing at the National Geotechnical Experimentation Sites (Phase II)", *Final Report FHWA Contract No. DTFH61-97-P-00161*, Louisiana Transportation Research Center, Baton Rouge, Feb. 1997, 300 p.

Tumay, M.T., P.U. Kurup, and R.L. Boggess, "A Continuous Intrusion Electronic Miniature CPT", *Geotechnical Site Characterization*, Vol. 2, Balkema, Rotterdam, 1998, pp. 1183-1188.

Vesić, A.S. *Design of Pile Foundations*, Synthesis of Highway Practice, NCHRP Number 42, Transportation Research Board, National Academy Press, Washington, D.C., 1977, 68 pp.

Vlasblom, A., *The Electrical Penetrometer: A Historical Account of Its Development. LGM Mededelingen Report No.92*, Delft Soil Mechanics Laboratory, The Netherlands, Dec. 1985, 51 pp.

Vreugdenhil, R., R. Davis, and J. Berrill, "Interpretation of Cone Penetration Results in Multilayered Soils", *International Journal of Numerical and Analytical Methods in Geomechanics*, Vol. 18, 1994, pp. 585-599.

Wride, C.E. and P.K. Robertson, *CANLEX: The Canadian Liquefaction Experiment: Data Review Report*, (Five Volumes), BiTech Publishers Ltd, Richmond, BC, Canada, 1999, total 1081 pp.

Wride, C.E., P.K. Robertson, K.W. Biggar, and R.G. Campanella, "Interpretation of In-Situ Test Results From the CANLEX Sites", *Canadian Geotechnical Journal*, Vol. 37, No. 3, 2000, pp. 505-529.

Wroth, C.P., "The Interpretation of In-Situ Soil Tests", *Geotechnique* Vol. 34, No. 4, December 1984, pp. 449-489.

Wroth, C.P. "Penetration Testing: A More Rigorous Approach to Interpretation", *Penetration Testing 1988*, Vol. 1 (Proc. ISOPT-1, Orlando), Balkema, Rotterdam, 1988, pp. 303-311.

Wroth, C.P. and G.T. Housby, "Soil Mechanics: Property Characterization & Analysis Procedures", *Proceedings, 11th Intl. Conf. Soil Mechanics & Foundation Engrg*, Vol. 1, San Francisco, 1985, pp. 1-56.

Youd, T. L., I.M. Idriss, R.D. Andrus, I. Arango, G. Castro, J.T. Christian, et al., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 127, No. 10, 2001, pp. 817-833.

Yu, H-S., "James K. Mitchell Lecture: In-Situ Soil Testing: From Mechanics to Interpretation", *Geotechnical and Geophysical Site Characterization*, Vol. 1 (Proc. ISC-2, Porto), Millpress, Rotterdam, 2004, pp. 3-38.

Yu, H-S. and J.K. Mitchell, "Analysis of Cone Resistance: Review of Methods", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol 124, No. 2, 1998, pp. 140-149.

Zhang, Z. and M.T. Tumay, "Statistical to Fuzzy Approach Toward CPT Soil Classification", *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 125, No. 3, 1999, pp. 179-186.

APPENDIX - WEBSITE LINX ON IN-SITU TESTING

A partial listing of manufacturers and suppliers of in-situ testing systems and their websites is given below. This is followed by a list of service companies that offer field testing and related specialty in-situ test methods. Also listed are websites containing information about software specific to the post-processing of in-situ test data.

GENERAL SITES OF INTEREST:

GT In-Situ Research Group: www.ce.gatech.edu/~geosys

Links to In-Situ Info: <http://geosystems.ce.gatech.edu/misclinks.htm>

GeoEngineer Website: www.geoengineer.org

ISSMGE Technical Committee on In-Situ Testing: www.geoforum.com/tc16

CPT Equipment Manufactured Systems and Distributors

AMAP static-dynamic CPT: <http://www.elphicom.com/sicinfra42/>

A.P. van den Berg: www.apvdberg.nl

EAPS for enhanced access: <http://cpt.ara.com/projects/wireline.html>

Envi AB, Sweden (Memocone): www.envi.se

Geomil: www.geomil.com

Geonor (for van den Berg): www.geonor.com

Geotech AB: www.geotech.se

Gouda Manufacturing (GMF): www.goudageo.com

Hogentogler & Company: www.hogentogler.com

Pagani: www.pagani-geotechnical.com

Precision Sampling: <http://www.precisionssampling.com/index.html>

SAGE mini-cones: <http://www.sage-engineering.net/>

Vertek/ARA: www.vertek.ara.com

Flat Plate Dilatometer Test (DMT) for soils

General

<http://www.marchetti-dmt.it>

Suppliers

<http://www.gpe.org>

<http://www.pagani-geotechnical.com/>

http://www.cambridge-insitu.com/DMT/Marchetti_Index.html

Dilatometers for Testing Rocks

<http://www.cambridge-insitu.com/specs/Instruments/73HPDSPC.htm>

Pressuremeter Testing (PMT)

<http://www.cambridge-insitu.com/>

<http://www.pagani-geotechnical.com/>

<http://www.roctest.com/index.php?module=CMS&func=view&id=69>

<http://www.apageo.com/>

Vane Shear Test (VST) or field vane (FV):

General

<http://www.pagani-geotechnical.com/>

http://www.geonor.com/field_vane_testing.html

<http://www.envi.se/>

<http://www.apvdberg.nl/>

<http://www.geomil.com/>

Geophysical testing

General

<http://www.geoforum.com/knowledge/texts/bodare/index.asp?Lang=Eng>

<http://appliedgeophysics.berkeley.edu:7057/>

Suppliers

<http://www.geometrics.com/>

<http://www.geonics.com/products.html>

<http://www.sensoft.ca/>

<http://www.oyo.com/>

<http://www.gdsinstruments.com>

<http://www.greggdrilling.com/methodology.html#sasw>

<http://www.gemsys.ca/>

Field Instrumentation Equipment

<http://www.geokon.com/>
<http://www.geocomp.com/>
<http://www.slopeindicator.com/>
<http://www.solinst.com/>
<http://www.geocon.com>
[Rieker, Inc \(http://www.riekerinc.com/\)](http://www.riekerinc.com/)

Laboratory Testing Equipment Suppliers

<http://www.geocomp.com/>
<http://www.eleusa.com/>
<http://www.terratek.com/>
<http://www.gcts.com/>
<http://www.durhamgeo.com/>
<http://www.gdsinstruments.com/>

Related books on In-Situ Testing available at:

[In-situ Soil Testing and Sampling Bookstore:](#)
(<http://www.guideme.com/Bookstores/INSITU.HTM>)
[Papers on Cone Penetration:](#) (<http://www.geoengineer.org/cpt10.html>)
[Papers on Flat Dilatometer:](#) (<http://www.geoengineer.org/dilatometer10.html>)
[Reference Documents on In-Situ Testing:](#)
<http://www.geoforum.com/tc16/home/page.asp?sid=92&mid=2&PageId=5648>

In-Situ Testing Firms

Applied Research Associates (ARA), USA: wwwара.com

Aquaterra Engineering, Baton Rouge, LA: <http://www.aquaterraeng.com/>

Ardaman Associates, Tampa and Orlando, FL: <http://www.ardaman.com/subsurface.htm>

ConeTec Investigations, USA and Canada: www.conetec.com

ECS Limited, Savannah, GA: www.ecslimited.com

Fugro Geosciences, USA: www.fugro.com

GeoCim, Puerto Rico: <http://geocim.com/>

GeoConsult, Inc, San Juan: <http://www.geoconsult-inc.com/>

Gregg Drilling and Gregg In-Situ, California: www.greggdrilling.com/insitu.html

In-Situ Soil Testing, LC, Washington, DC: <http://www.2006dmt.com/>

Minnesota GeoServices: www.mngeoservices.com

Soil & Material Engineers (S&ME), Charleston, SC: www.smeinc.com

Southern Earth Sciences (SES), Mobile AL and Baton Rouge, LA: www.soearth.com

Stratigraphics, Chicago: <http://stratigraphy.com/>

Whitaker Laboratories, Savannah, GA: http://www.whitakerlab.net/services_cpt.htm

Williams Earth Sciences, Tampa, FL: <http://www.williamsearthsciences.com/>

Wright-Padgett-Christopher (WPC), Charleston and Savannah: www.wpceng.com

In-Situ Websites of Interest and Information:

Cone Penetration:

www.conepenetration.com

GT Geolinks:

<http://www.ce.gatech.edu/~geosys/misc/links.htm>

ISSMGE Technical Committee 16 on In-Situ Testing:

www.geoforum.com/tc16

Minnesota GeoServices CPT Links:

http://www.mngeo.com/CPT_related_links

MN DOT soil borings and CPT soundings:

<http://www.mrr.dot.state.mn.us/geotechnical/foundations/borings/borings.asp>

US DOE Report on Cone Penetrometer Innovations:

<http://www.external.ameslab.gov/cmst/CMSTSsite/Projects/ITSR/coneopen/index.html>

Ames Laboratory - Site Characterization by CPT:

<http://www.external.ameslab.gov/cmst/CMSTSsite/characterization.html>

IN-SITU TESTING SOFTWARE WEBSITES

The below links are for software programs available for post-processing cone penetration test data:

Conrad is a program offered by the Swedish Geotechnical Institute for reducing CPT data:

<http://www.swedgeo.se/varatjanster/program-e.html>

CPTINT is software developed by Prof. Dick Campanella at UBC and provides interpretations of 35 different soil parameters from CPT results:

<http://www.civil.ubc.ca/home/rgc/>

CPTLIQ offers evaluation of soil liquefaction potential and estimate on magnitude of ground displacements in seismic concerns:

<http://www.geosoftwaresolutions.com/>

CPT-LOG offers graphics software for plotting channels from CPT readings:

www.geotech.se

CPT-Pro is a program for complex interpretation of CPT soundings and preparation of geotechnical cross-sections:

www.geosoft.com.pl

DCCONE is a program for presentation of CPT results in profiling:

<http://www.dc-software.com/html/dccone.html>

DMT ELAB is a Windows-based program to reduce flat dilatometer test data:

www.marchetti-dmt.it

Edison is a CPT program from the Swedish Geotechnical Institute to obtain soil parameters:

<http://www.swedgeo.se/varatjanster/program-e.html>

Geotechnical & Geoenvironmental Service Directory (GGSD) is a website link to over 1500 geotechnical software programs ranging from free-ware, public-domain, share-ware, to commercially-sold programs. Subtopics with geotechnics can be sorted to the user's interests:

www.ggsd.com

Geotechnical Directory lists some 12 computer software programs for the CPT:

http://www.geotechnicaldirectory.com/page/Software/Insitu_testing.html

Geotechnical Integrated (gINT) software is used to create boring logs, sounding records, and subsurface cross-section profiles, as well as data managing:

www.gintsoftware.com

LPD-CPT is a computer program for evaluating axial pile capacity for driven concrete pilings by three different CPT methods. It is a free download from the LTRC website at:

<http://www.ltrc.lsu.edu/downloads.html>

MFoundation is a commercial code developed at GeoDelft for using CPT data to obtain capacity evaluations of pile bearing, tension piles, and shallow foundations:

<http://www.delftgeosystems.nl/>

PClass-CPT is freeware used to determine fuzzy soil classification profiles by CPT. It has been developed by the Louisiana Transportation Research Center (LTRC) under the efforts of Dr. Mehmet T. Tumay:

<http://www.coe.lsu.edu/cpt/>

PL-aid is a program (pile load-settlement analysis from in-situ data) that uses electric CPT data to evaluate axial capacity and pile displacements:

<http://mctrans.ce.ufl.edu/>

Rapid CPT is a subroutine add-on to gINT for reduction of cone penetration test data. It produces various plots of CPT readings, interpreted soil types, and soil parameters and allows the user to graph these in gINT to create subsurface profiles across a project site. It further allows multiple comparison plots of resistances or parameters from a series of CPT soundings to produce summary graphs of results:

www.dataforensics.net

Shake2000 is a commercial program for site-specific estimates of ground motions and soil liquefaction evaluation from CPT data:

<http://www.shake2000.com/>

ShearPro is a freeware program that reduces downhole shear wave velocity data from SCPT using the cross-correlation method:

<http://www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html>

Static Probing is software for plotting CPT results and interpreted soil values for 19 different parameters, output in tabular or graphical format:

<http://www.geostru.com/>

UniCone offers a means to plot soil profiles from CPT data and interpret axial pile capacity:

www.unisoftltd.com

SI CONVERSION FACTORS

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm ²	square millimeters	0.0016	square inches	in ²
m ²	square meters	10.764	square feet	ft ²
ha	hectares	2.47	acres	ac
km ²	square kilometers	0.386	square miles	mi ²
VOLUME				
ml	millimeters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m ³	cubic meters	35.71	cubic feet	ft ³
m ³	cubic meters	1.307	cubic yards	yd ³
MASS				
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
TEMPERATURE				
°C	Celsius	1.8 C + 32	Fahrenheit	°F
WEIGHT DENSITY				
g/cc	grams per cubic centimeter	62.4	poundforce /cubic foot	pcf
kN/m ³	kilonewton /cubic meter	6.36	poundforce /cubic foot	pcf
FORCE and LOAD				
N	newtons	0.225	poundforce	lbf
kN	kilonewtons	225	poundforce	lbf
kg	kilogram (force)	2.205	poundforce	lbf
MN	meganewtons	112.4	tons (force)	t
PRESSURE and STRESS*				
kPa*	kilopascals	0.145	poundforce /square inch	psi
kPa	kilopascals	20.9	poundforce /square foot	psf
MPa	megapascal	10.44	tons per square foot	tsf
kg/cm ²	kilograms per square cm	1.024	tons per square foot	tsf

*Notes: 1 kPa = kN/m² = one kilopascal = one kilonewton per square meter.

For dimensionless graphs and equations, a reference stress of one atmosphere can be used, such that $\sigma_a = p_{atm} = 1$ bar = 100 kPa \approx 1 tsf \approx 1 kg/cm².