# GRIP MEETING 2020, Draft Final Report

## Project Title: Comparison of Standard Penetration Test (SPT) N-value with Alternative Field Test Methods in Determining Moduli for Settlement Predictions

PRESENTED BY

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# Outline

**Selected Topics:**

- Technical review
- Conical load field testing program
- Laboratory testing program
- Numerical modeling and correlations
- Final recommendations

# Project goals: reminder

Improve immediate settlement calcs. for Florida soils

- Field load testing under stress-controlled conditions to assess:
  - ➢ Conical weight/pressure vs time
  - ➢ Settlement vs time

Identify correlations based on SPTs and CPTs for soil modulus

- Review of correlations for $E$, (and … $\phi$, $D_r$, $G_0$)

Identify suitable immediate settlement methods and validate with numerical models

- Review methods

**Final recommendations for: $E$ and $S_i$**

- Analysis based on lab/field load tests and numerical simulations

# Technical review

Summary **correlations for *E*** with SPT, CPT, DMT, PMT ≈ 73 correlations

Examples:

$E/Pa=5N_{60}$ (sand with fines)
$E/Pa=10N_{60}$ (clean NC sand)
$E/Pa=15N_{60}$ (clean OC sand)

$E_s = 7.5 + 0.8N$, MPa

$E_s = 2.5q_c$  L/B=1 to 2
$E_s = 3.5q_c$  L/B≥10

$E = 3.33\ (N + 5)$, tons/ft$^2$
(Clayey saturated sands)

*Bachelier and Parez (1965)*
*Begemann (1974)*
*Bogdanovic' (1973)*
*Bowles (1996)*
*Buisman (1940)*
*Chaplin (1963)*
*Clayton et al. (1985)*
*CPT Guide-2015*
*De Beer (1967)*
*DeBeer (1974b)*
*Denver (1982)*
*Farrent (1963)*
*FHWA-IF-02-034*
*Gielly et al. (1969)*
*Kulhawy and Mayne (1990)*
*Meigh and Corbett (1969)*
*Papadopoulos (1982)*
*Sanglerat et al. (1972)*
*Schmertmann (1970)*
*Schmertmann et al. (1978)*
*Schultze and Melzer (1965)*
*Thomas (1968)*
*Totani et al. (2001)*
*Trofunenkov (1964)*
*Trofunenkov (1974)*
*Vesic (1970)*
*Webb (1969)*

Summary **methods for $S_i$** calculation. ≈ 32 methods:
- Theory of elasticity
- Semi-empirical with SPT, CPT, DMT, PMT

Examples:

$$s = \frac{q \cdot d \cdot I_G \cdot I_F \cdot I_E \cdot (1 - \nu^2)}{E_0}$$

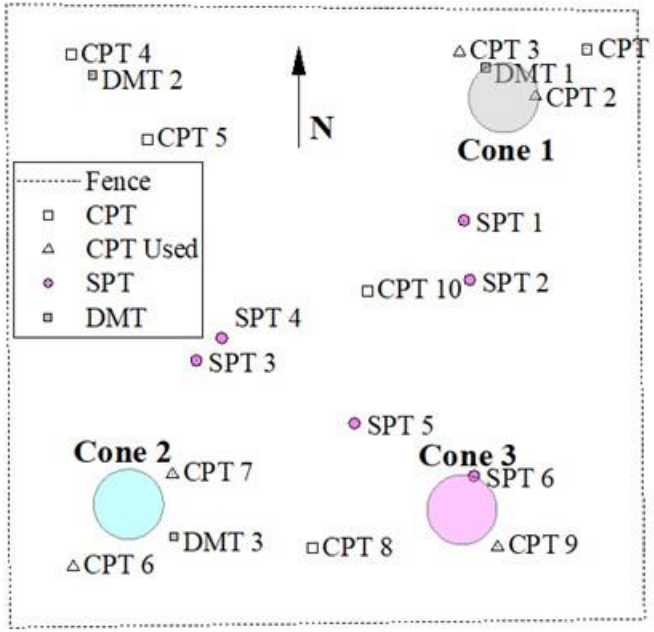
$$s = C_1 C_2 q \sum_{i=1}^{n} (I_z / E_s) z_i$$

*Alpan (1964)*
*Anagnostopoulos et al. (1991)*
*Arnold (1980)*
*Berardi et al. (1991)*
*Bowles (1987)*
*Briaud (1992)*
*Burland and Burbidge (1985)*
*Can. Found. Manual (1975)*
*D'Appolonia (1968)*
*DeBeer (1965)*
*DeBeer and Martens (1957)*
*Elastic half-space method*
*Hough method (1959)*
*Leonards and Frost (1988)*
*Mayne and Poulos (1999)*
*Menard and Rousseau (1962)*
*Meyerhof (1965)*
*Meyerhof (1974)*
*Oweiss (1979)*
*Papadopoulos (1992)*
*Parry (1971)*
*Peck and Bazaraa (1969)*
*Peck et al. (1974)*
*Robertson (1991)*
*Schmertmann (1970)*
*Schmertmann (1986)*
*Schultze and Sherif (1973)*
*Teng (1962)*
*Terzaghi and Peck (1967)*
*Tschebotarioff (1953, 1971)*
*Webb (1969)*

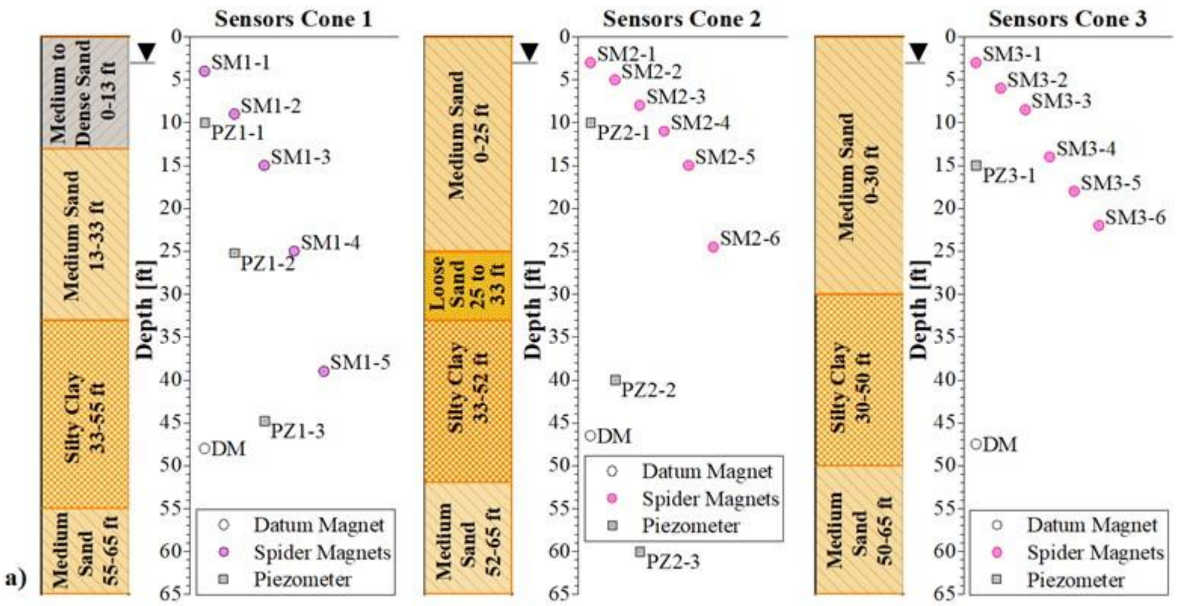
**Note:** only authors are shown. See report for correlations and methods.

# Plan view of field tests

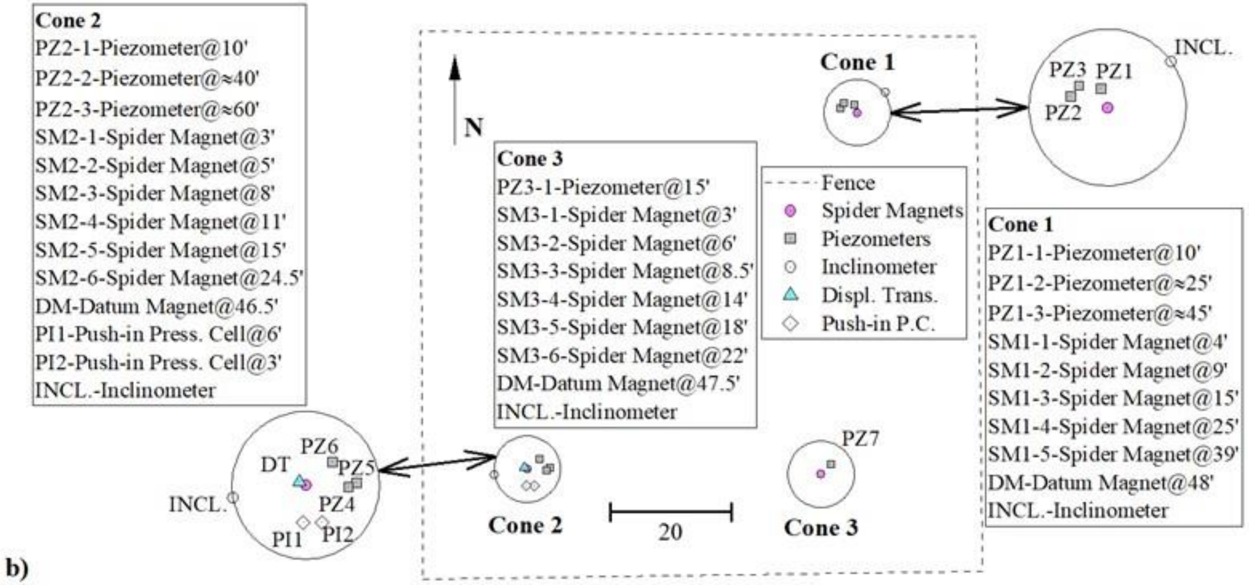
## Conical load and field tests



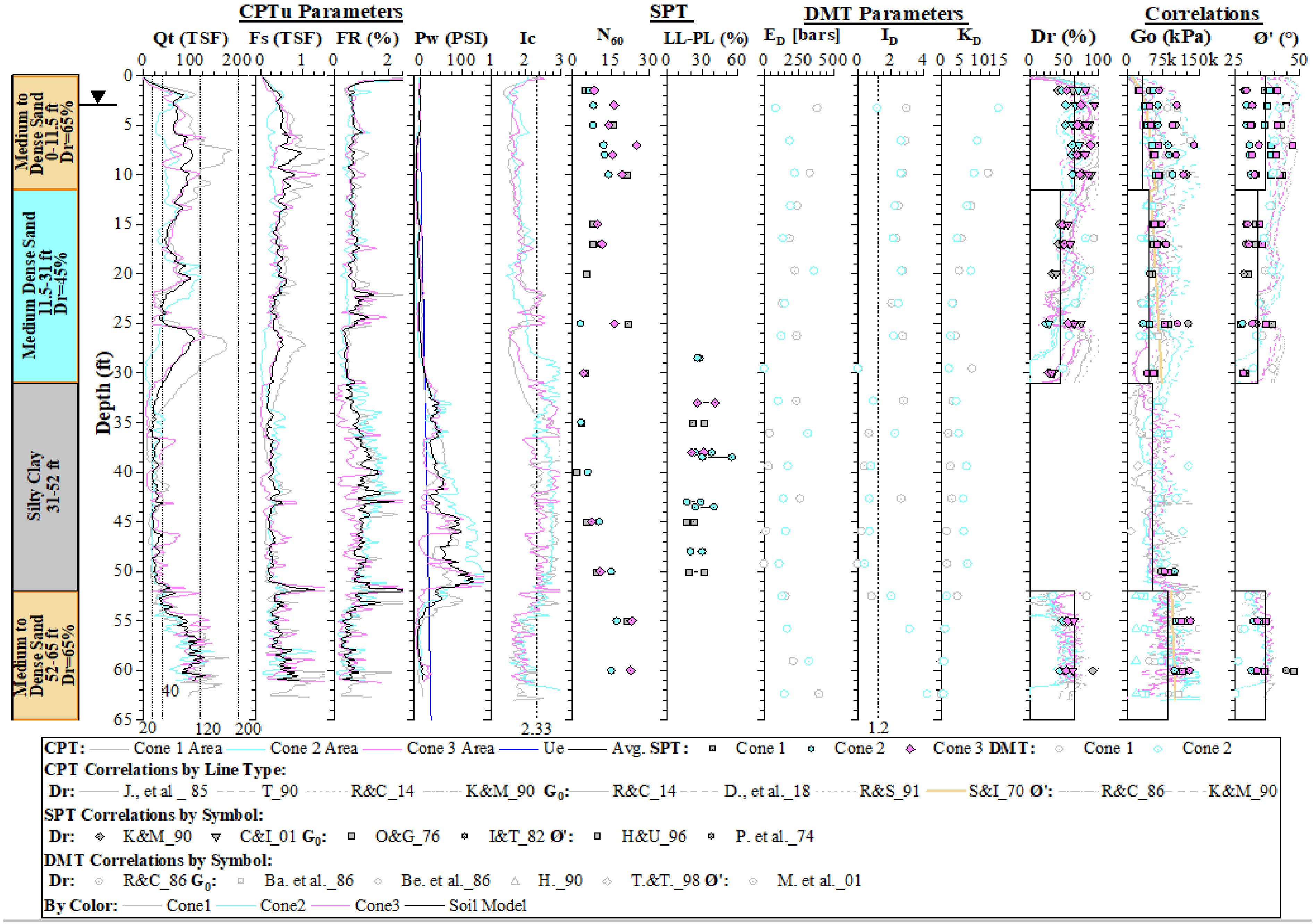
## Field instrumentation location: elevation view



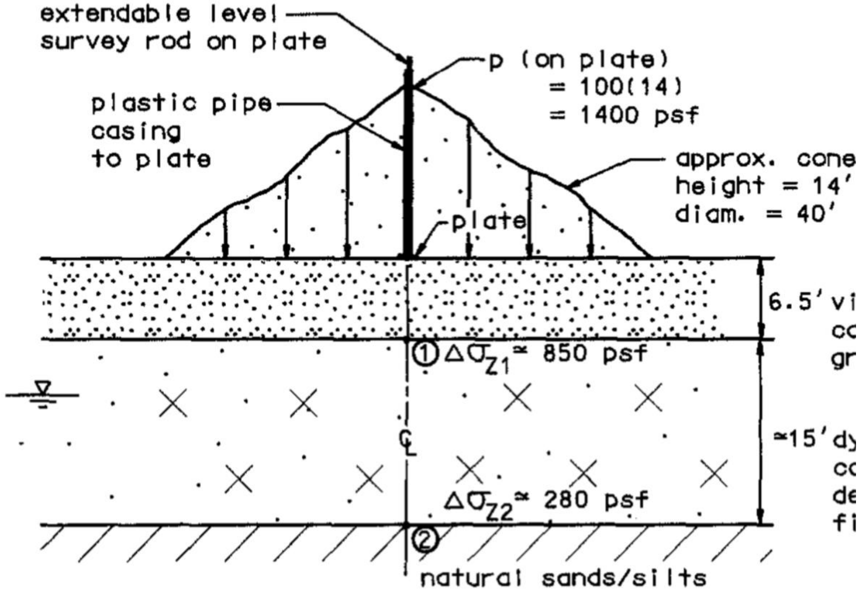
## Field instrumentation location: plan view

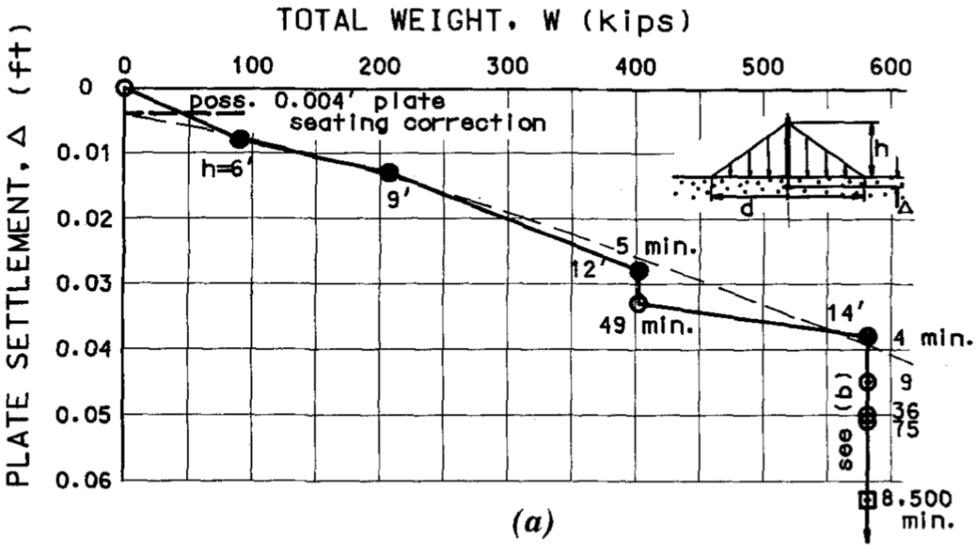


# Summarized soil profile at project site



# Conical load tests (Schmertmann 1993)

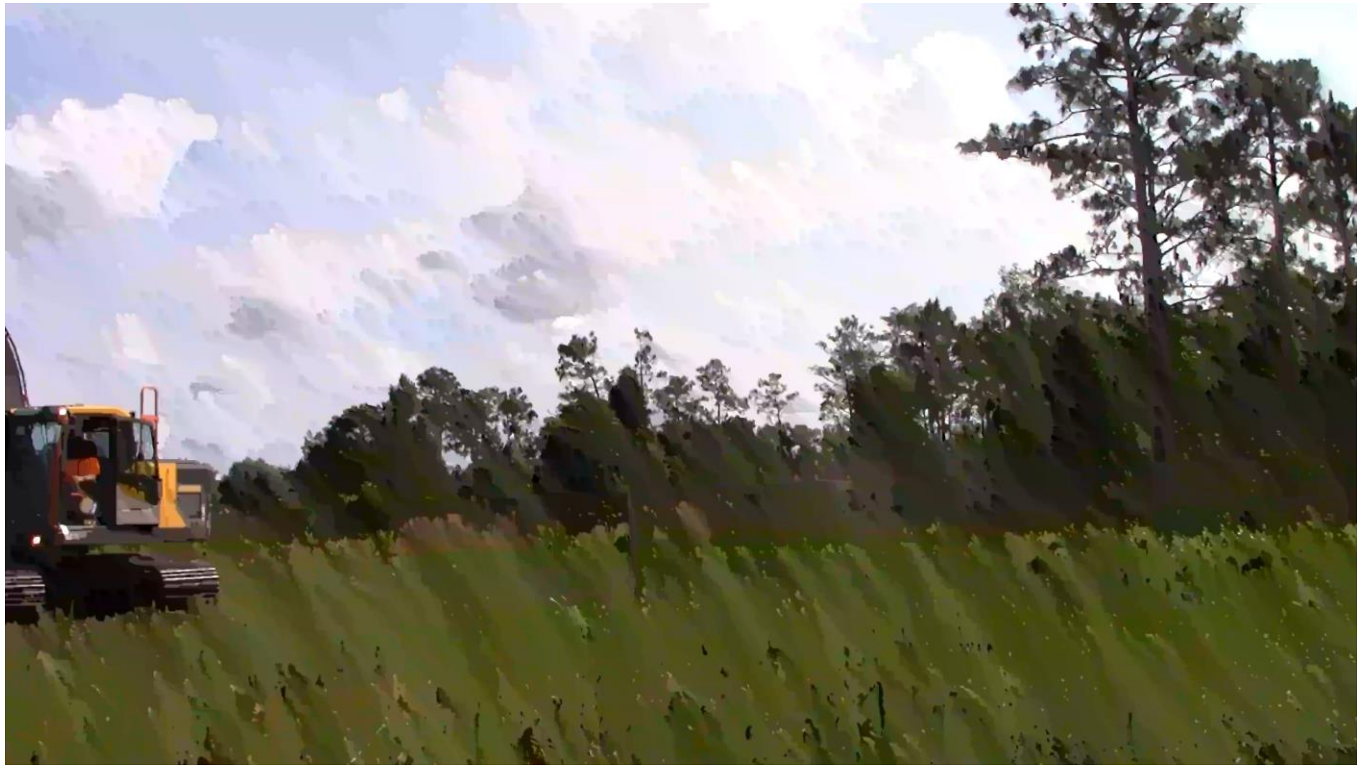




**Measurements:**

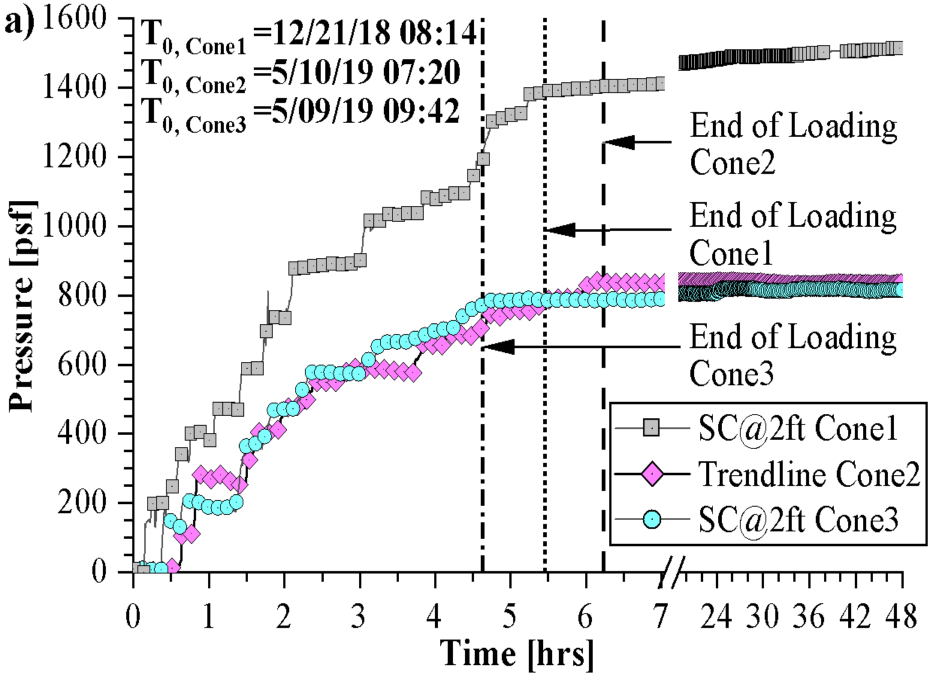
- Settlement at cone centerline (3 ways)
- Pore water pressures
(hydrostatic + excess)
- Stresses at ground surf.
- Density of soil loading
- Horiz. stresses with push-in cells
- Horiz. deformations with inclinometers

**Video:**

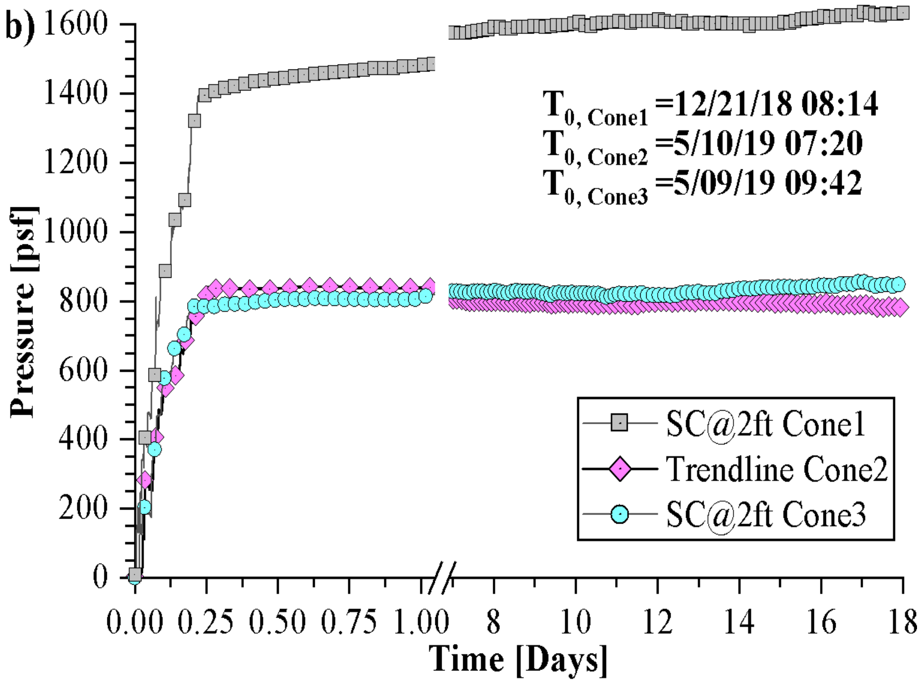


# Results field tests: stress cells vs time

## During the tests

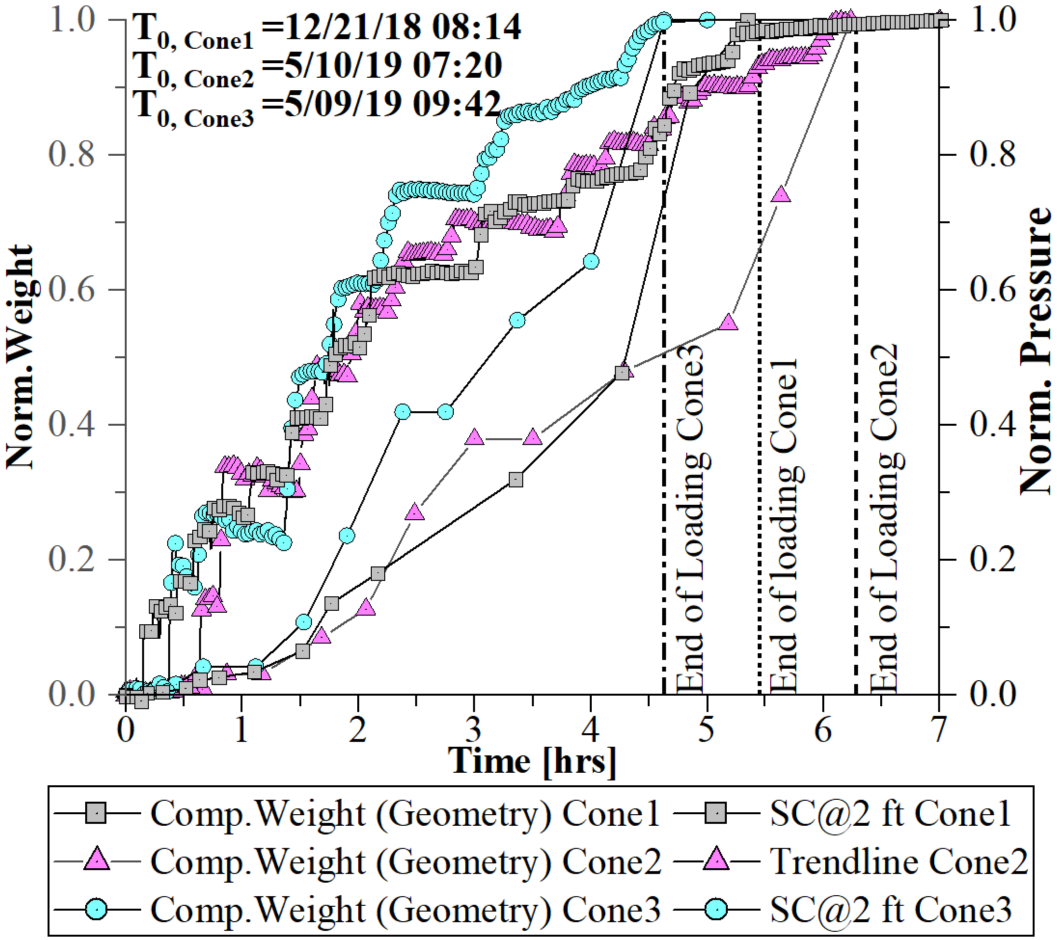


## Long-term data



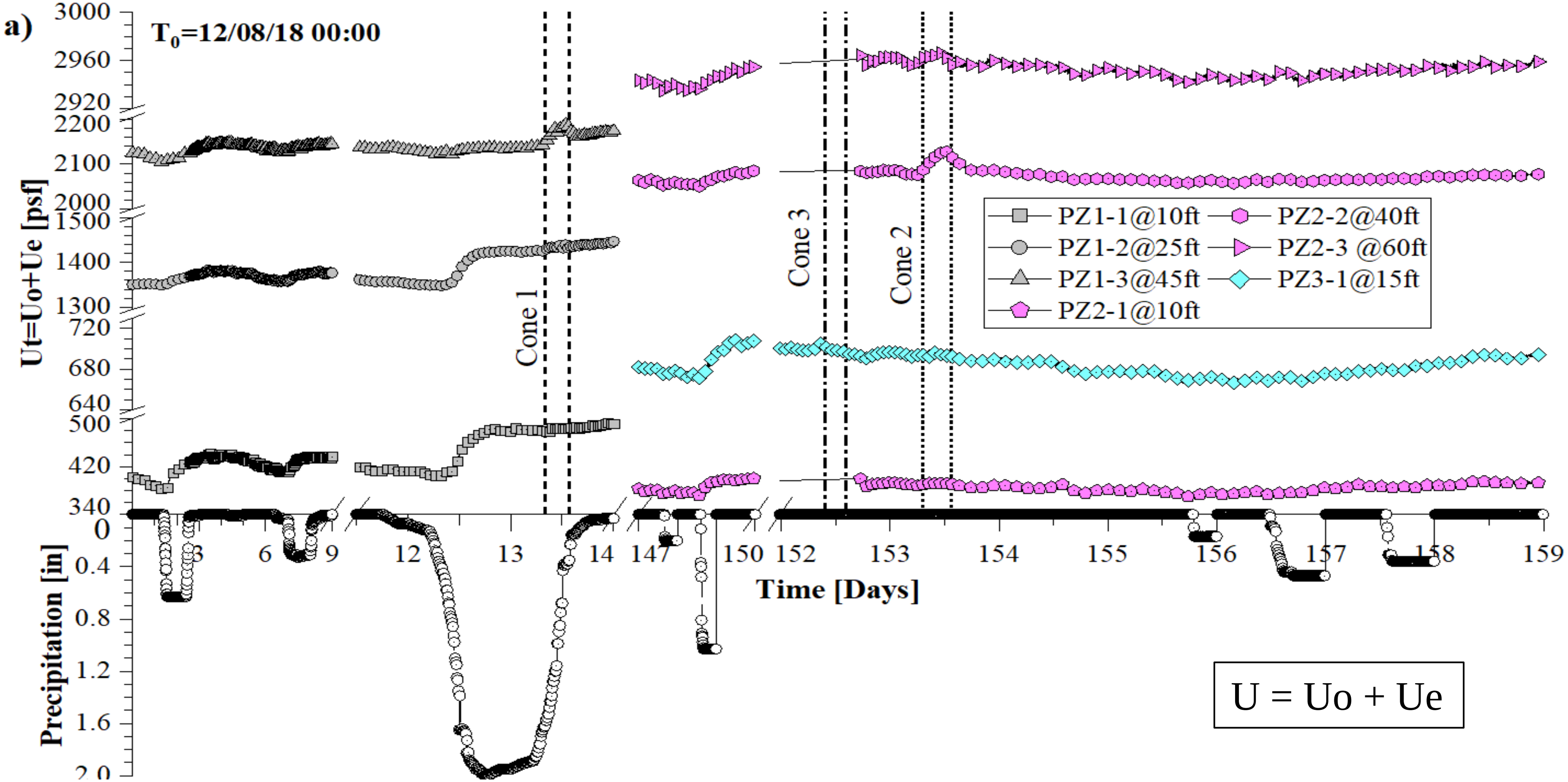
- Pressures near the cone centerline are shown
- Soil unit weight at cone 1 (*100pcf*) and cones 2-3 (*90pcf*). (ASTM sand cone test)
- Higher water content of the loading material at cone 1 than cones 2-3 (10% vs 6%)
- Final cone volumes (7335, 5734, and 5990 ft$^3$ at cones 1 to 3, respectively)
- Slightly larger long-term variation at cone 1 because of rainy season
- Negligible long-term variation of settlement, which means they are immediate.

# Results field tests: normalized weights and pressures vs time



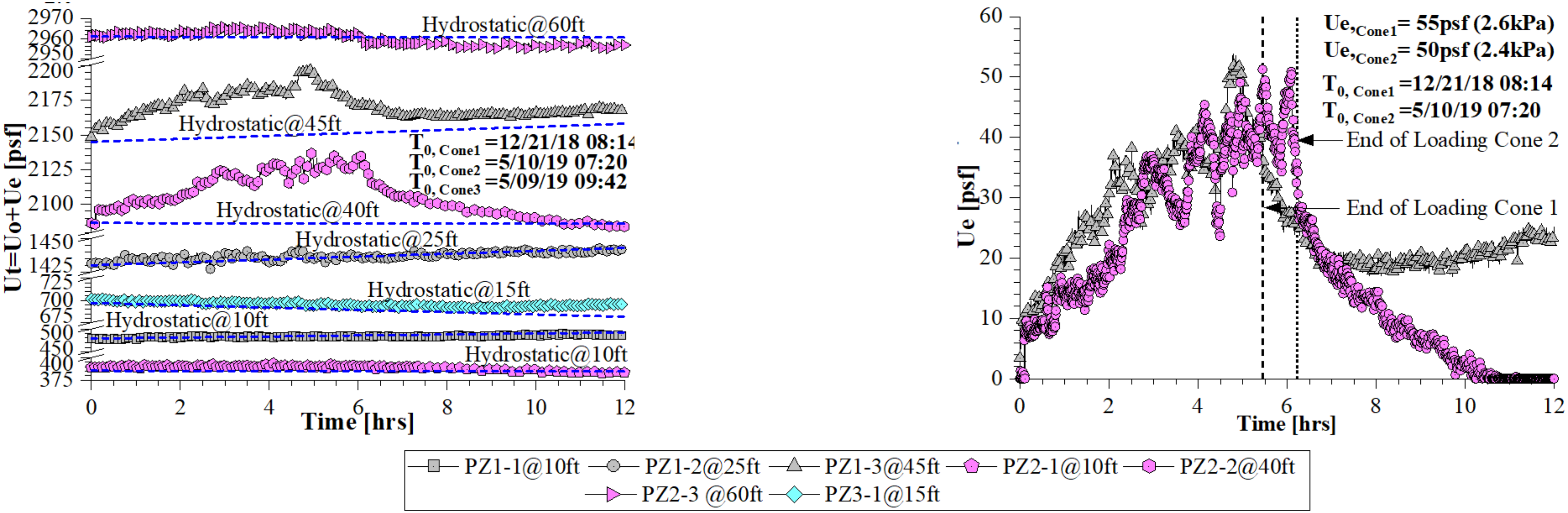
- Differences between normalized weights and pressures show the stiffness effect of the conical load material (deformable soil body).
- Differences in loading rates for all tests were negligible.
- Stress redistributions and soil “arching” in conical soil arrangement were identified. Stiffness of the applied load is important.

# Results field tests: porewater pressures (long-term)



- Measurements completed using 7 piezometers. Recall: *U = Uo + Ue*.
- Water table fluctuations correlated well with precipitation data to find *Uo*. Conical load-induced excess porewater pressure is *Ue*.
- Results were used to check: 1) variation of water table vs. time, 2) soil type at each location, 3) type of settlement: $S_i$, $S_c$ or $S_s$, and 4) possible downward flow conditions.

# Results field tests: porewater pressures vs time (short-term)

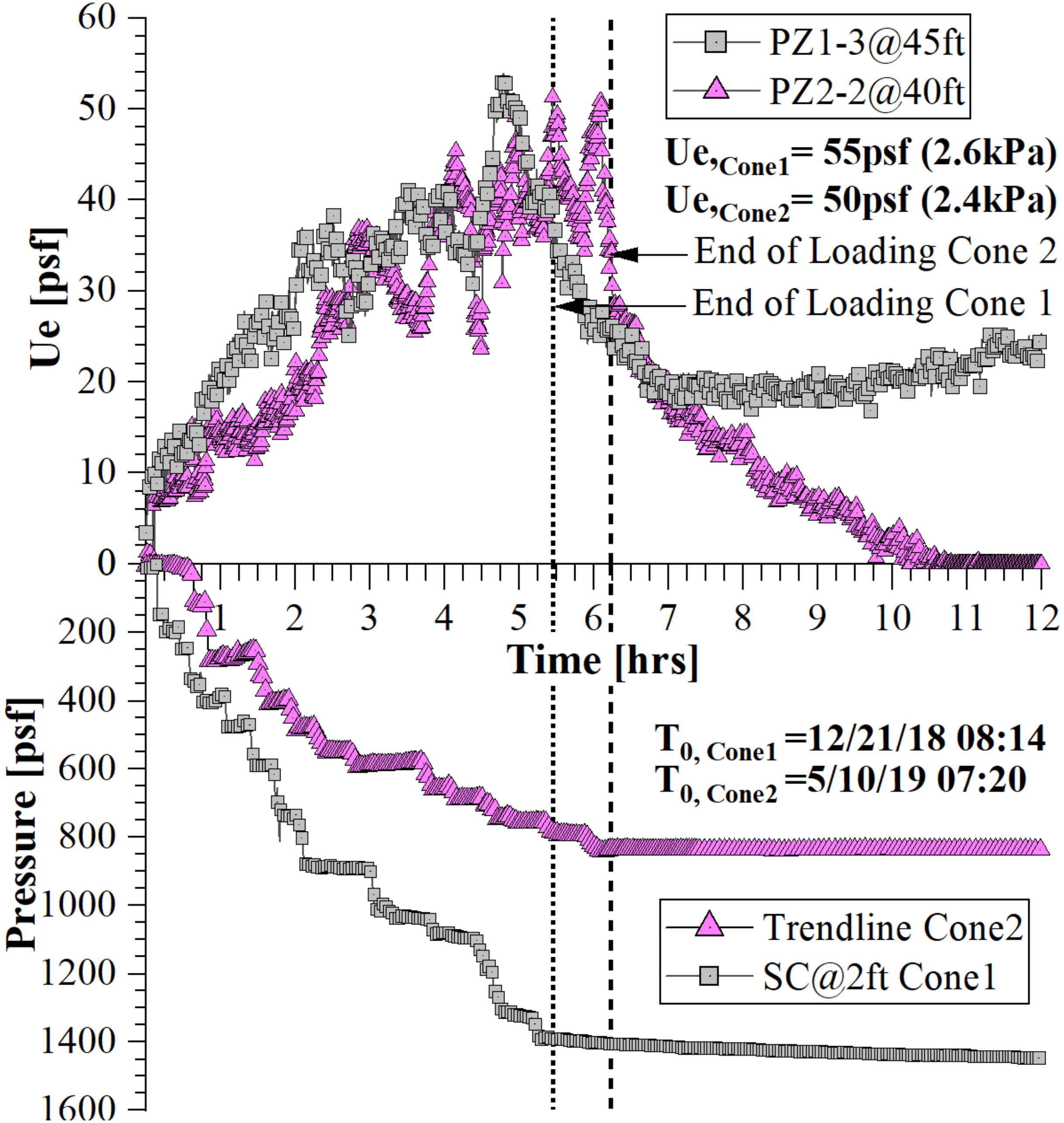


Total porewater pressure (during test)

Excess porewater pressure (during test)

- Negligible *Ue* measured. Even at deep fine soil layer… then only $S_i$ was measured.
- *Ue* dissipated fast after test was completed.
- Confirmation of 40 ft deep vertical influence zone, initially estimated using Boussinesq analyses.
- Observe small *Ue* in right-hand side figure installed in the deep silty clay layer.

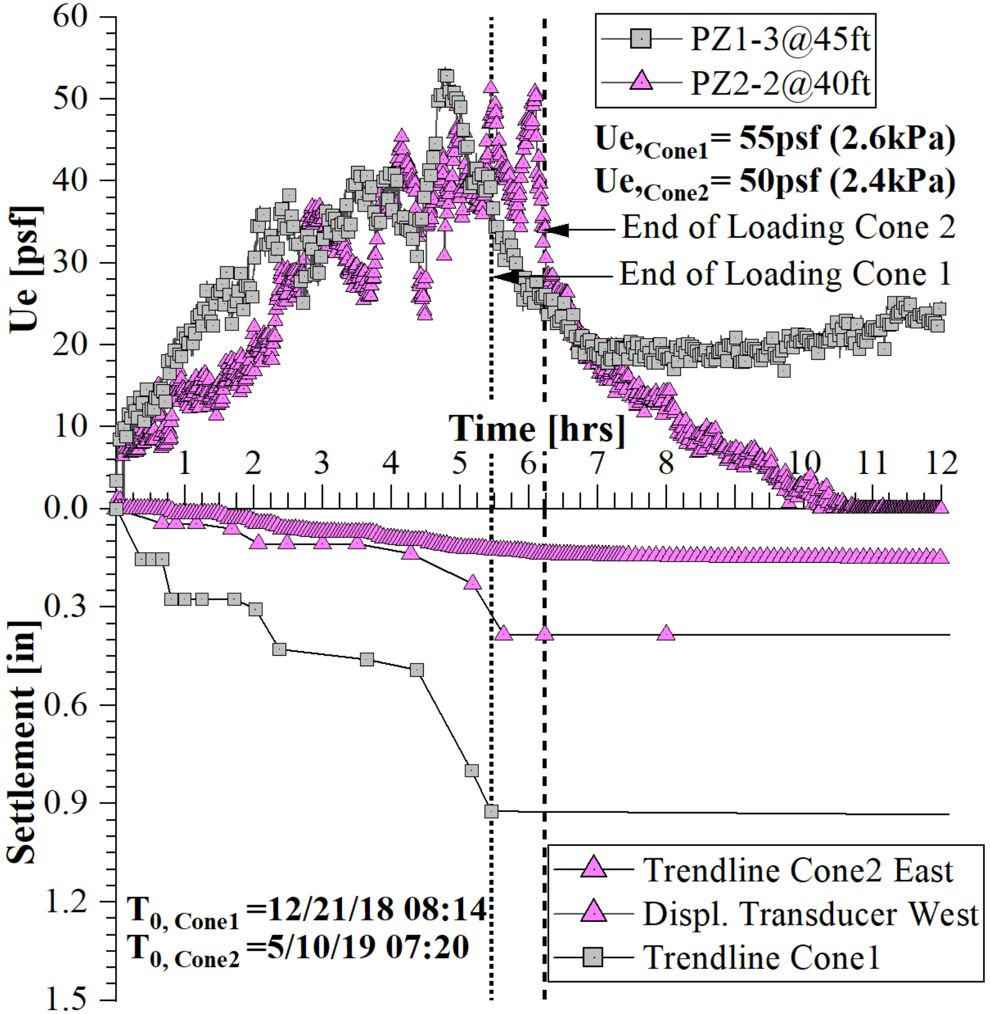
# Results field tests: *Ue* and earth pressures at ground surface



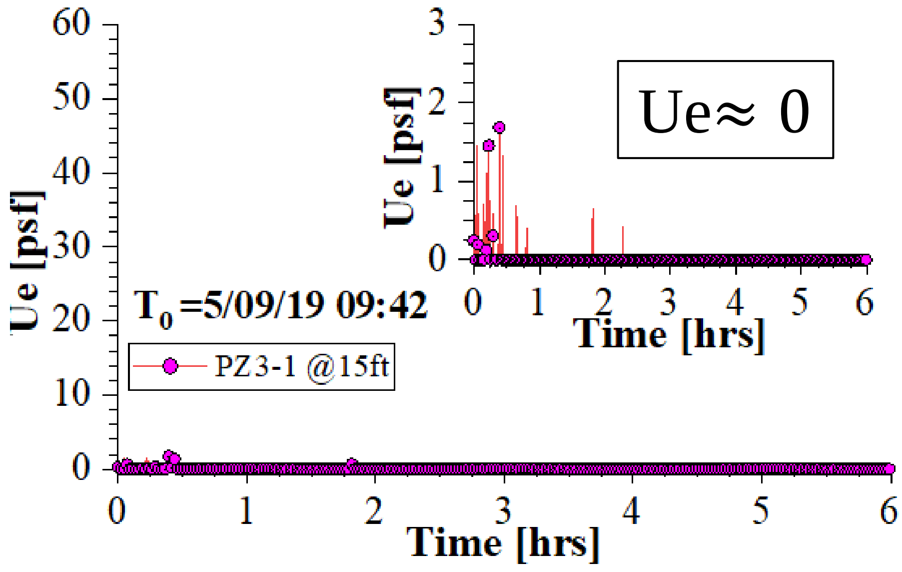
- *Ue* are less than 10% of the vertical effective stress at that depth, which implies negligible *Ue* at 40 ft.
- As conical loading increases, *Ue* builds up.
- As conical loading was completed, and *Ue* dissipated… what happened to $S_c$?

# Results field tests: *Ue* and settlement at ground Surface

**Typical piezometers at the silty clay layer (@40ft)**



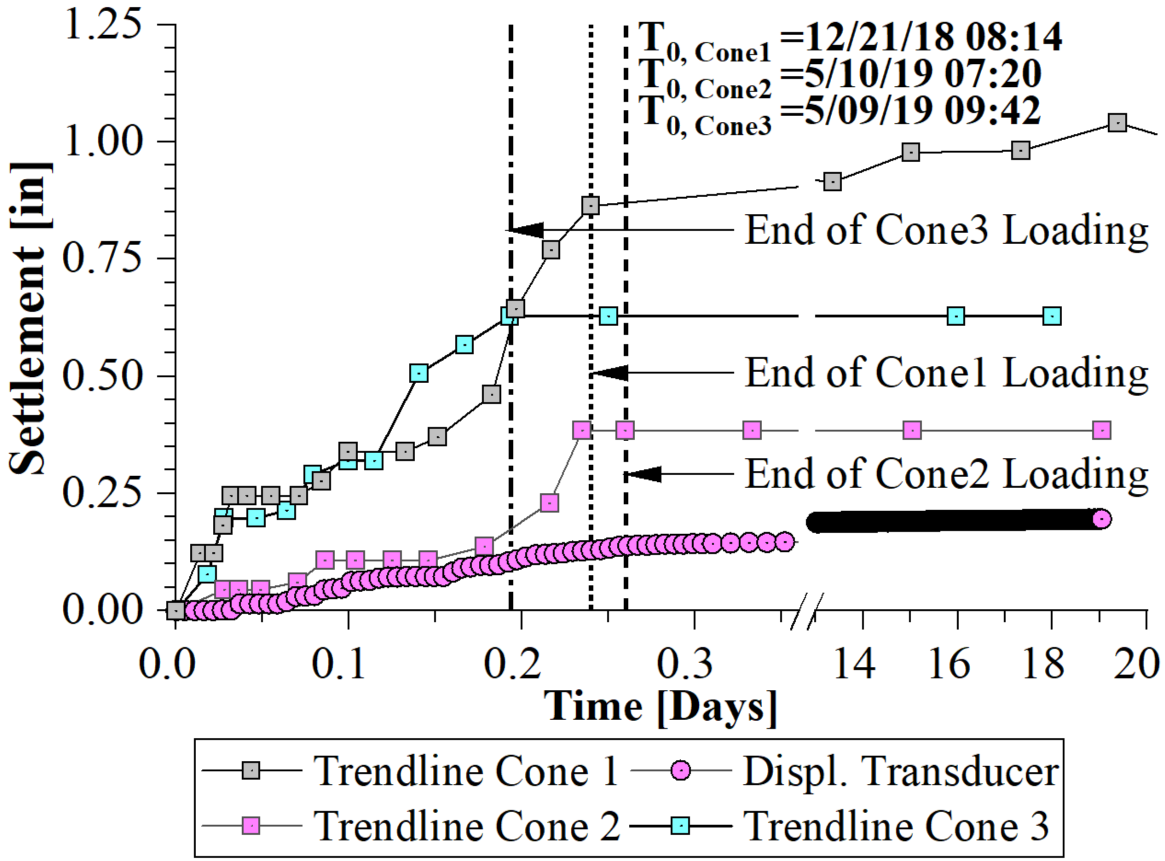
**Typical piezometers at the sandy layers (above 40ft)**



Remember: $S_{total} = S_i + S_c + S_s$

- $S_c$? Nope!
- $S_s$? Nope!
- $S_i$? Yup!

# Results field tests: settlement (long-term)



Cone 1: $S_i \approx 0.75"$ to 1"
Cones 2-3: $S_i \approx 0.4"$ to 0.6"

Differences arising from:

- Unit weight and volume of loading material

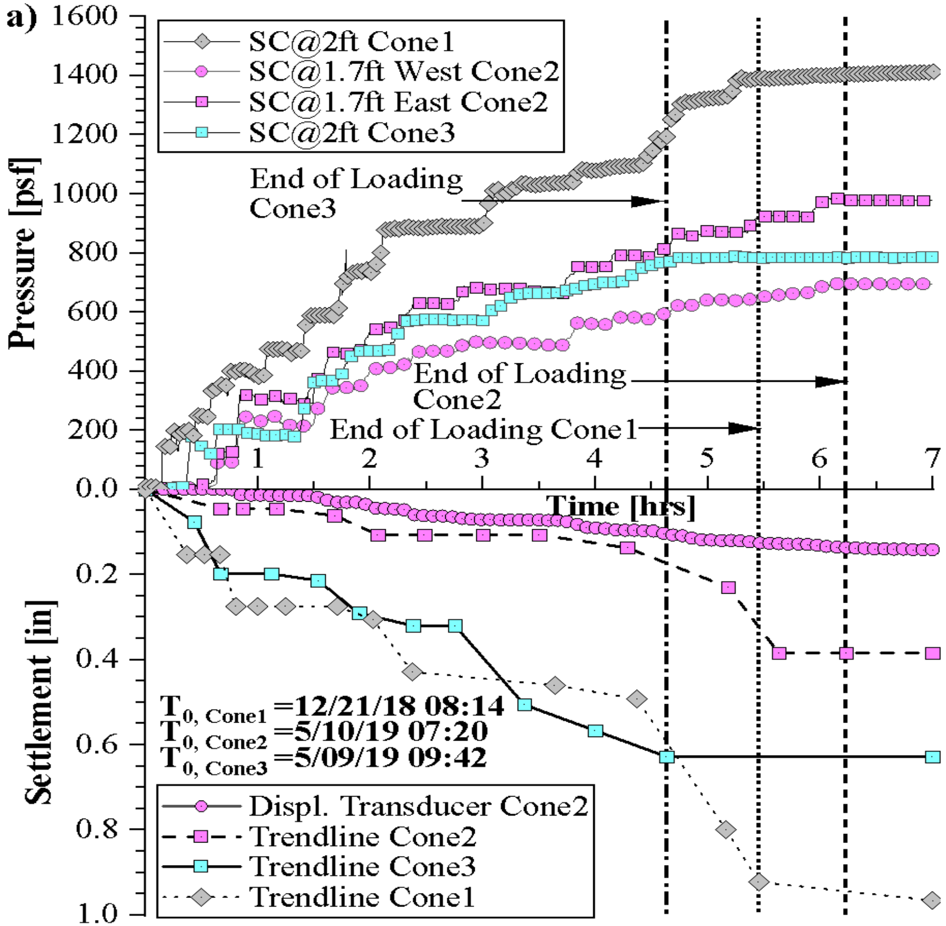
- Slightly different soil conditions for each cone

- Negligible long-term settlement data, negligible $S_c$ after *Ue* dissipated, zero $S_s$ was measured. Thus, only $S_i$ was measured!

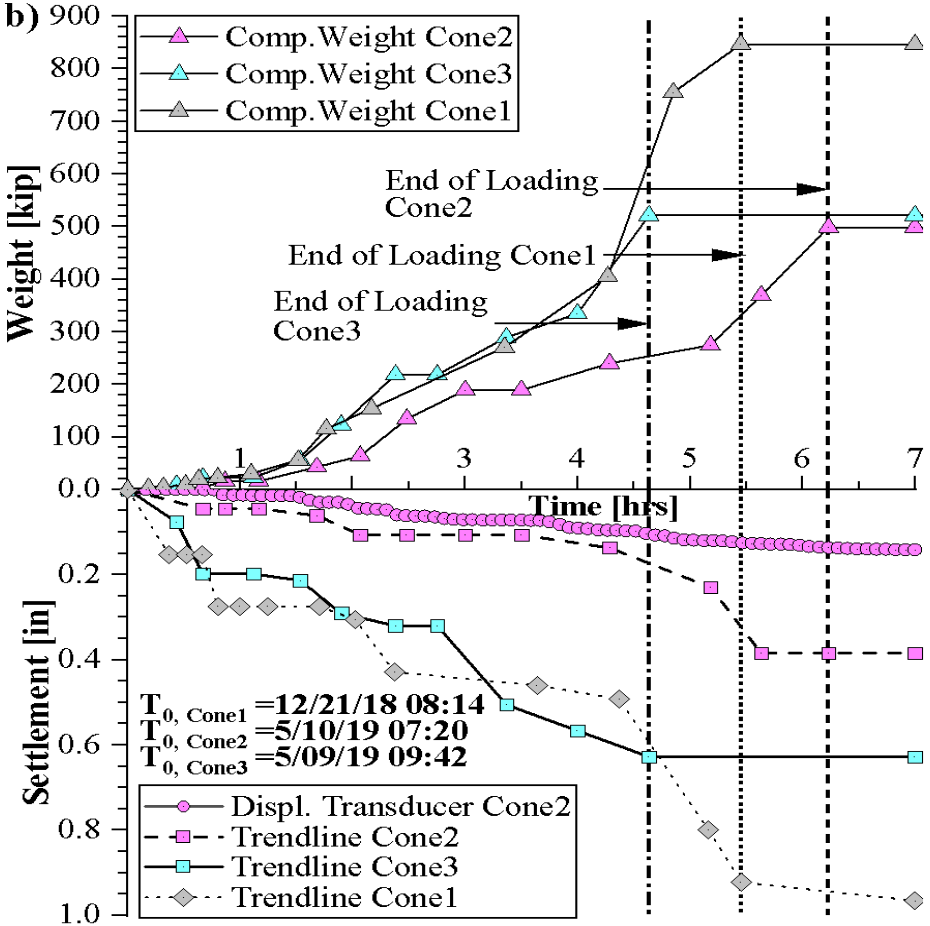
- For an influence zone of 40 ft, computed axial strains ($\varepsilon_a$) mobilized by conical loading were about 0.1 to 0.2%. (Important because *E* is a function of mobilized strains by the applied loading!)

# Results field tests: settlement, pressure, and weights vs time

Pressure and settlement vs time



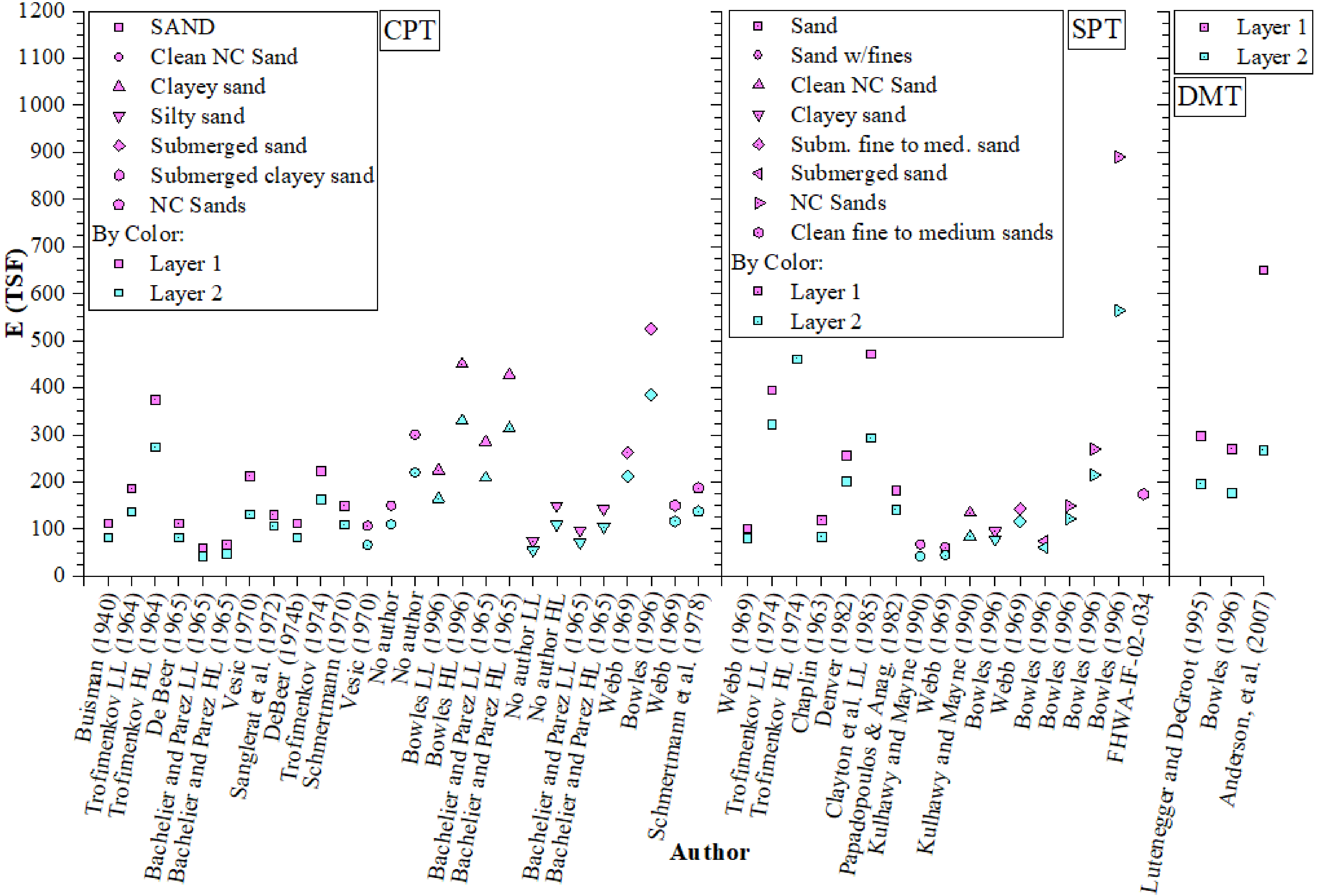
Weight and settlement vs time



- Settlement variations were best described by pressures rather than weights.
- As expected, larger pressures due to conical loads caused larger settlements
- Stiffness of conical load can be evaluated with stress cells, not as good using weights as Schmertmann did.

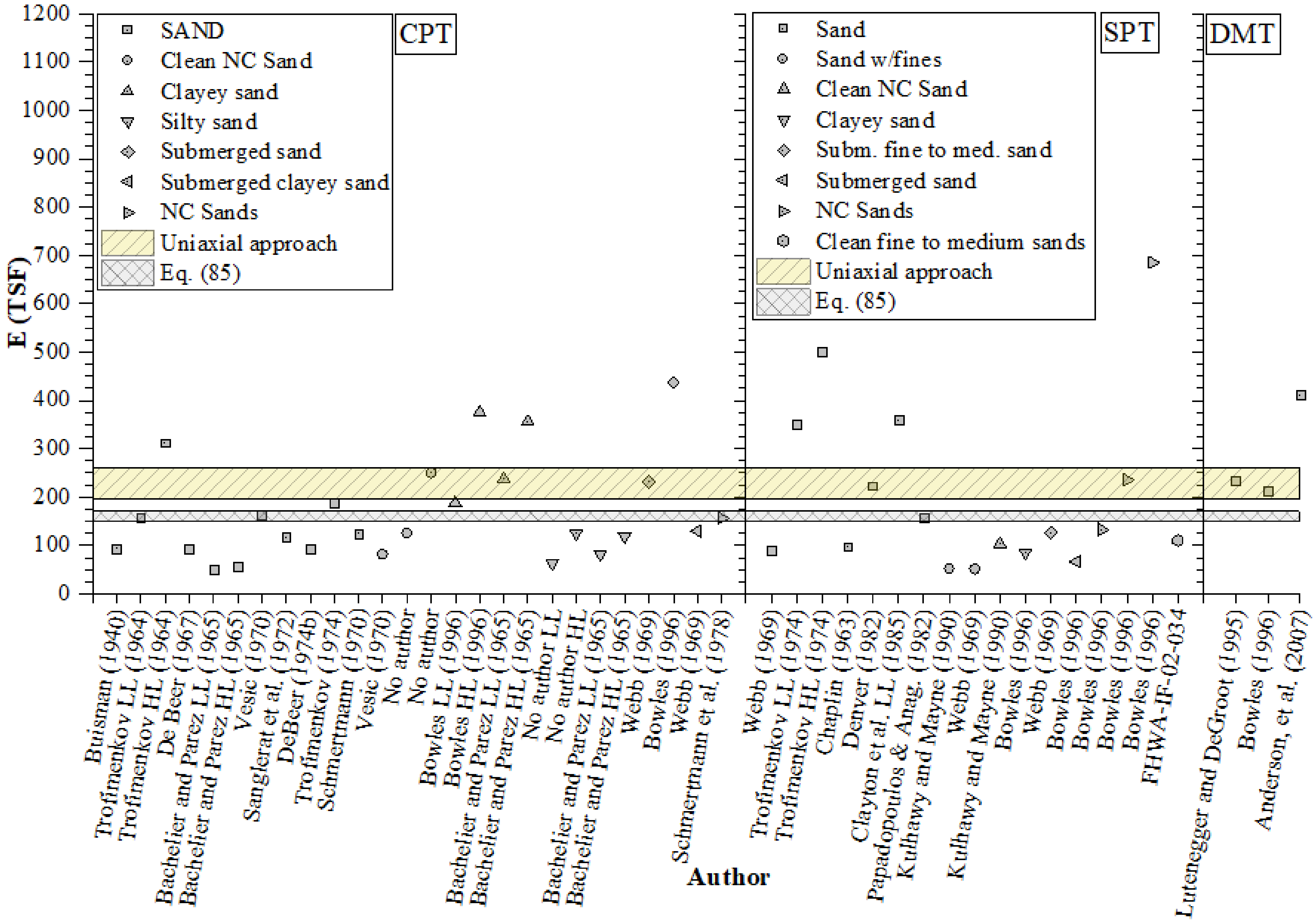
# Conclusions about conical load test results

## $E_{computed}$ using published correlations with field tests (SPT, CPT, DMT)



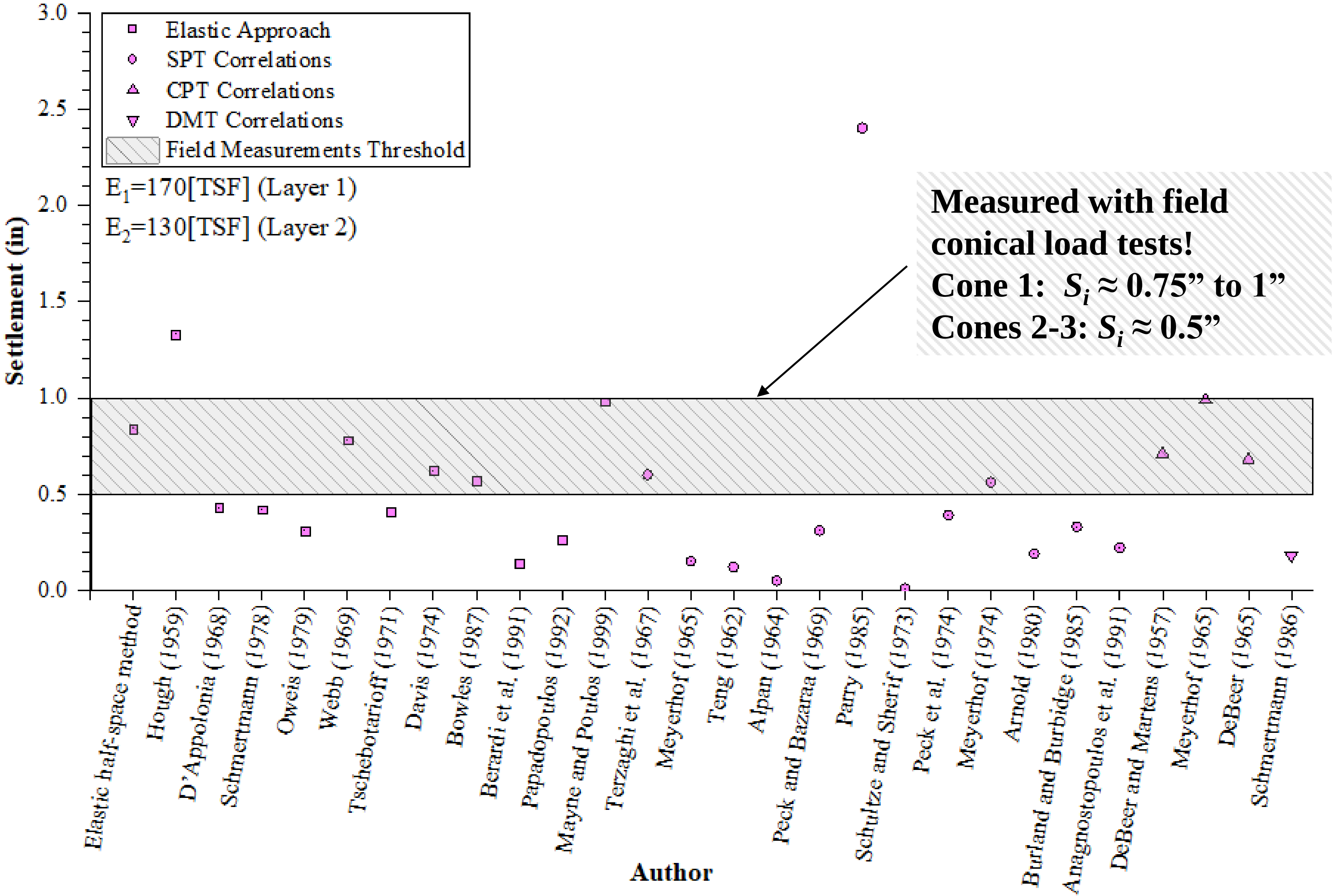
# Conclusions about conical load test results

## $E_{computed}$ for topmost sandy layers versus $E_{measured}$ with conical load tests



# Conclusions about conical load test results

**Computed $S_i$ using published methods versus measured values from conical load tests**

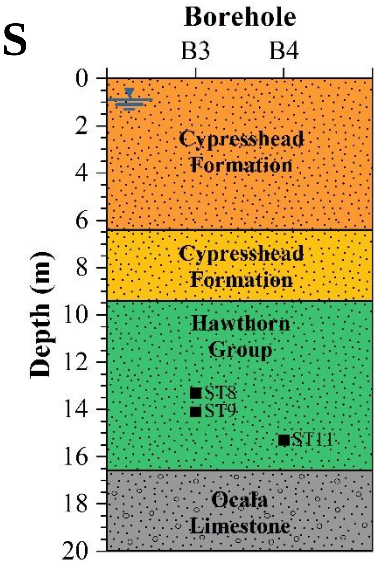


**Measured with field conical load tests!**
**Cone 1: $S_i \approx 0.75"$ to 1"**
**Cones 2-3: $S_i \approx 0.5"$**

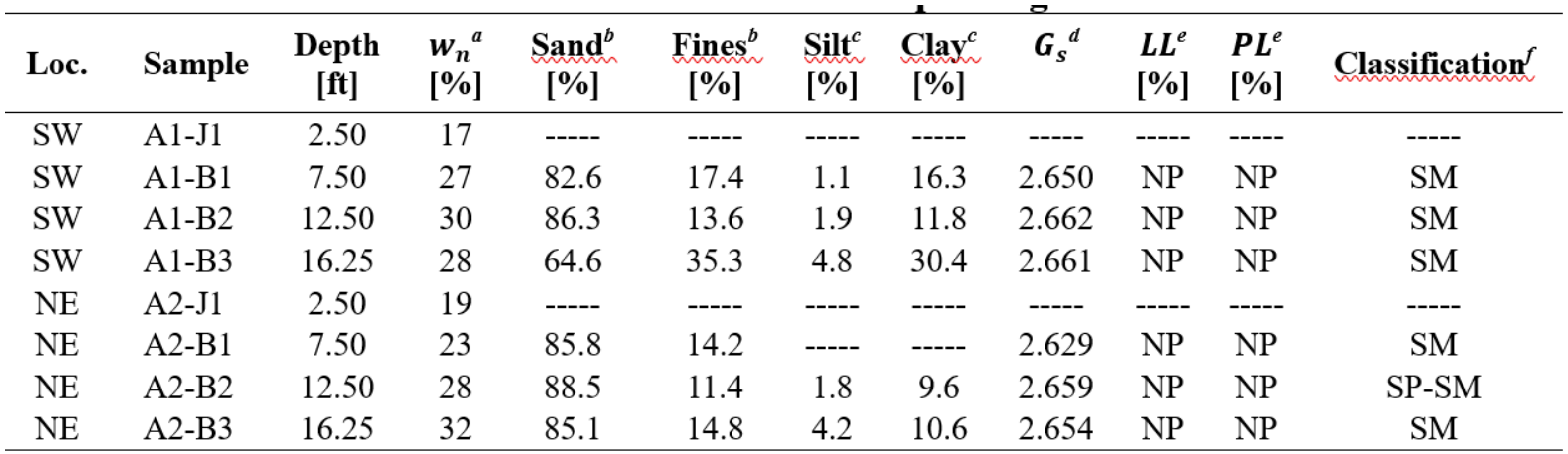
# Laboratory testing program: part 1

## TOPMOST GRANULAR SOILS

## Geologic soil profile

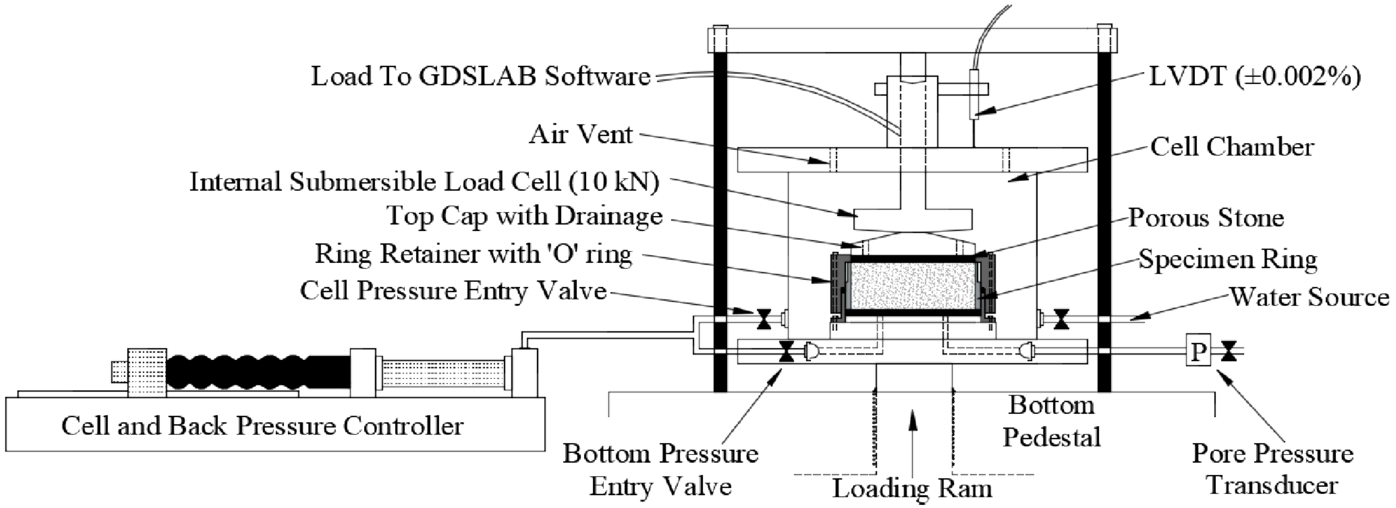


## Index tests

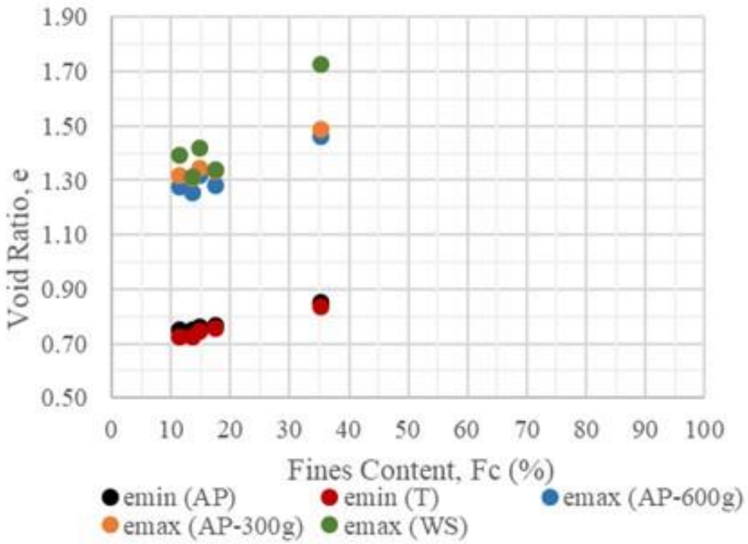


| Loc. | Sample | Depth [ft] | $w_n$[a] [%] | Sand[b] [%] | Fines[b] [%] | Silt[c] [%] | Clay[c] [%] | $G_s$[d] | $LL$[e] [%] | $PL$[e] [%] | Classification[f] |
|---|---|---|---|---|---|---|---|---|---|---|---|
| SW | A1-J1 | 2.50 | 17 | ----- | ----- | ----- | ----- | ----- | ----- | ----- | ----- |
| SW | A1-B1 | 7.50 | 27 | 82.6 | 17.4 | 1.1 | 16.3 | 2.650 | NP | NP | SM |
| SW | A1-B2 | 12.50 | 30 | 86.3 | 13.6 | 1.9 | 11.8 | 2.662 | NP | NP | SM |
| SW | A1-B3 | 16.25 | 28 | 64.6 | 35.3 | 4.8 | 30.4 | 2.661 | NP | NP | SM |
| NE | A2-J1 | 2.50 | 19 | ----- | ----- | ----- | ----- | ----- | ----- | ----- | ----- |
| NE | A2-B1 | 7.50 | 23 | 85.8 | 14.2 | ----- | ----- | 2.629 | NP | NP | SM |
| NE | A2-B2 | 12.50 | 28 | 88.5 | 11.4 | 1.8 | 9.6 | 2.659 | NP | NP | SP-SM |
| NE | A2-B3 | 16.25 | 32 | 85.1 | 14.8 | 4.2 | 10.6 | 2.654 | NP | NP | SM |

## CRS device for IL, CRS, CRL compressibility tests

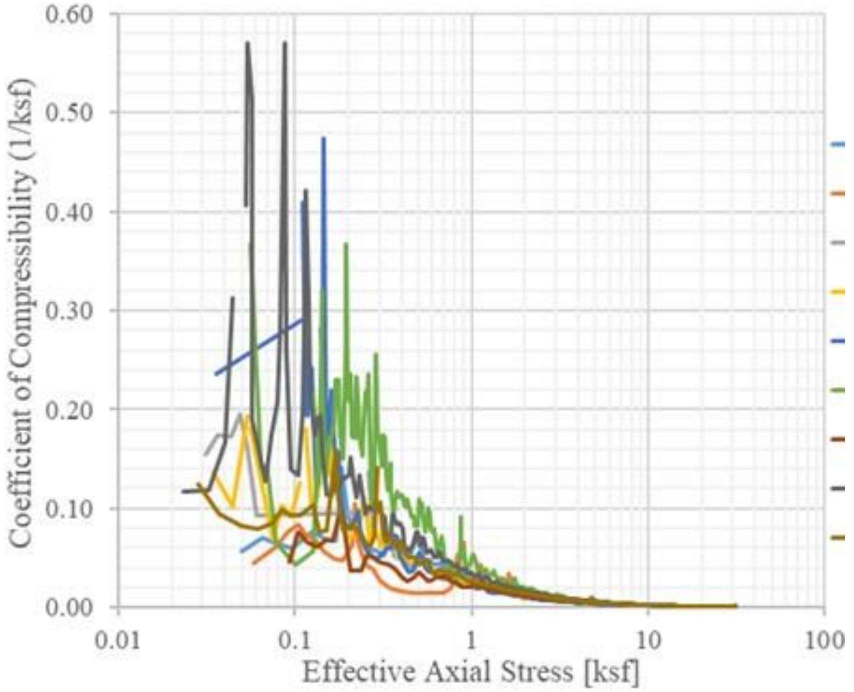


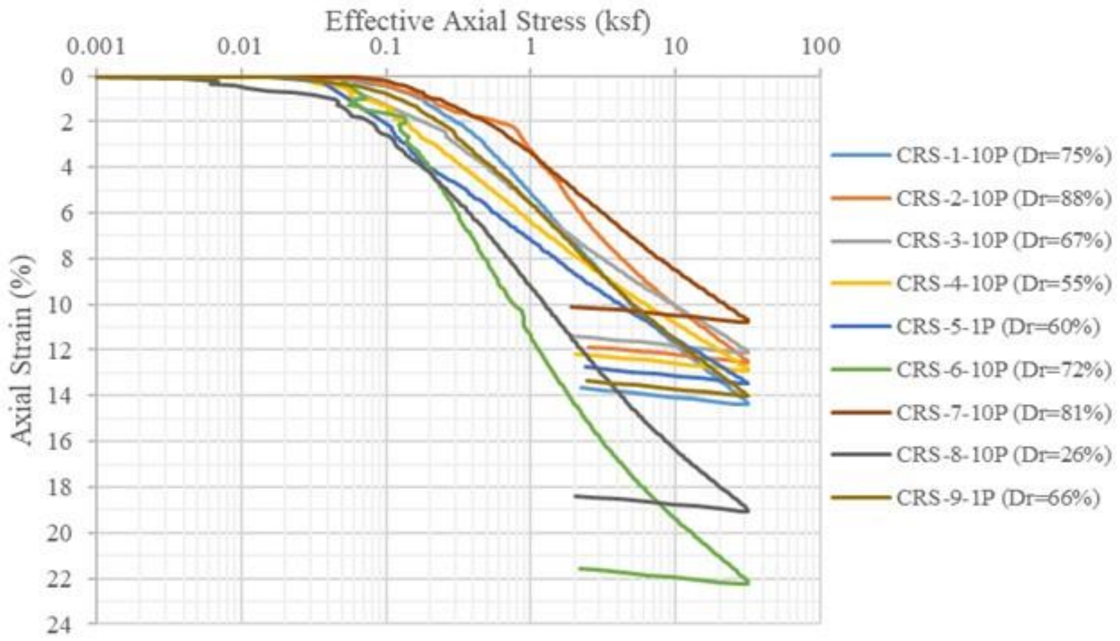
## $e_{min}$ and $e_{max}$ for $D_r$ calcs.



# Laboratory testing program: part 1

## Coefficient of compressibility and stress-strain behavior





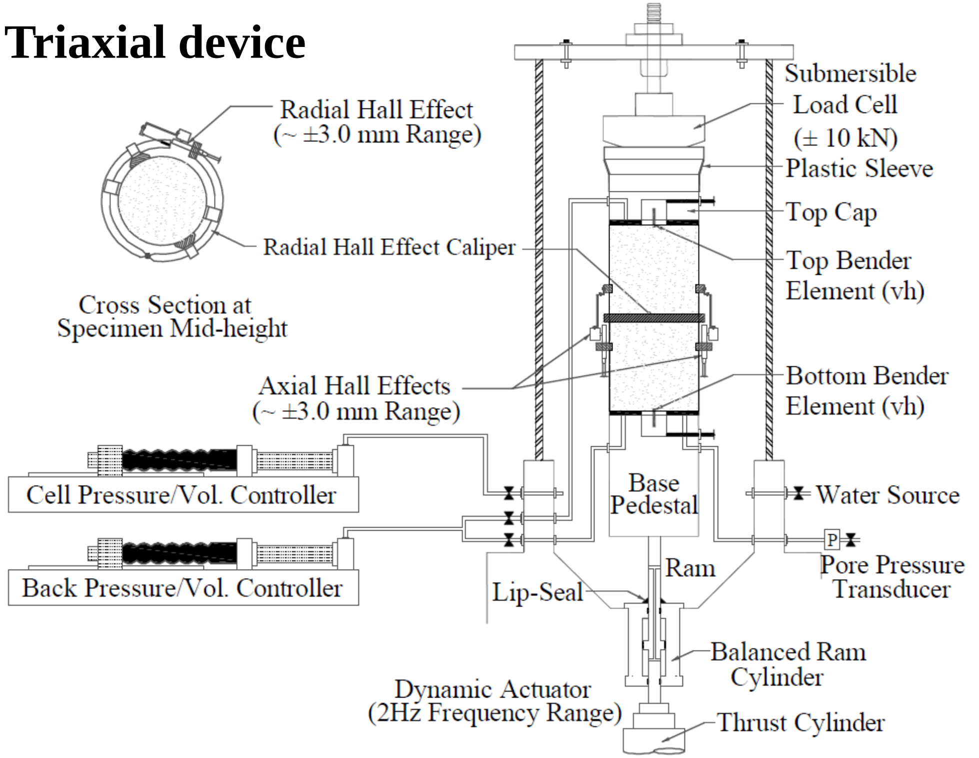
**Soil compressibility**

| Location | Sample | Specimen | Strain Rate [%/hr.] | Depth [ft] | $D_r$ [%] | $C_c$ | $C_r$ | $C_r/C_c$ |
|---|---|---|---|---|---|---|---|---|
| SW | A1-B1 | CRS-1-10P | 10 | 7.50 | 75 | 0.119 | 0.012 | 0.10 |
| SW | A1-B1 | CRS-2-10P | 10 | 7.50 | 88 | 0.115 | 0.012 | 0.10 |
| SW | A1-B2 | CRS-3-10P | 10 | 12.50 | 67 | 0.088 | 0.012 | 0.14 |
| SW | A1-B2 | CRS-4-10P | 10 | 12.50 | 55 | 0.093 | 0.013 | 0.14 |
| SW | A1-B2 | CRS-5-1P | 1 | 12.50 | 60 | 0.088 | 0.013 | 0.15 |
| SW | A1-B3 | CRS-6-10P | 10 | 16.25 | 72 | 0.177 | 0.013 | 0.07 |
| NE | A2-B2 | CRS-7-10P | 10 | 12.50 | 81 | 0.090 | 0.011 | 0.12 |
| NE | A2-B2 | CRS-8-10P | 10 | 12.50 | 26 | 0.157 | 0.013 | 0.08 |
| NE | A2-B2 | CRS-9-1P | 1 | 12.50 | 66 | 0.114 | 0.012 | 0.11 |

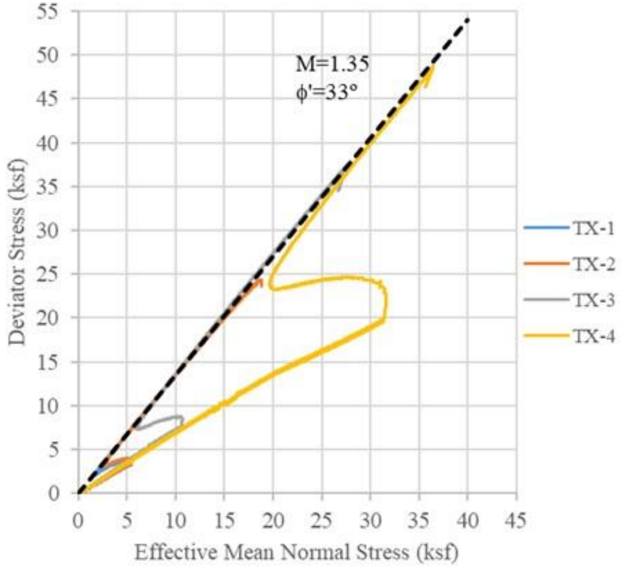
Note: For constrained modulus and
hydraulic conductivity (see report)

# Laboratory testing program: part 1

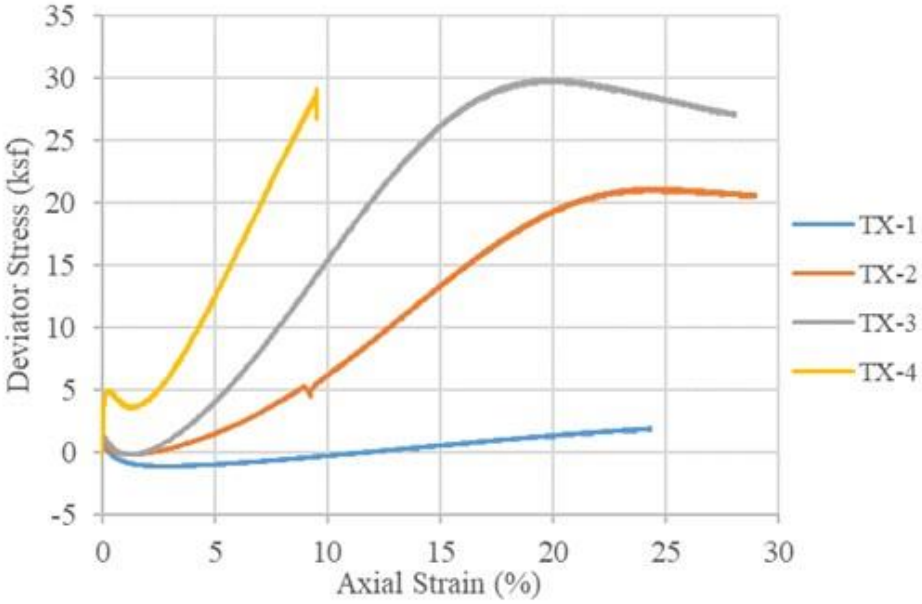
## Triaxial device



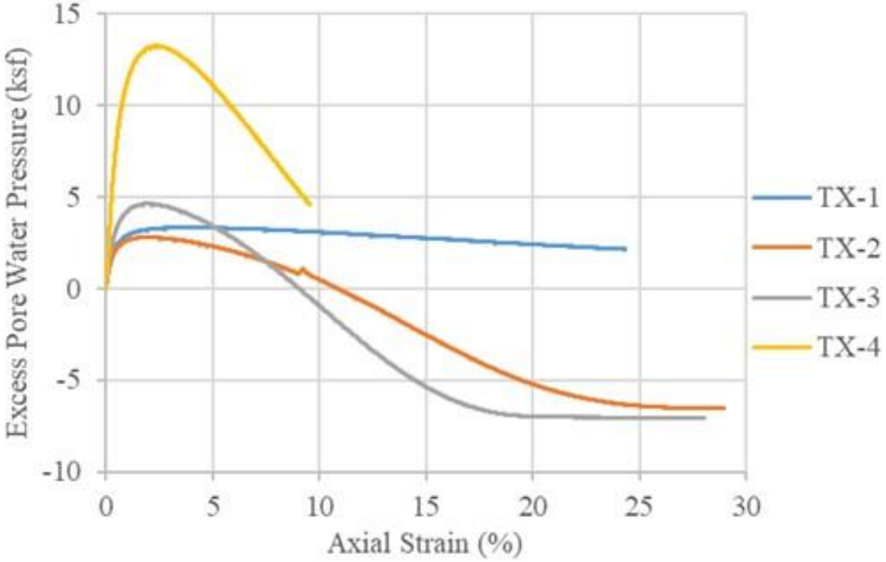
## Stress paths



## Shear stress-strain behavior

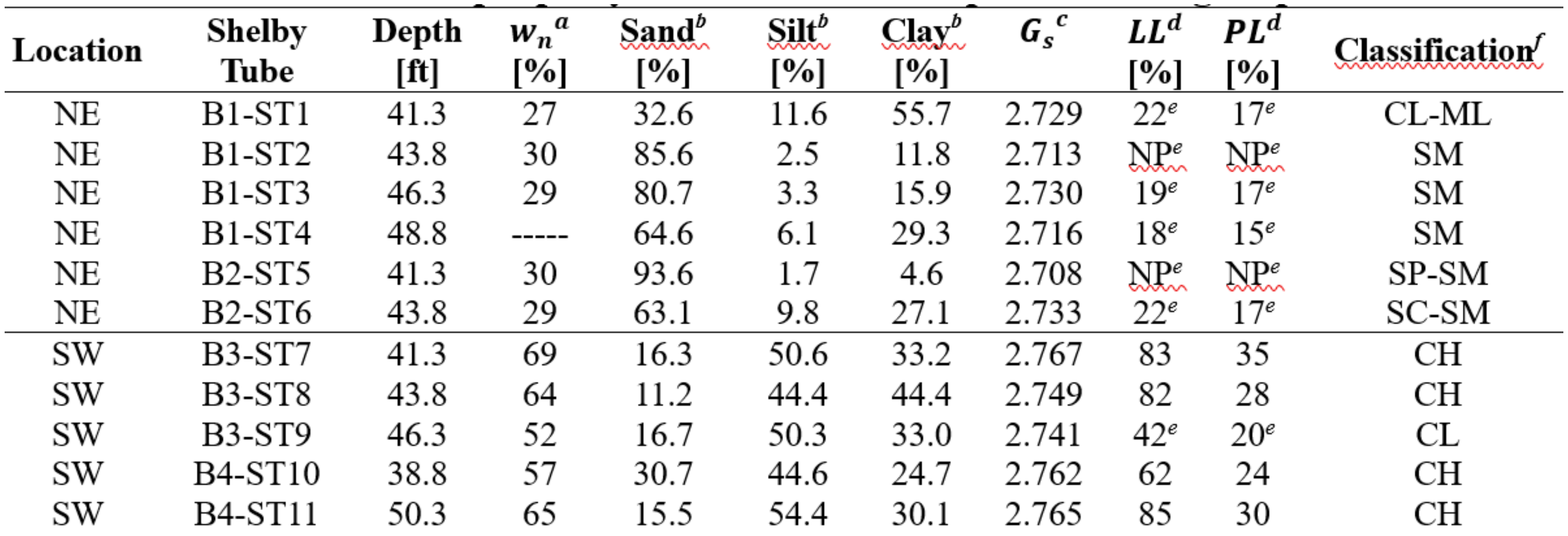


## Excess porewater pressures



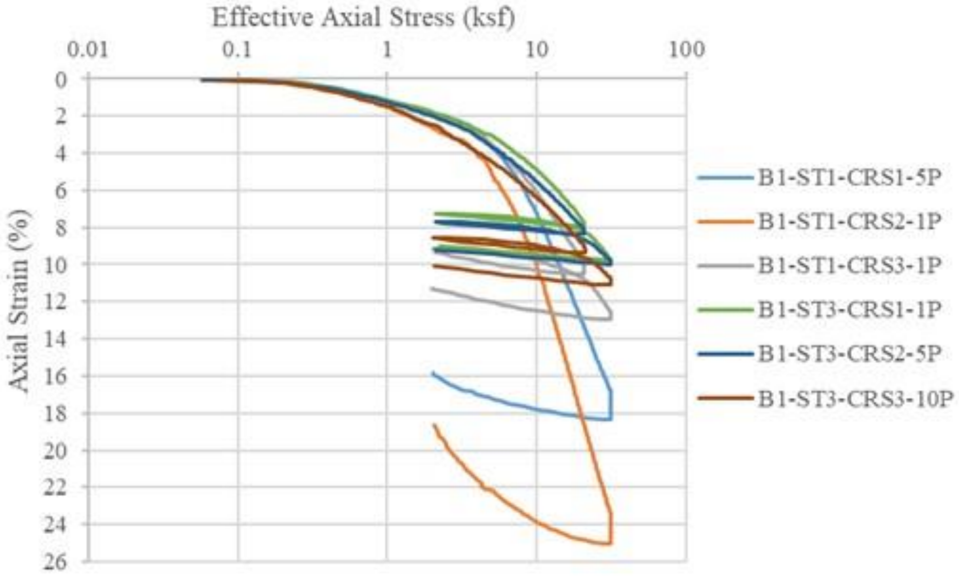
# Laboratory testing program: part 2

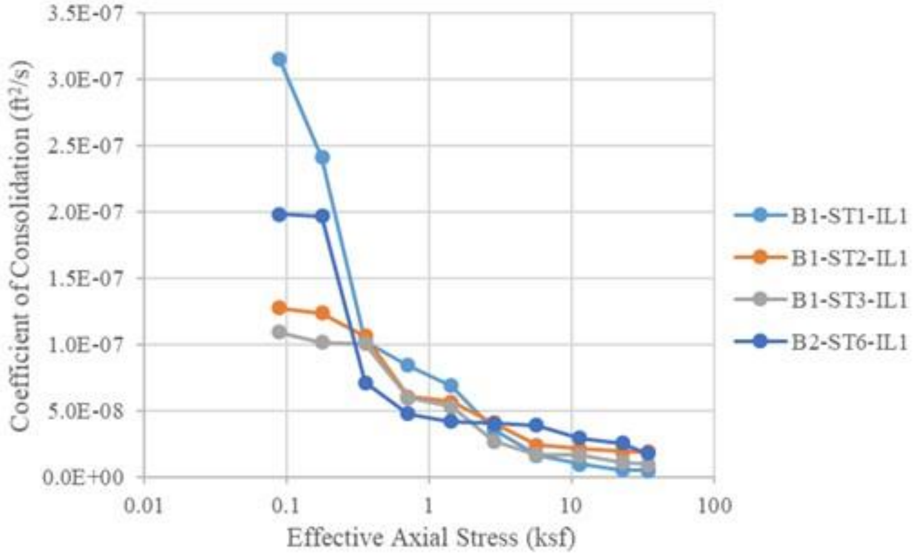
**DEEP FINE-GRAINED SOILS**



| Location | Shelby Tube | Depth [ft] | $w_n$[a] [%] | Sand[b] [%] | Silt[b] [%] | Clay[b] [%] | $G_s$[c] | $LL$[d] [%] | $PL$[d] [%] | Classification[f] |
|---|---|---|---|---|---|---|---|---|---|---|
| NE | B1-ST1 | 41.3 | 27 | 32.6 | 11.6 | 55.7 | 2.729 | 22[e] | 17[e] | CL-ML |
| NE | B1-ST2 | 43.8 | 30 | 85.6 | 2.5 | 11.8 | 2.713 | NP[e] | NP[e] | SM |
| NE | B1-ST3 | 46.3 | 29 | 80.7 | 3.3 | 15.9 | 2.730 | 19[e] | 17[e] | SM |
| NE | B1-ST4 | 48.8 | ----- | 64.6 | 6.1 | 29.3 | 2.716 | 18[e] | 15[e] | SM |
| NE | B2-ST5 | 41.3 | 30 | 93.6 | 1.7 | 4.6 | 2.708 | NP[e] | NP[e] | SP-SM |
| NE | B2-ST6 | 43.8 | 29 | 63.1 | 9.8 | 27.1 | 2.733 | 22[e] | 17[e] | SC-SM |
| SW | B3-ST7 | 41.3 | 69 | 16.3 | 50.6 | 33.2 | 2.767 | 83 | 35 | CH |
| SW | B3-ST8 | 43.8 | 64 | 11.2 | 44.4 | 44.4 | 2.749 | 82 | 28 | CH |
| SW | B3-ST9 | 46.3 | 52 | 16.7 | 50.3 | 33.0 | 2.741 | 42[e] | 20[e] | CL |
| SW | B4-ST10 | 38.8 | 57 | 30.7 | 44.6 | 24.7 | 2.762 | 62 | 24 | CH |
| SW | B4-ST11 | 50.3 | 65 | 15.5 | 54.4 | 30.1 | 2.765 | 85 | 30 | CH |

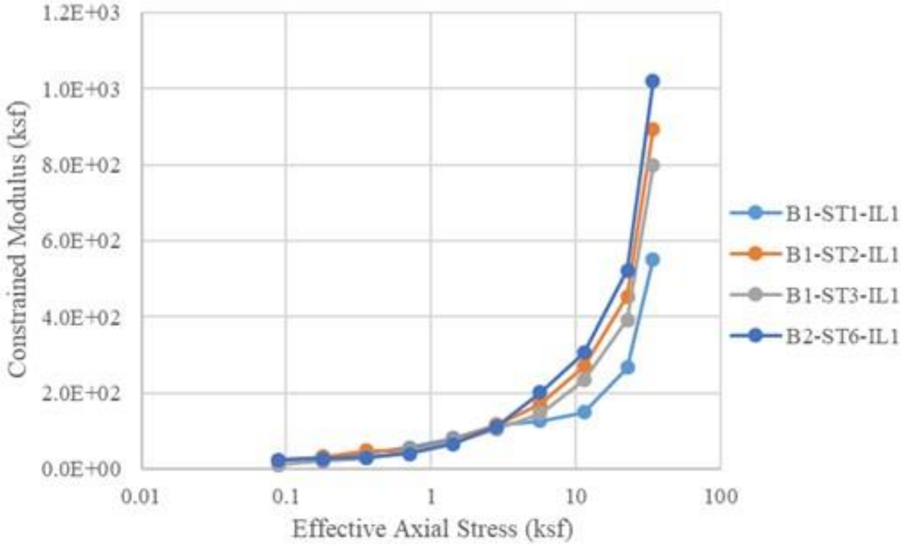
## Stress-strain behavior and coefficient of consolidation



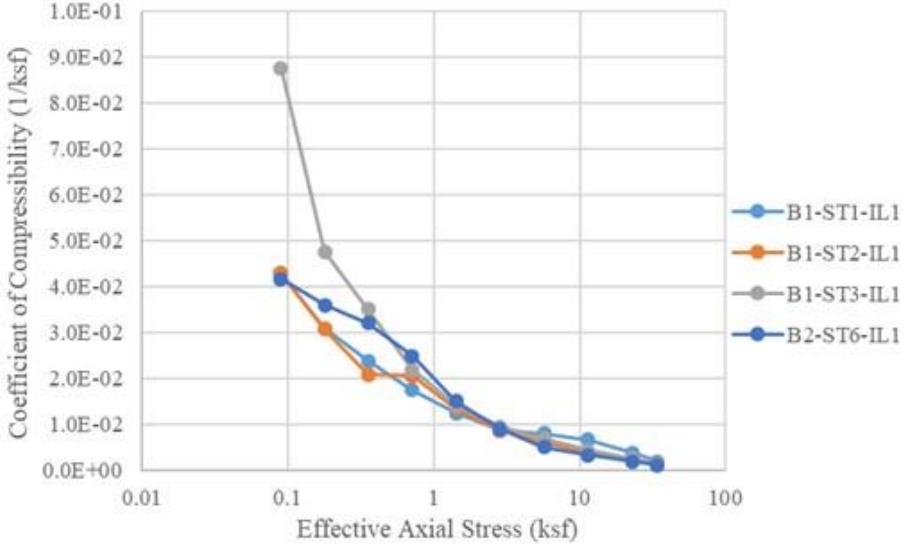


# Laboratory testing program: part 2

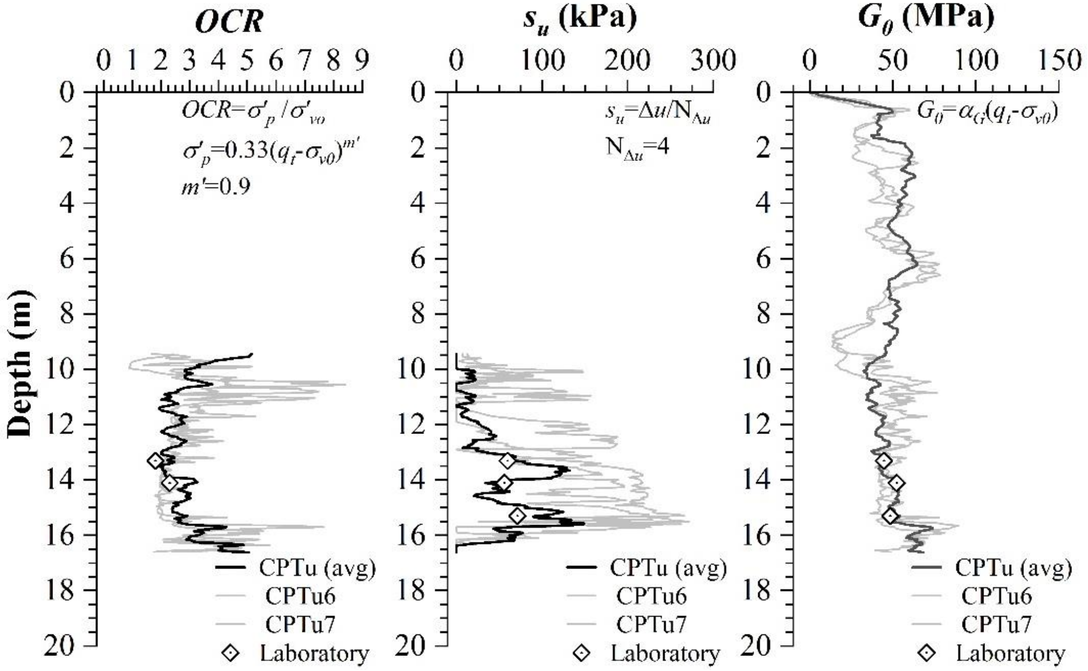
**Constrained modulus**



**Coefficient of compressibility**

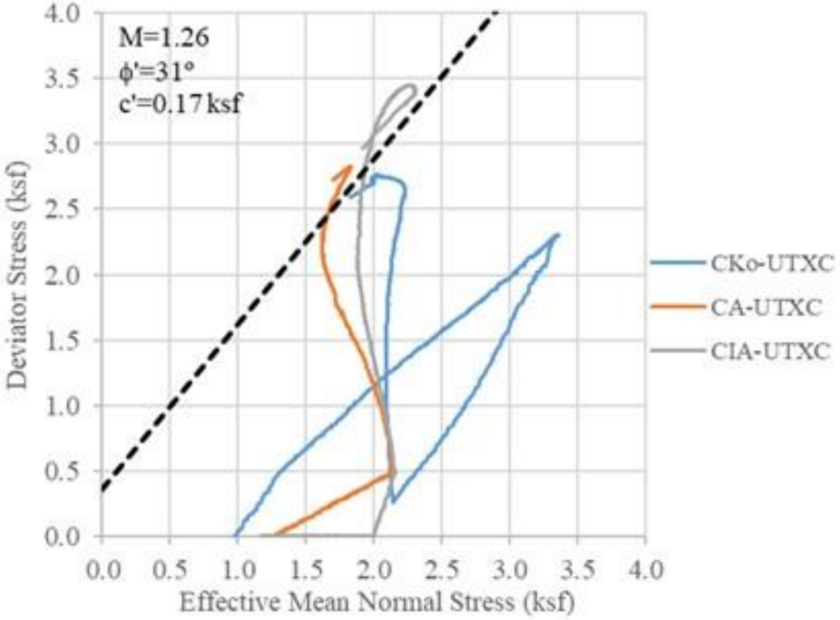


**Comparison with field tests**

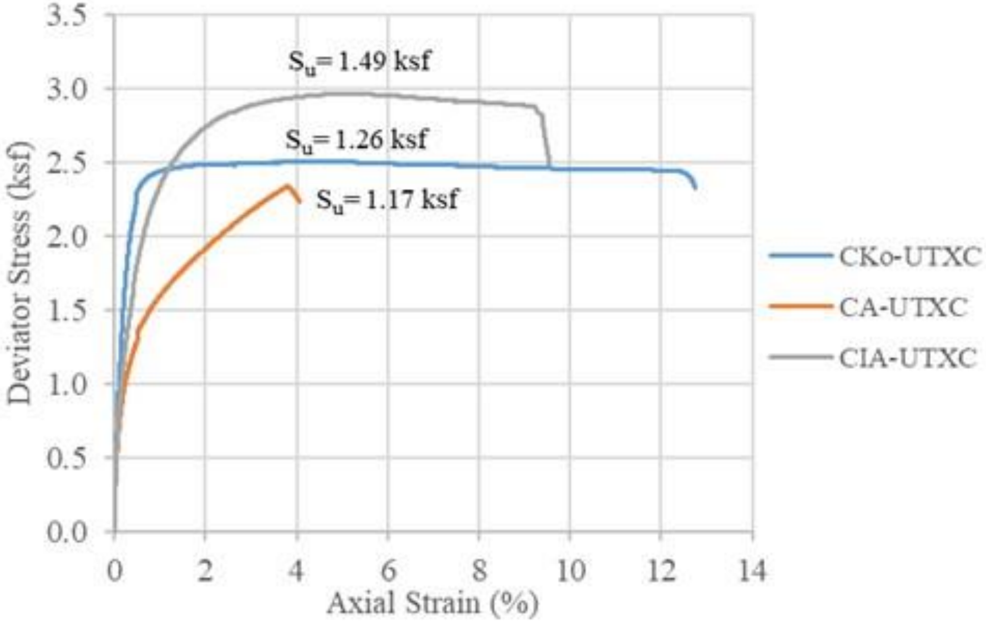


# Laboratory testing program: part 2

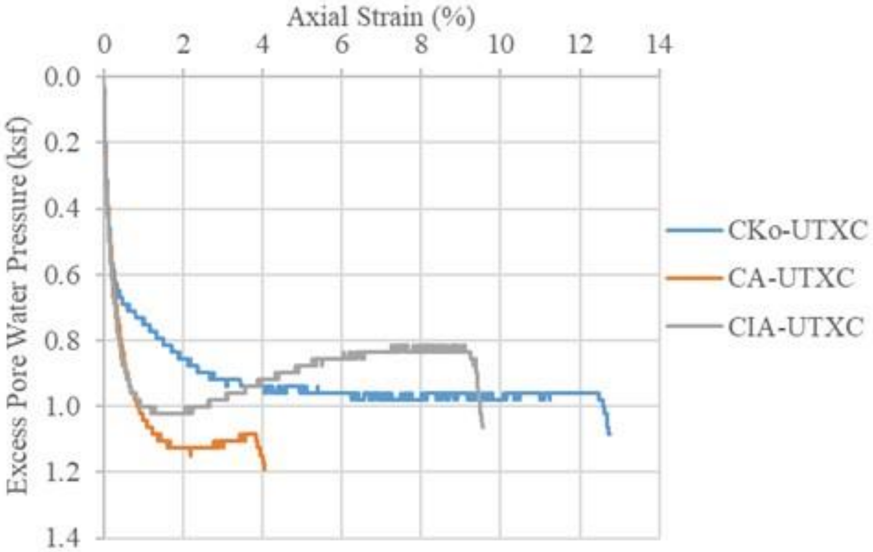
## Stress paths



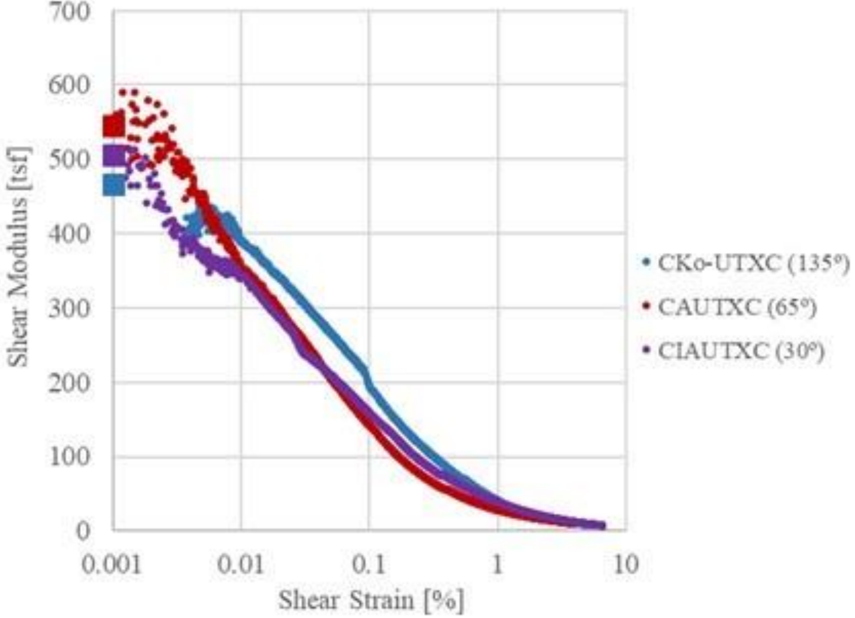
## Shear stress-strain behavior



## Excess porewater pressures



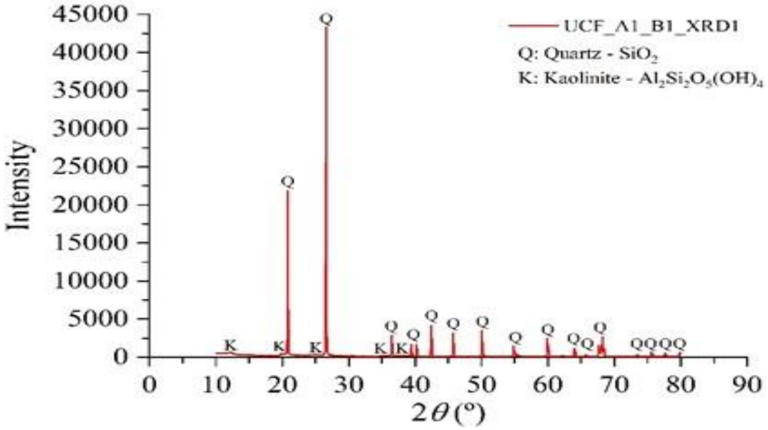
## Soil stiffness degradation



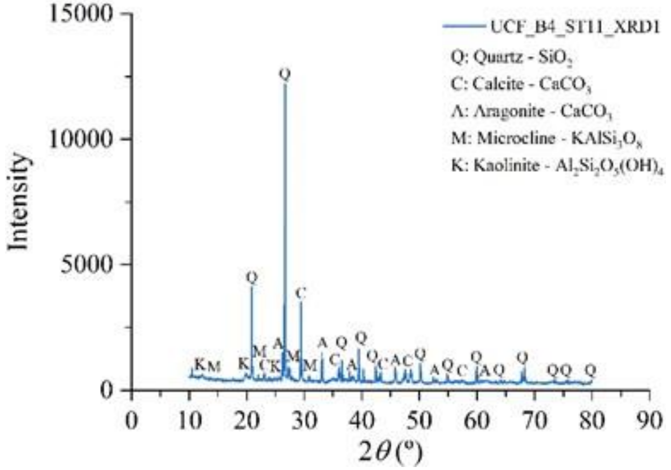
# Laboratory testing program: part 3

**X-Ray diffraction tests**

**Typical for granular soils**



**Typical for fine grained soils**



- Minerals from XRD (quartz, calcite, aragonite, microcline, and kaolinite) coincide with those found in Florida cover materials (Scott 1988, Upchurch et al. 2019).

- Presence of carbonate minerals explain high void ratios and soil compressibility.

- Microscopy tests matched previous studies on Cypresshead formation lithology (i.e., quartz and clay minerals formed during the late Pliocene to early Pleistocene) and Hawthorn group (i.e., clayey sands to silty clays formed during the Miocene).

- Preconsolidation of the Hawthorn group soil was mainly associated with change in porewater pressure, soil structure, and precipitation of cementing agents caused by the sea-level fluctuations during the Miocene age, aging, and presence of carbonate minerals, respectively.

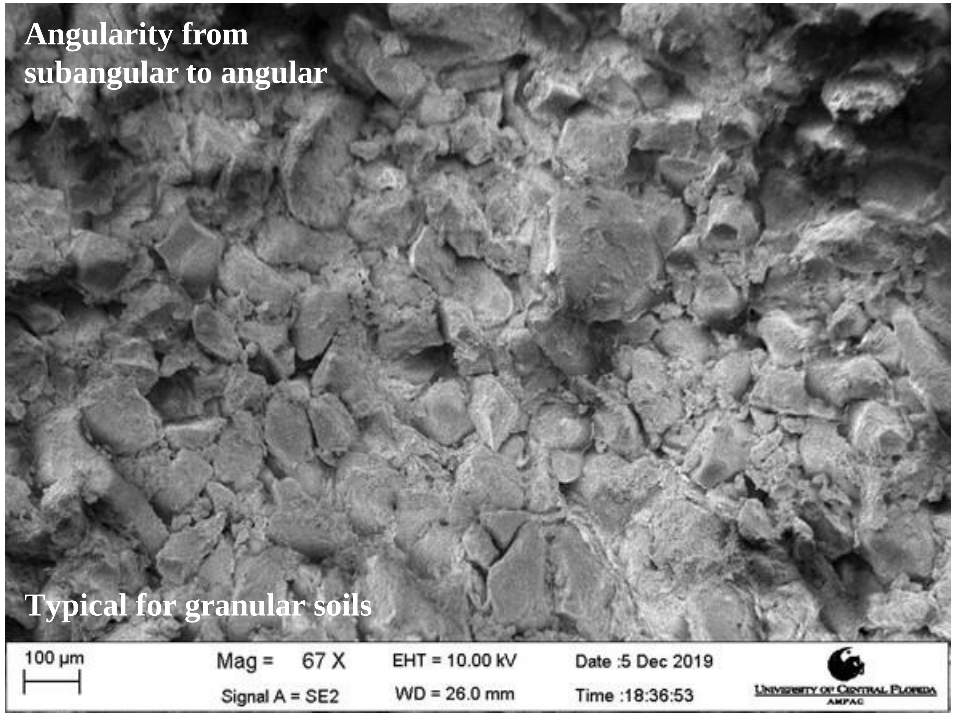
# Laboratory testing program: part 3

**SEM images**

- Results showed particles of plane sides with rounded to sharp edges, indicating angularity of the coarse-grained fraction from subangular to angular.

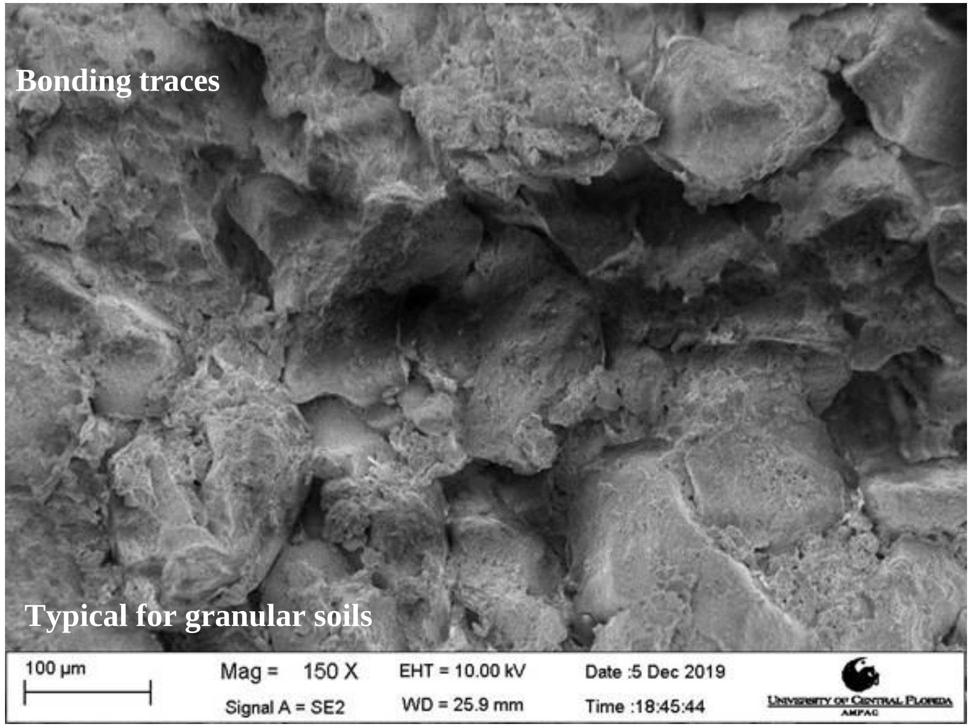
- Internal structure of the soil, in terms of bonding between particles, was attributed to the presence of kaolinite, found also in XRD tests.

- Results showed soil structure composed mainly of flat sheets (phyllosilicates), with presence of kaolinite, calcite, and aragonite based on XRD results and cementation between particles.



Angularity from subangular to angular

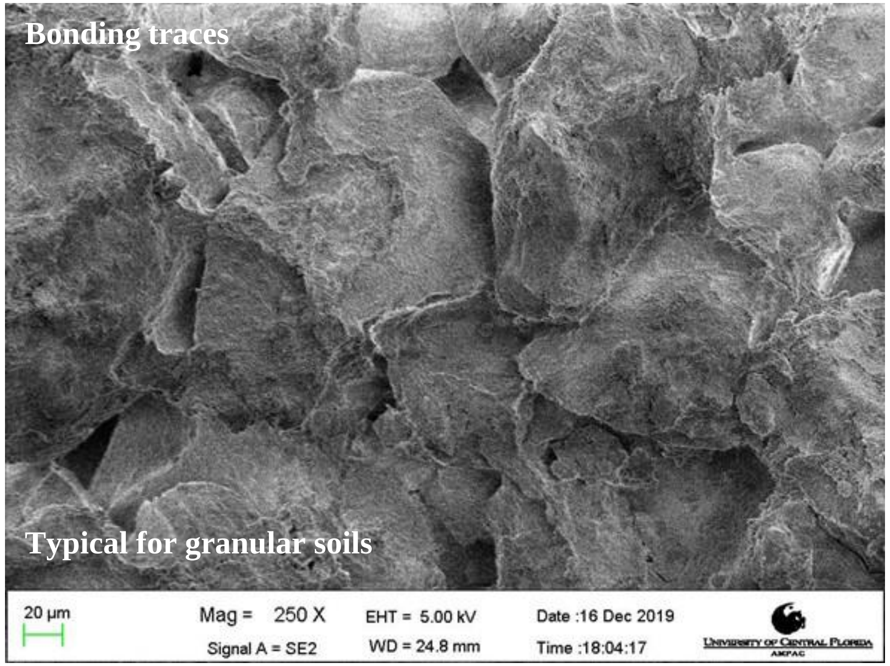
Typical for granular soils



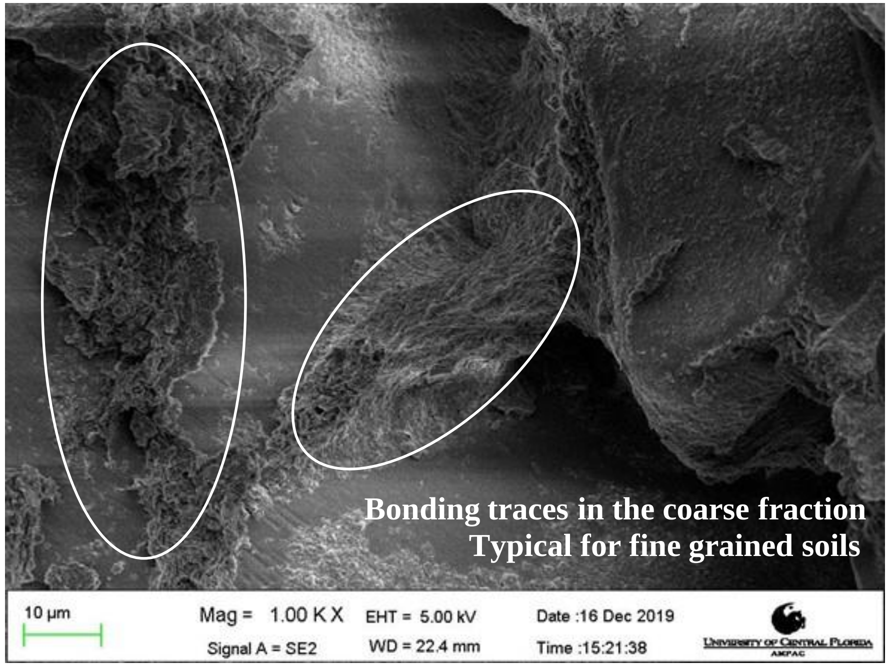
Bonding traces

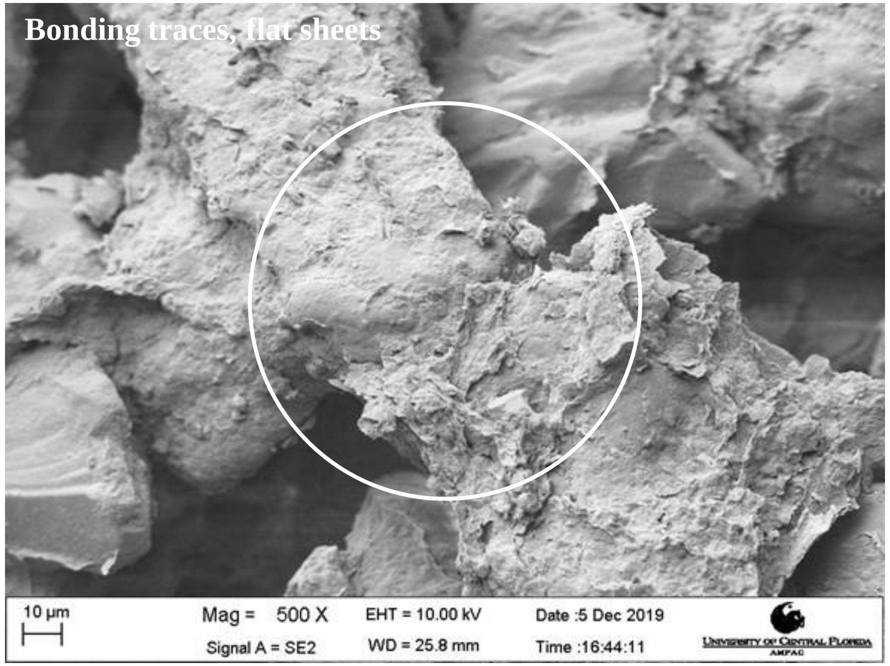
Typical for granular soils

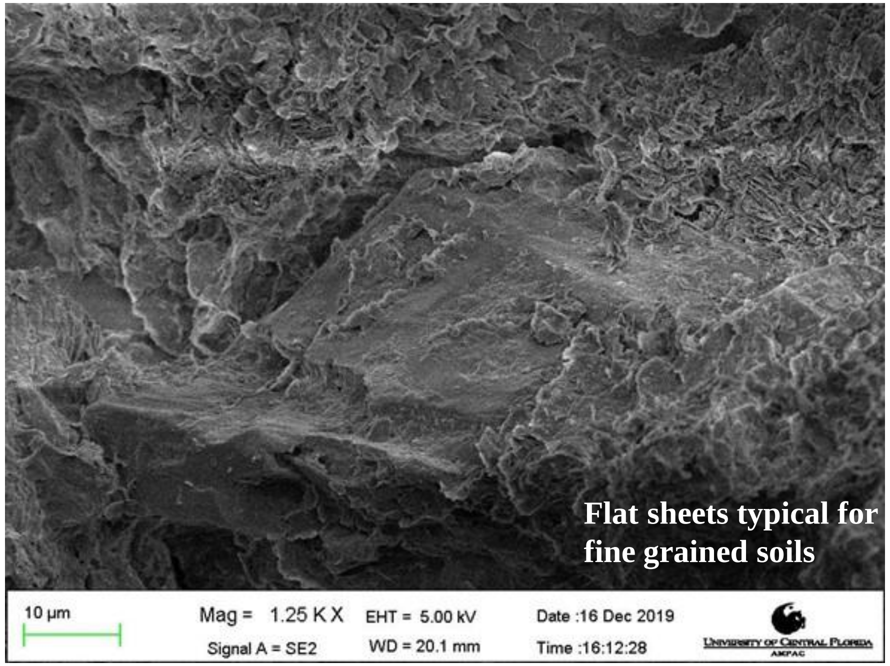
# Laboratory testing program: part 3


Bonding traces

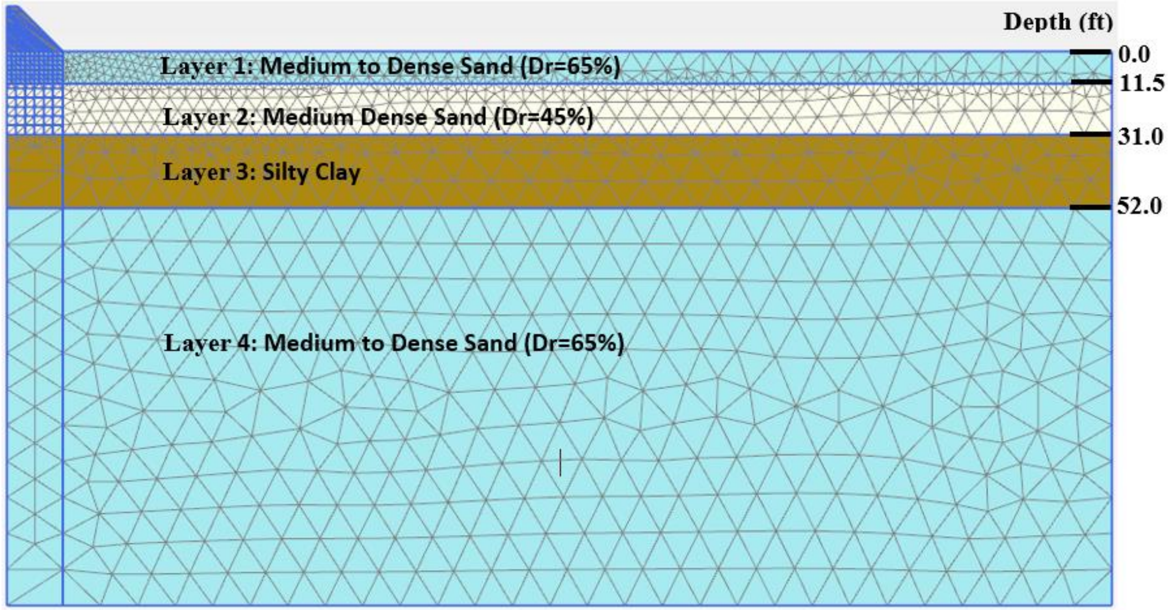
Typical for granular soils


Bonding traces in the coarse fraction
Typical for fine grained soils


Bonding traces, flat sheets


Flat sheets typical for
fine grained soils

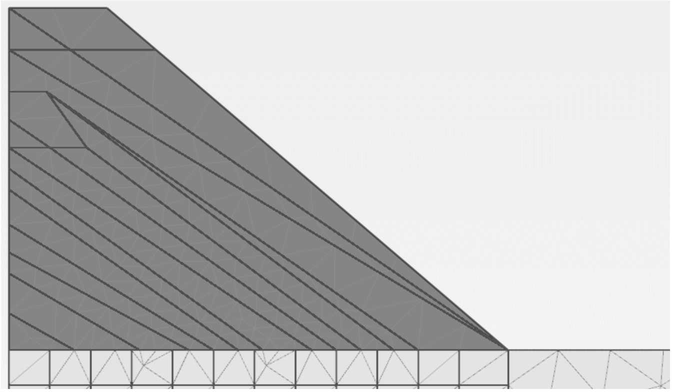
# Numerical modeling of conical load tests



**Finite element model:**

1. Set geometry and soil layers
2. Discretize domain
3. Set material parameters
4. Set boundary conditions
5. Initialize stresses ($K_0$-conditions)
6. Run in stage construction

**Conical load testing sequence**



Key issues in the model definition:

- Groundwater-soil behavior: <u>drained</u> vs. undrained vs. partially drained vs. fully coupled analyses

- Soil parameters: i) <u>drained conditions= effective stress parameters</u>; ii) partially drained conditions= effective stress parameters; iii) undrained conditions: either total or effective stresses depending on the finite element formulation

- Goal: Reproduce conical load testing

# Numerical modeling of conical load tests

Selection of soil model is key, depending on: purpose of the analysis, quality of data for calibration of parameters, type of information desired as outcome

Soil characteristics: Remember…. soils are incrementally non-linear plastic materials with hardening-softening or dilative and contractive behaviors

Use a model that only is as complicated as justified/needed!

Some constitutive soil models and # of parameters:

**Easy to calibrate… but ignore numerous features of soil behavior**

Mohr Coulomb: 5 parameters
Duncan Chang: 8 parameters
Modified Cam Clay: 5 parameters
Anisotropic MCC: 6

**Good balance!**

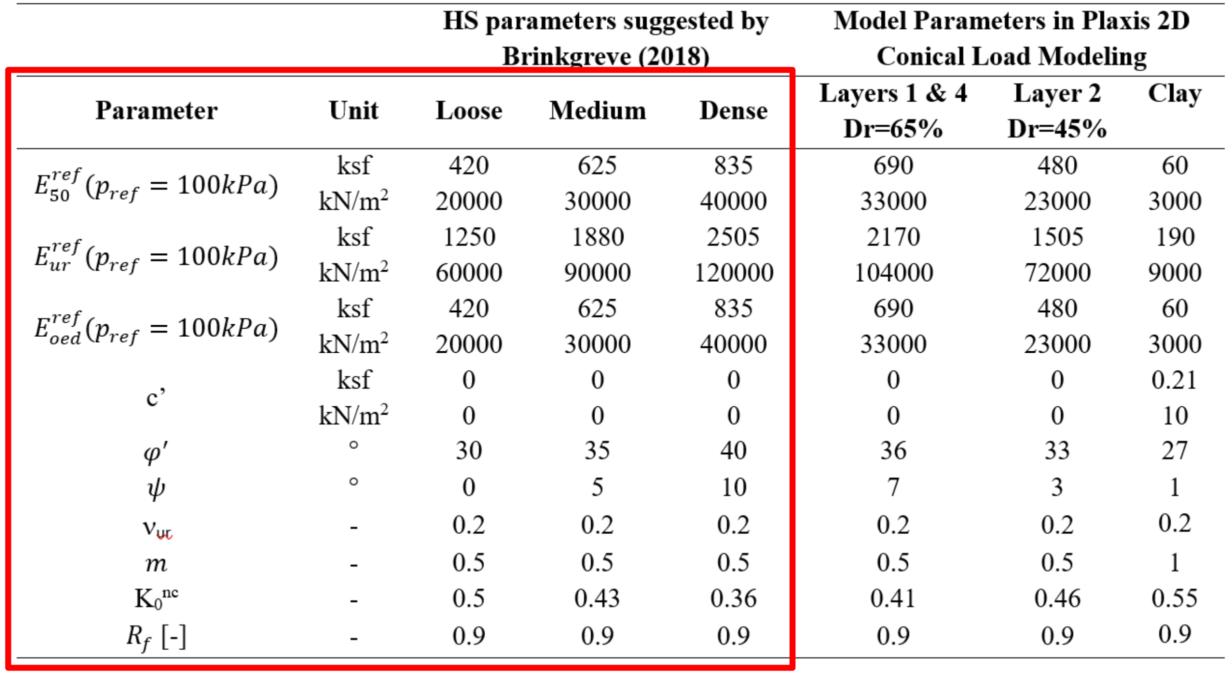
Hardening Soil: 11 parameters
Hardening Soil Small: 13 parameters

**Numerous soil parameters, not easy to calibrate… but capture soil behavior**

Hypoplasticity Clay: 11 parameters
Hypoplasticity Sand: 11 parameters
MIT-S1 (13 clay, 14 sand)
MIT-E3 (15 for clay)
PM4 sand: primary input: 6, secondary: 18
Dafalias and Manzari (BSPM): 14 parameters
PDMYM: 21 parameters

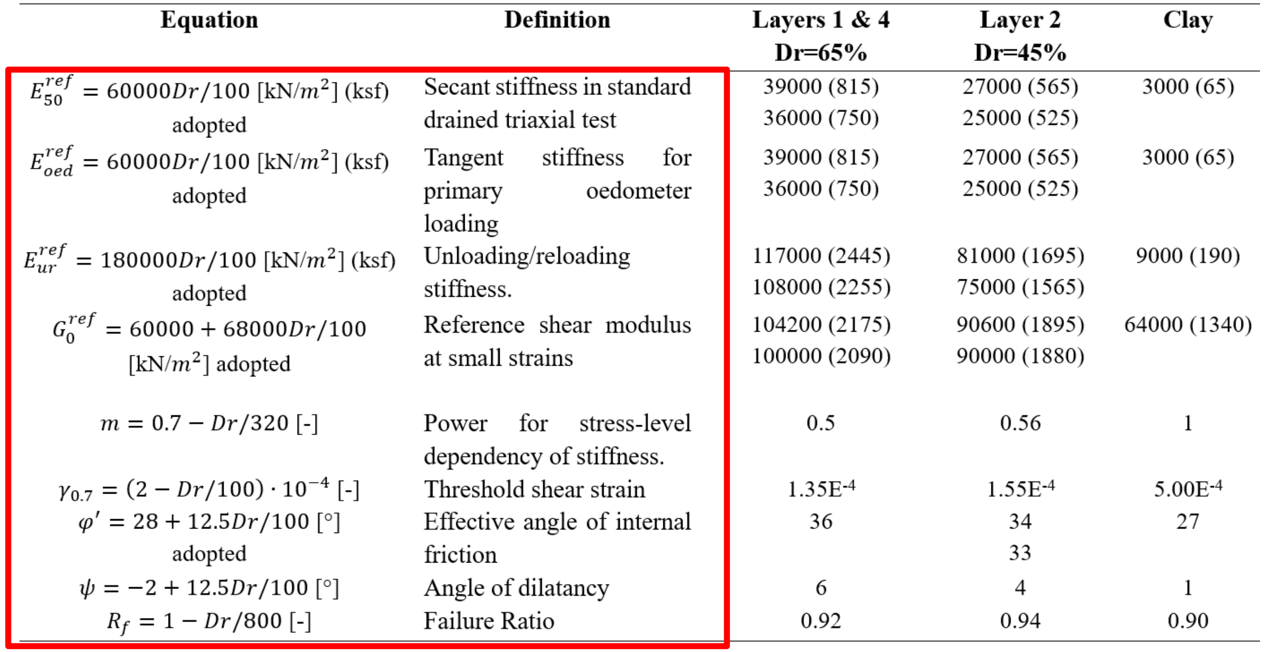
# Constitutive model parameters

**Hardening soil model (HS)**



| Parameter | Unit | HS parameters suggested by Brinkgreve (2018) | | | Model Parameters in Plaxis 2D Conical Load Modeling | | |
|---|---|---|---|---|---|---|---|
| | | Loose | Medium | Dense | Layers 1 & 4 Dr=65% | Layer 2 Dr=45% | Clay |
| $E_{50}^{ref} (p_{ref} = 100kPa)$ | ksf | 420 | 625 | 835 | 690 | 480 | 60 |
| | kN/m$^2$ | 20000 | 30000 | 40000 | 33000 | 23000 | 3000 |
| $E_{ur}^{ref} (p_{ref} = 100kPa)$ | ksf | 1250 | 1880 | 2505 | 2170 | 1505 | 190 |
| | kN/m$^2$ | 60000 | 90000 | 120000 | 104000 | 72000 | 9000 |
| $E_{oed}^{ref} (p_{ref} = 100kPa)$ | ksf | 420 | 625 | 835 | 690 | 480 | 60 |
| | kN/m$^2$ | 20000 | 30000 | 40000 | 33000 | 23000 | 3000 |
| c’ | ksf | 0 | 0 | 0 | 0 | 0 | 0.21 |
| | kN/m$^2$ | 0 | 0 | 0 | 0 | 0 | 10 |
| $\varphi'$ | ° | 30 | 35 | 40 | 36 | 33 | 27 |
| $\psi$ | ° | 0 | 5 | 10 | 7 | 3 | 1 |
| $\nu_{ur}$ | - | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 | 0.2 |
| $m$ | - | 0.5 | 0.5 | 0.5 | 0.5 | 0.5 | 1 |
| $K_0^{nc}$ | - | 0.5 | 0.43 | 0.36 | 0.41 | 0.46 | 0.55 |
| $R_f$ [-] | - | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 | 0.9 |

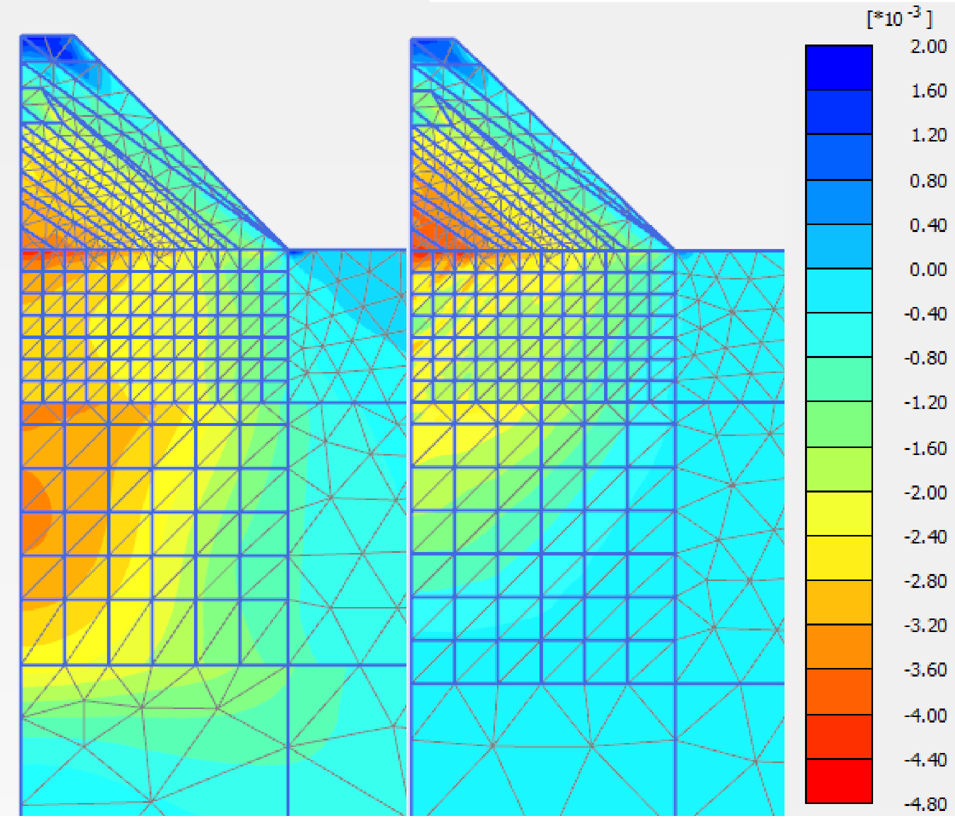
**Hardening soil small model (HSS)**



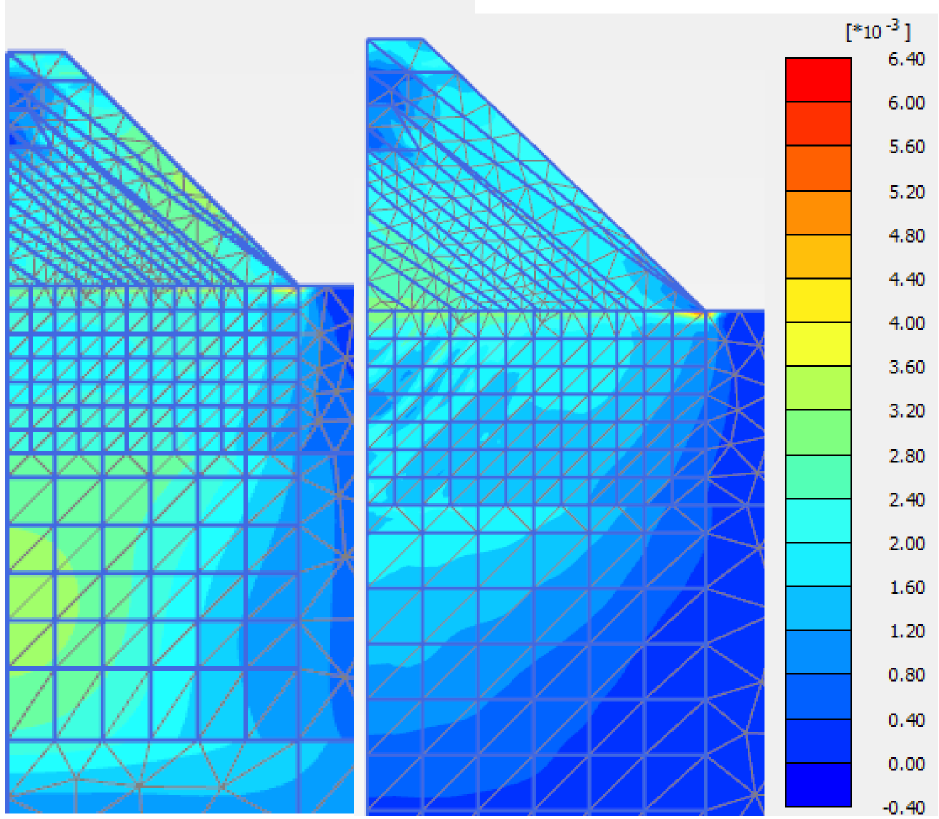
| Equation | Definition | Layers 1 & 4 Dr=65% | Layer 2 Dr=45% | Clay |
|---|---|---|---|---|
| $E_{50}^{ref} = 60000Dr/100$ [kN/$m^2$] (ksf)<br>adopted | Secant stiffness in standard drained triaxial test | 39000 (815)<br>36000 (750) | 27000 (565)<br>25000 (525) | 3000 (65) |
| $E_{oed}^{ref} = 60000Dr/100$ [kN/$m^2$] (ksf)<br>adopted | Tangent stiffness for primary oedometer loading | 39000 (815)<br>36000 (750) | 27000 (565)<br>25000 (525) | 3000 (65) |
| $E_{ur}^{ref} = 180000Dr/100$ [kN/$m^2$] (ksf)<br>adopted | Unloading/reloading stiffness. | 117000 (2445)<br>108000 (2255) | 81000 (1695)<br>75000 (1565) | 9000 (190) |
| $G_0^{ref} = 60000 + 68000Dr/100$ [kN/$m^2$] adopted | Reference shear modulus at small strains | 104200 (2175)<br>100000 (2090) | 90600 (1895)<br>90000 (1880) | 64000 (1340) |
| $m = 0.7 - Dr/320$ [-] | Power for stress-level dependency of stiffness. | 0.5 | 0.56 | 1 |
| $\gamma_{0.7} = (2 - Dr/100) \cdot 10^{-4}$ [-] | Threshold shear strain | 1.35E$^{-4}$ | 1.55E$^{-4}$ | 5.00E$^{-4}$ |
| $\varphi' = 28 + 12.5Dr/100$ [°]<br>adopted | Effective angle of internal friction | 36 | 34<br>33 | 27 |
| $\psi = -2 + 12.5Dr/100$ [°] | Angle of dilatancy | 6 | 4 | 1 |
| $R_f = 1 - Dr/800$ [-] | Failure Ratio | 0.92 | 0.94 | 0.90 |

**Recommendation!**

# Numerical modeling of conical load tests

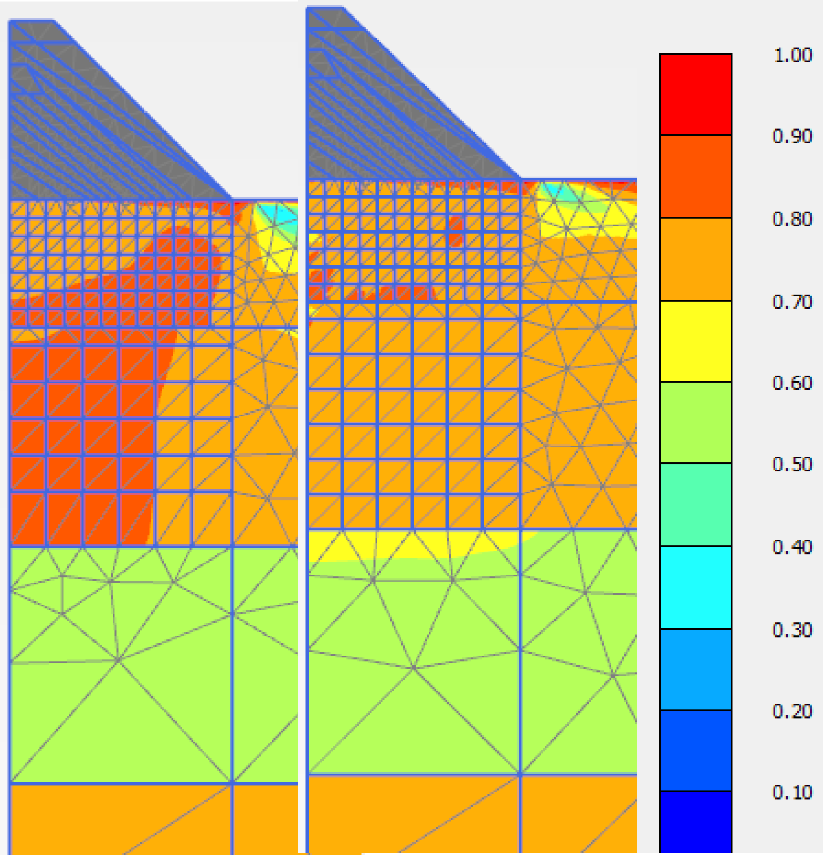


Contours of cartesian vertical strains at the end of conical loading:
*HS model (left)* and *HSS model (right)*



Contours of shear strains (≈ 0.3%) induced by conical load testing:
*HS model (left)* and *HSS model (right)*

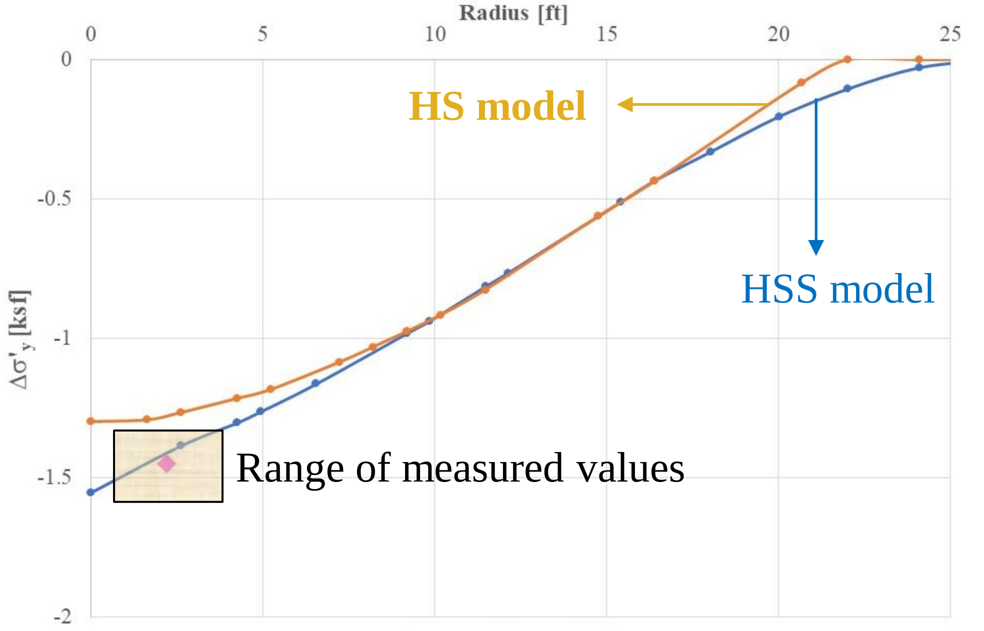
# Numerical modeling of conical load tests



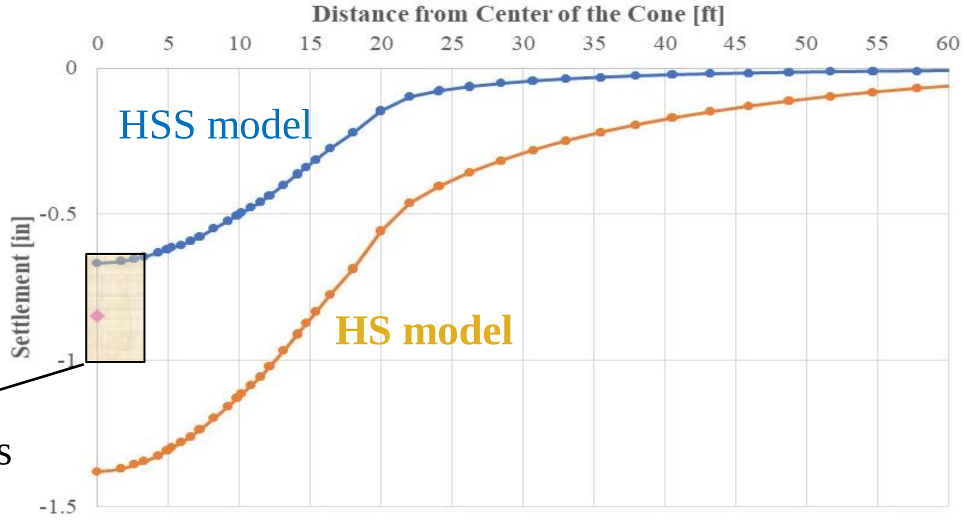
Contours of <u>relative shear stresses</u> ($\tau_{rel}$) induced by conical load testing: *HS model (left)* and *HSS model (right)*

$$\tau_{rel} = \frac{\tau_{mob}}{\tau_{max}}$$

## Pressure at ground surface

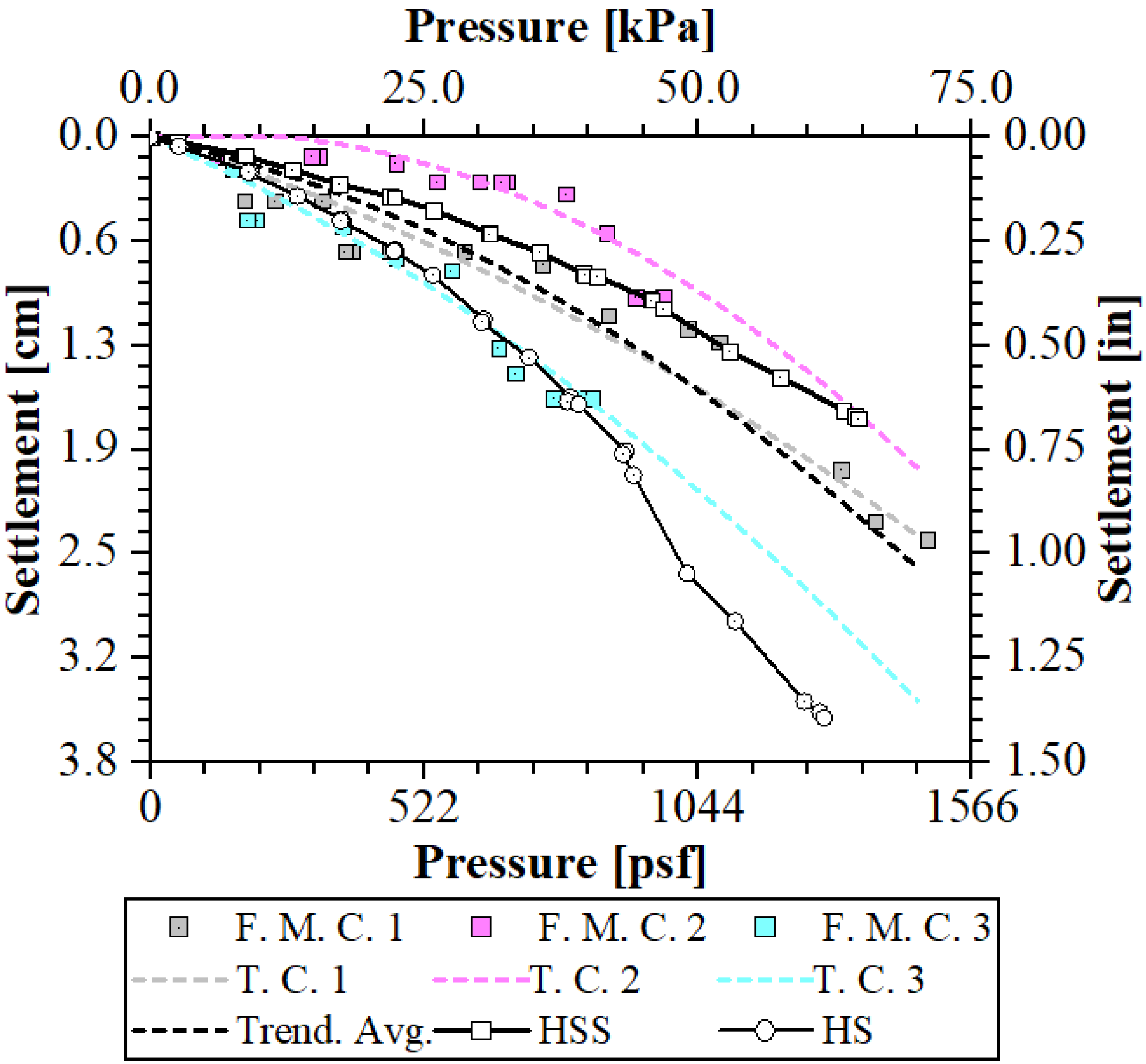


## Settlement at ground surface



Range of measured values

# Numerical modeling of conical load tests



F. M. C. # = Field Measurement @ Cone #

T. C. # = Trendline @ Cone #  - HSS = Hardening Soil Small model

HS = Hardening Soil model

# Conclusions and recommendations

- Recommended correlations for $E$ calculations:

Using CPTs: Buisman (1940), De Beer (1967), Bachelier and Parez (1965), Vesic (1970), Sanglerat et al. (1972), DeBeer (1974), Schmertmann (1970), and Schmertmann (1978).

Using SPTs: Webb (1969), Chaplin (1963), Papadopoulos (1982), Kulhawy and Mayne (1990), Webb (1969), Bowles (1996), FHWA 02-034.

Using DMTs: Lutenegger et al. (1995) and Bowles (1996).

- Recommended procedures for $S_i$ calculations:

D'Appolonia (1968), Schmertmann (1978, 1986), Bowles (1987), Mayne and Poulos (1999), Terzaghi et al. (1967), Meyerhof (1965, 1974), Peck et al. (1969, 1974).

- The conclusions drawn are only applicable to $S_i$. When $S_c$ or $S_s$ are expected (e.g., presence of clays or organics), those two components should be added and considered separately.

- *Ue* did not build-up in topmost layers, allowing the conical load sequence to take place by letting dissipation of *Ue* in the stressed soil layers. Thus, settlements measured were only $S_i$ (not $S_c$ or $S_s$).

- Conical load tests provide a good estimate of $S_i$ and are easy, fast, and reliable. For shallow loadings, make sure expected mobilized strains are in the same order of magnitude as those mobilized during conical load testing.

# Conclusions and recommendations

- For calculation of soil stiffness, the strain level in the soil due to applied loading is very important. Soils reduce their stiffness as a function of the mobilized strains. This dependency is key when computing ground deformations. Consider small-strain soil behavior!

- The geotechnical models reproduced well the conical load tests. Settlement at the centerline of the cone was better predicted with HSS. Results computed with HS overpredicted the overall measured response.

- Certain degree of conservatism was found when ignoring small strain soil behavior, as long as the input parameters are calibrated correctly! … But for reliable predictions of ground deformations, small-strain stiffness and its degradation should be considered in the design.

- Do not use Mohr-Coulomb elastic-perfectly plastic soil models for the calculation of soil deformations since the stress-strain characteristics of soils at strain levels below 0.1% cannot be reproduced accurately with such models. If measured and computed values of soil deformations match using Mohr-Coulomb models is because of compensating errors!

- Soil model parameters in this project are useful for future use in Florida for the calculation of $S_i$, mostly in granular soils. See final report for correlations of parameters for HS and HSS models with $D_r$ of granular soils.

# Conclusions and recommendations

- Computed settlement troughs using HS and HSS had similar distributions, but max. settlements using HS doubled those computed with HSS. Higher strains computed using HS were a consequence of a reduced stiffness inherent in the constitutive formulation in relation to more accurate models like HSS that consider small strain soil behavior.

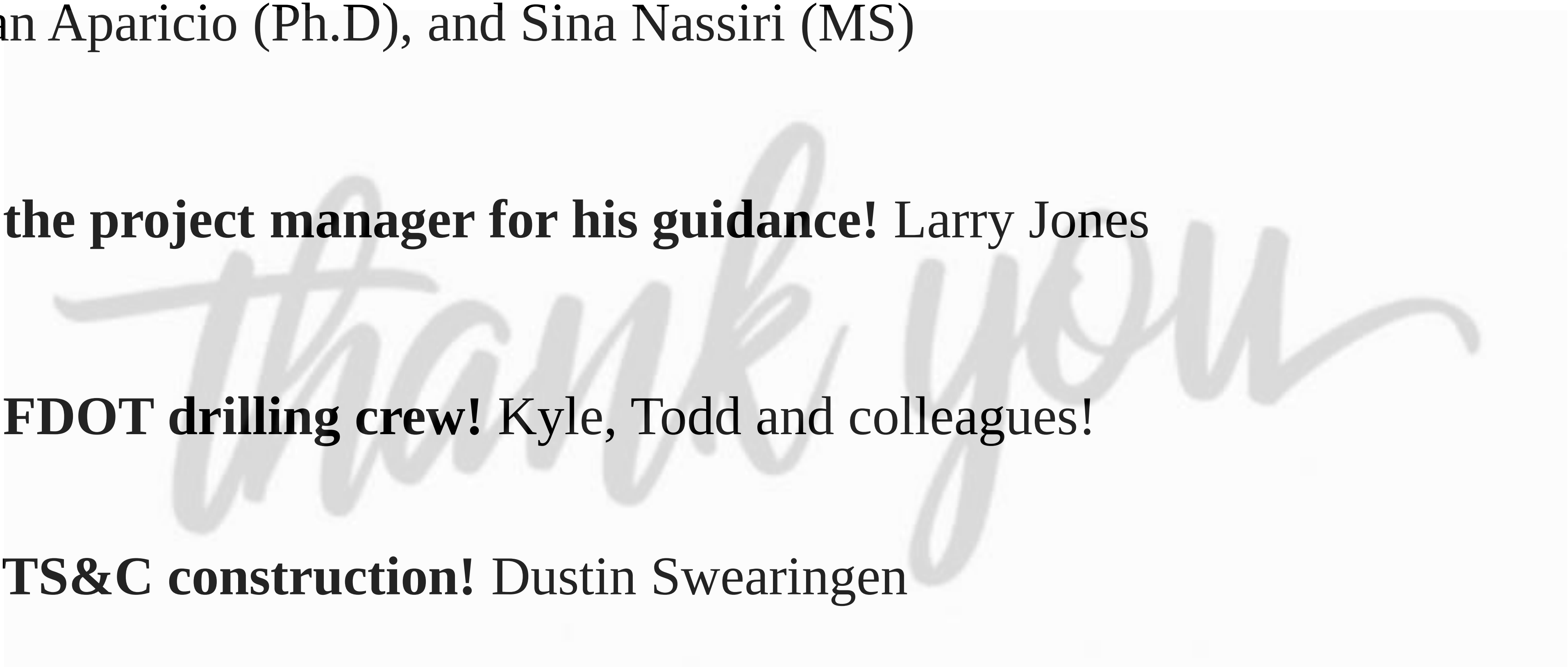
- Even though the estimation of soil parameters were based on correlations with commercially available field tests (mainly SPT, CPT, DMT), valuable information could also be obtained from other tests that provide information about small-strain soil behavior (e.g., “seismic” piezocones SCPTu or “small-strain” piezocones, and in the lab using bender elements).

- For projects involving multiple stress paths (e.g., excavations, installation of shallow/deep foundations, tunnels, etc.) that mobilize wide ranges of shear strains, Mohr-Coulomb-based models oversimplify soil behavior. More advanced constitutive soil models are recommended instead to capture more realistic features of soil response due to construction-induced loadings.

- Compressibility of soils with angular-shaped grains plus high carbonate contents can display a more compressible response than sands with rounded grains plus clean quartz. Granular shape and mineralogy can be used to understand why soil compressibility is low, medium, or high.

# Acknowledgements

**To our students for their dedication!**
Sergio Marin (Ph.D), A. Felipe Uribe-Henao (Ph.D), Jaime A. Mercado (Ph.D), Alan Aparicio (Ph.D), and Sina Nassiri (MS)



**To the project manager for his guidance!** Larry Jones

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**To TS&C construction!** Dustin Swearingen

**To FDOT in general!** For the research opportunity and support providing education and research training for 7 MS/Ph.D. at UCF!

# Questions

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