Phase II: Field Load Testing of Shallow Foundations in Florida Limestone, FDOT BDV31-977-124

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August 2022

Presentation Outline

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- Overview of Miami Limestone and Ocala Limestone (3 Load Tests)
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- Beam on Nonlinear Winkler Foundation (BNWF) model
- Current Findings
- Timeline and Acknowledgements

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Project Background

Phase I (FDOT BDV31-977-51):

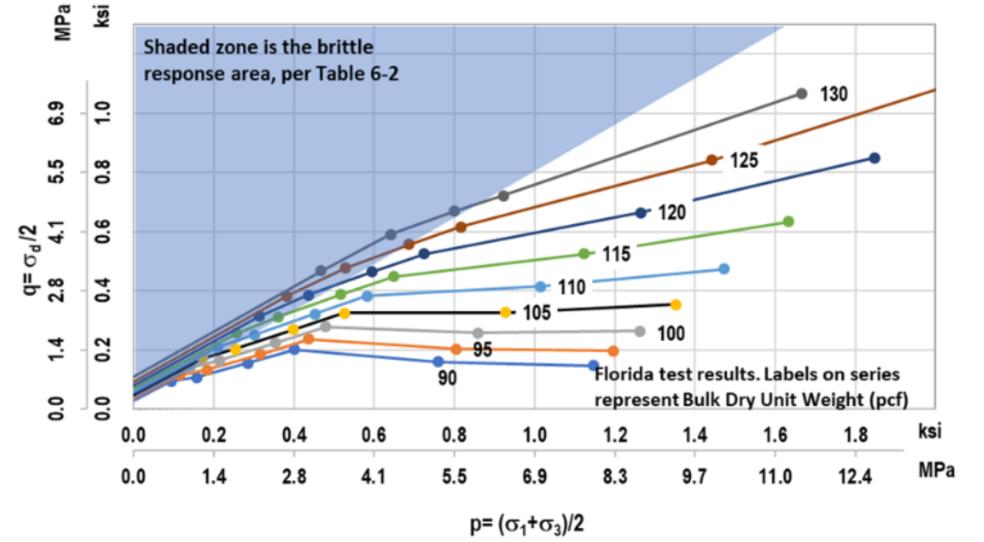
- Investigated the **strength envelope** of several **Florida limestone formations** near the ground surface – function of **dry unit weight** of rock and **formation** (Carbonate).
- Developed **Bearing Capacity Equation**, function of **rock strength** (homogeneous) and **moduli** (layered: rock over sand).

Phase II (FDOT BDV31-977-124):

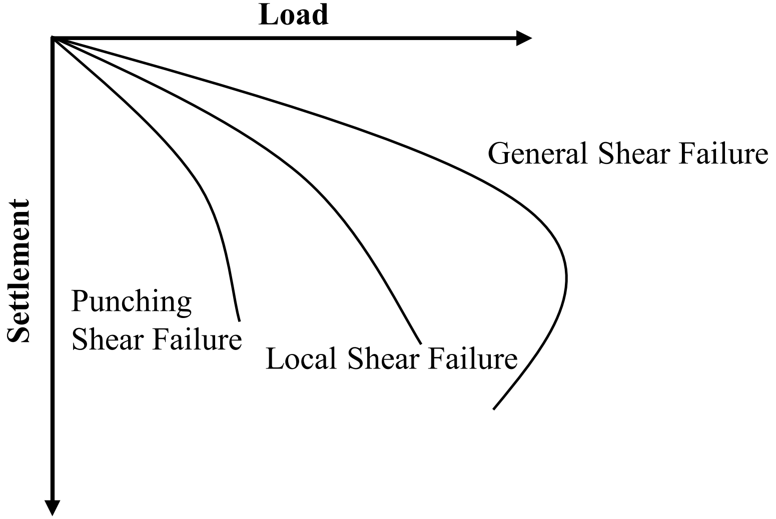
- **3 full scale field shallow foundation load tests** conducted to validate the **Bearing Capacity Equation** and predict the **load-settlement response** at different rock formations and layering (Hand Solution and Numerical Method).

Phase III (Planned):

- Implement the **Winkler Model** (distributed nonlinear springs) including Bearing Capacity and load-settlement for homogeneous and layered Limestone scenarios in Florida for **FB-Multipier**.



Strength Envelope – Miami Formation (FDOT BDV31-977-51)



Load-Settlement Response

Phase II Research Objectives

I. Conduct load test (900 tons) on shallow foundations at three sites having different Florida Limestone formations and layerings (Deliverables 2 to 4) and Validate the New Bearing Capacity Equations derived in FDOT research project BDV31-977-51.
   - Load Test 1: Cemex Site, **Homogeneous Miami Limestone**.
   - Load Test 2: SR-84 Site, **Miami Limestone overlying Medium-Dense Sand Layer**.
   - Load Test 3: Bell Site, **Ocala Limestone overlying weathered Rock and Loose Sand**

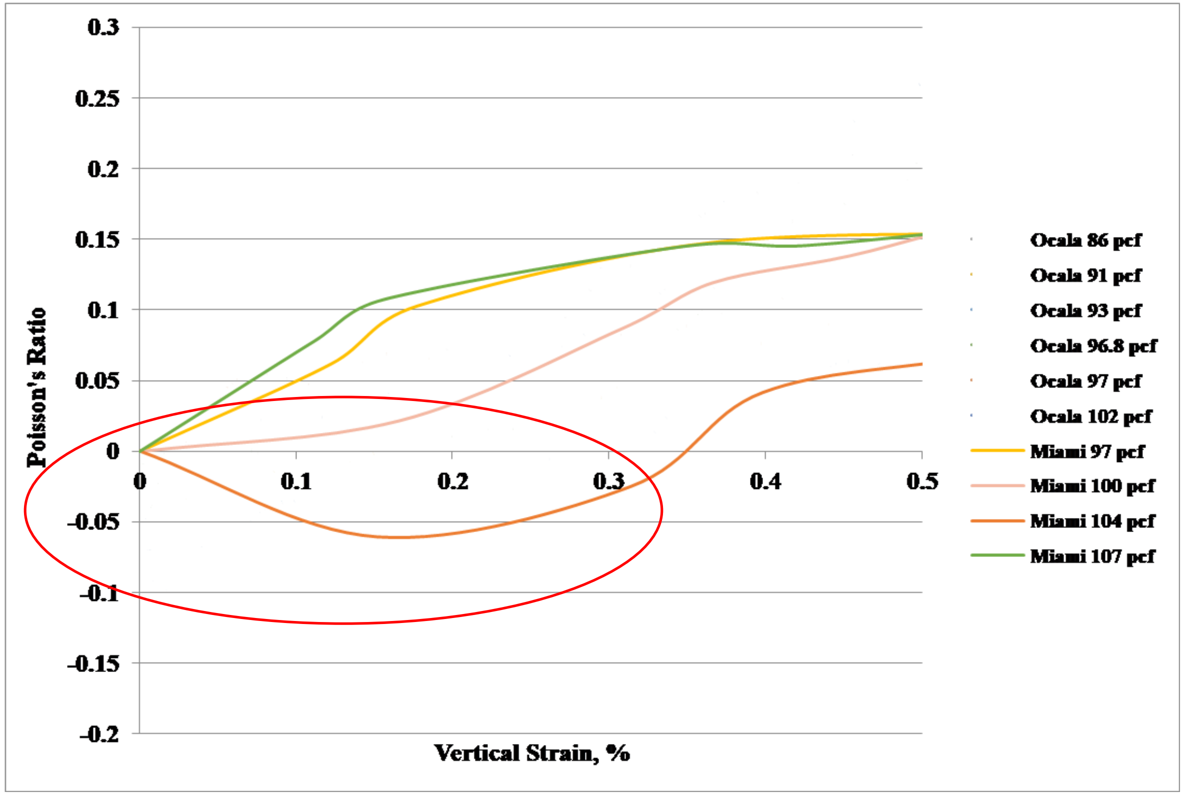
II. Measure and Predict Load versus Settlement for shallow foundation on homogeneous & heterogeneous (rock over sand), scenarios in Florida (Deliverable 6 to 7).
   - Homogeneous – Single Layer: **Fenton & Griffiths Method (2002)**
   - Heterogeneous – Two Layer: **Burmister Method (1958), FEM, FB-Multipier (Winkler Model)**

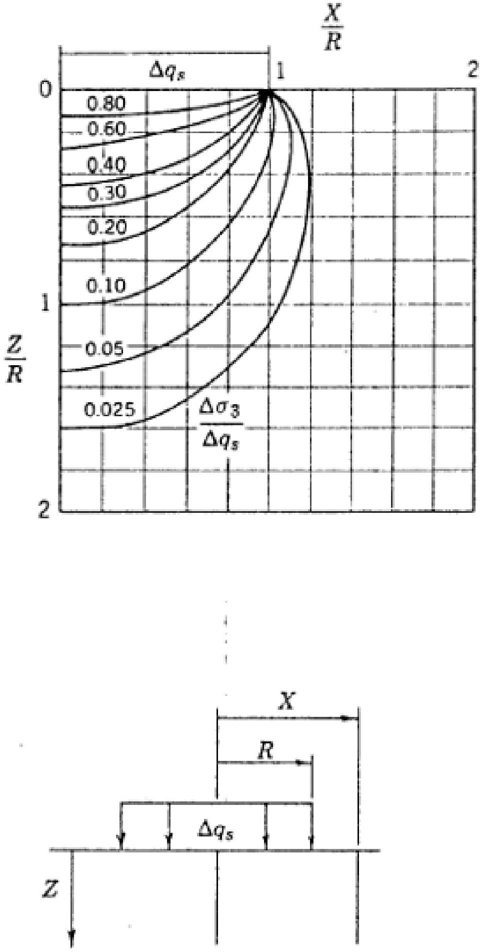
III. For I & II -Assess rock strength, Young's modulus (**Secant - E_{secant}**) and rock unit weight from laboratory tests (**q_u, q_{dt} and triaxial tests**, Phase 1) and in-situ methods – a **newer seismic method** (Deliverable 5, used for characterizing the Dry unit weight).

Overview of Miami Limestone and Ocala Limestone (Phase 1)

Triaxial Poisson's Ratio

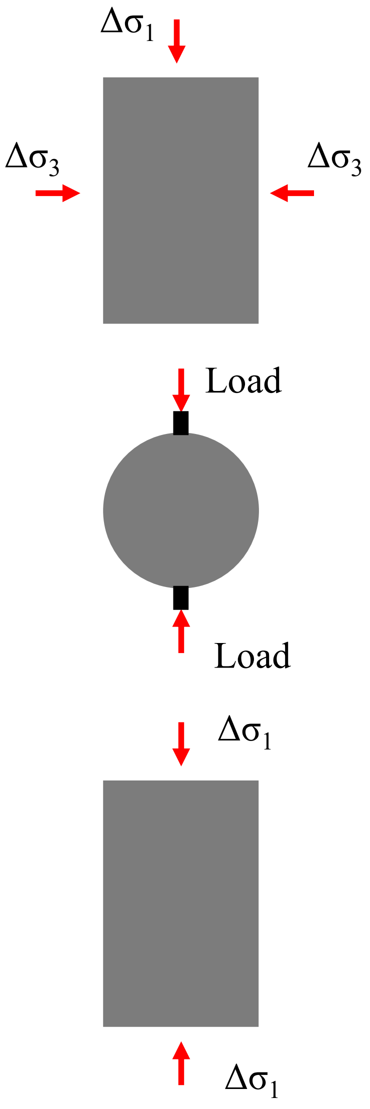
- **Low or Negative Poisson's Ratio at Initial Loading** was found for most dry density rock samples from shallow depth in 50 psi triaxial tests, indicating the rock is crushing (high porosity found in Phase I, 40% versus 15% found in literature).
- **Due to low Poisson's Ratio (0.1), Low Confining Stress ($\Delta\sigma_3$)** was observed from FEM and Associated Stress Path for shallow footing application.
- Boussinesq Solution: Settlement Average Point at 3R/2, assume a working bearing pressure Δq_s = 30 tsf, $\Delta\sigma_3 \approx 0.025 \times 30 \times 13.89 =$ **10.4 psi**

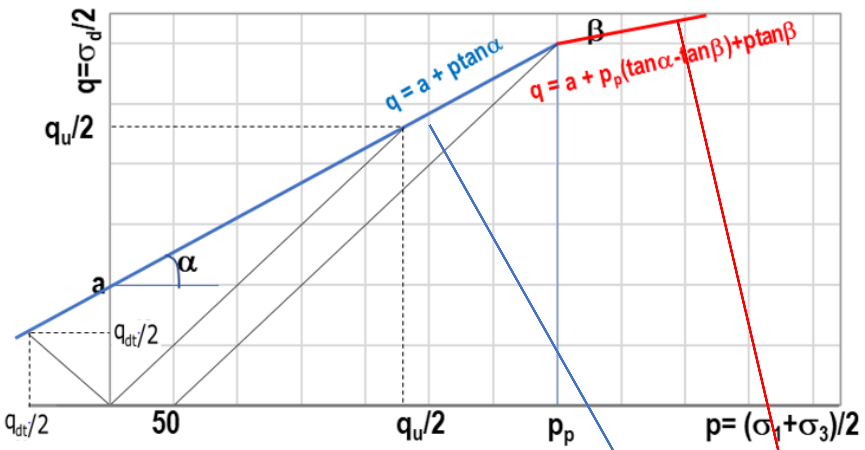




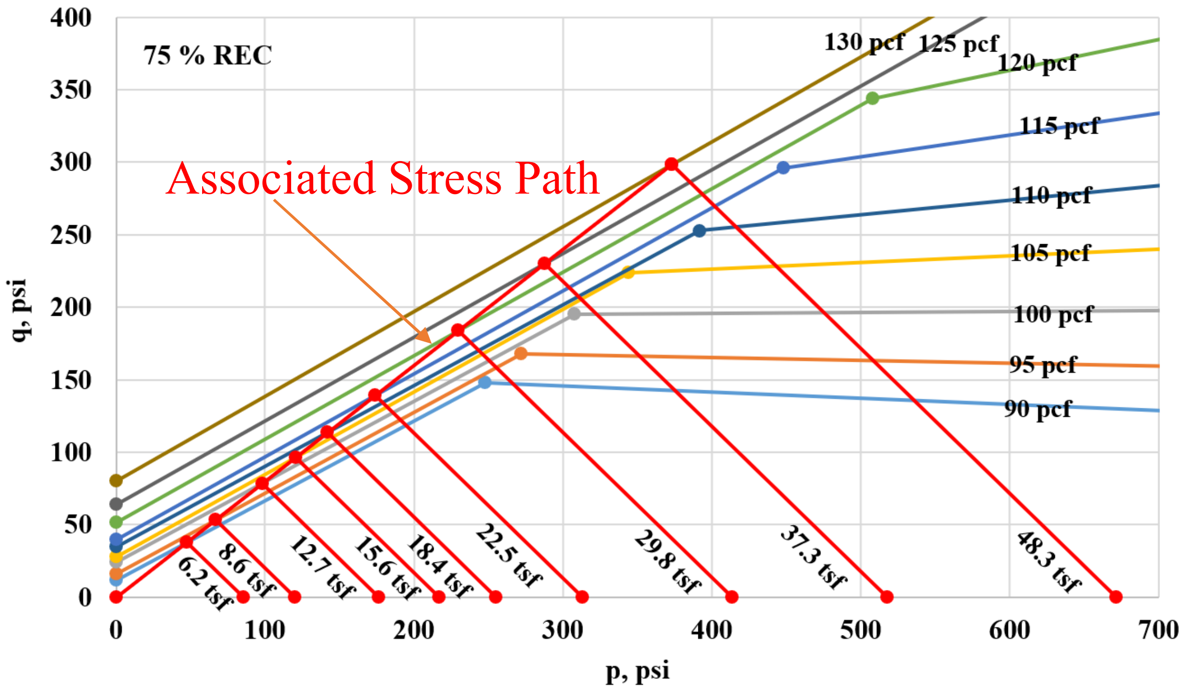
Strength Envelope and Bearing Capacity Equations (Phase 1)

Bi-linear Strength Envelope and Bearing Capacity Equations





Stress path and Strength Envelope



Strength Envelope with 75% Recovery, and Associated Stress path for each dry unit weight, Miami Limestone

$Q_u = \min(Q_{u1}, Q_{u2}) * \xi / N_R$ ξ = Shape factor; N_R = Rock over sand reduction factor

$Q_{u1} = n*c*N_c + q*N_q$

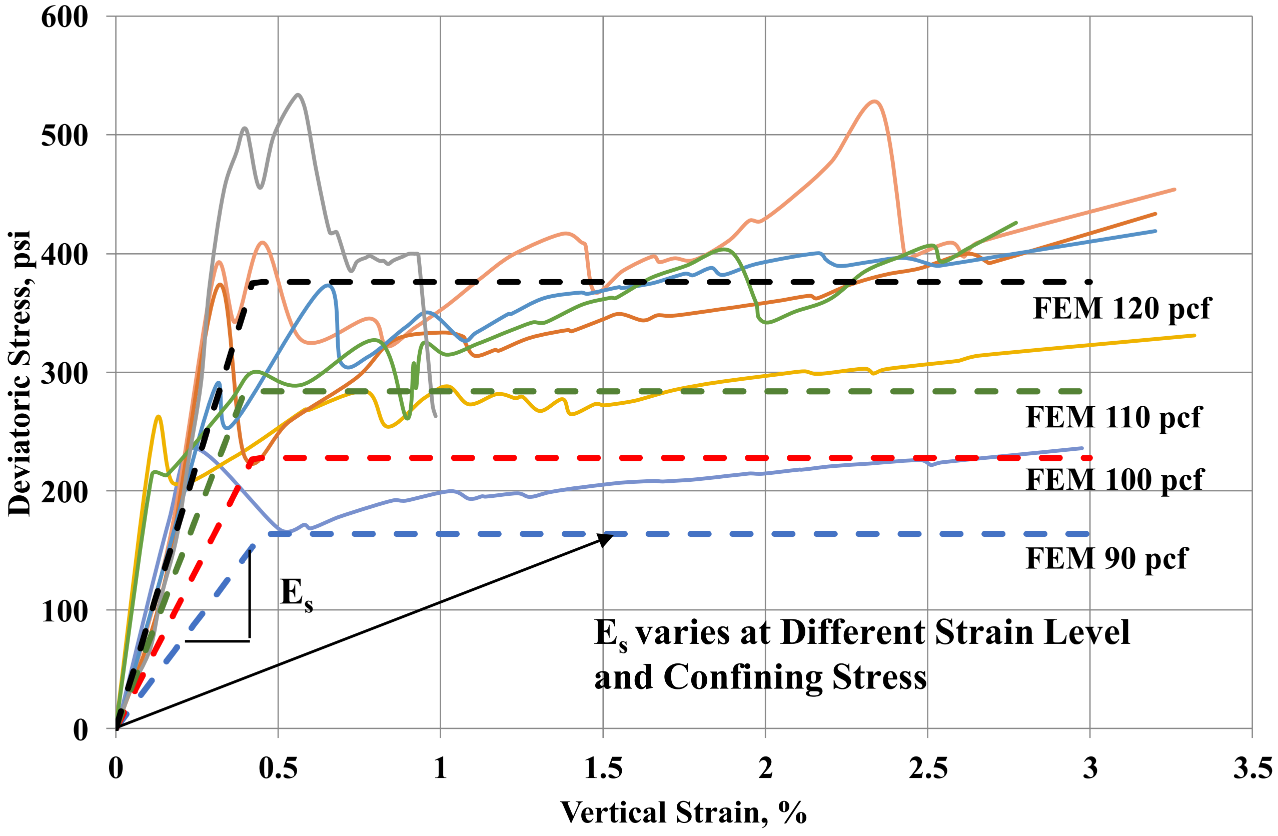
$Q_{u2} = n*[c*N'_c + p_p*N_\gamma] + q*N_q$

Florida Bearing Capacity Equations

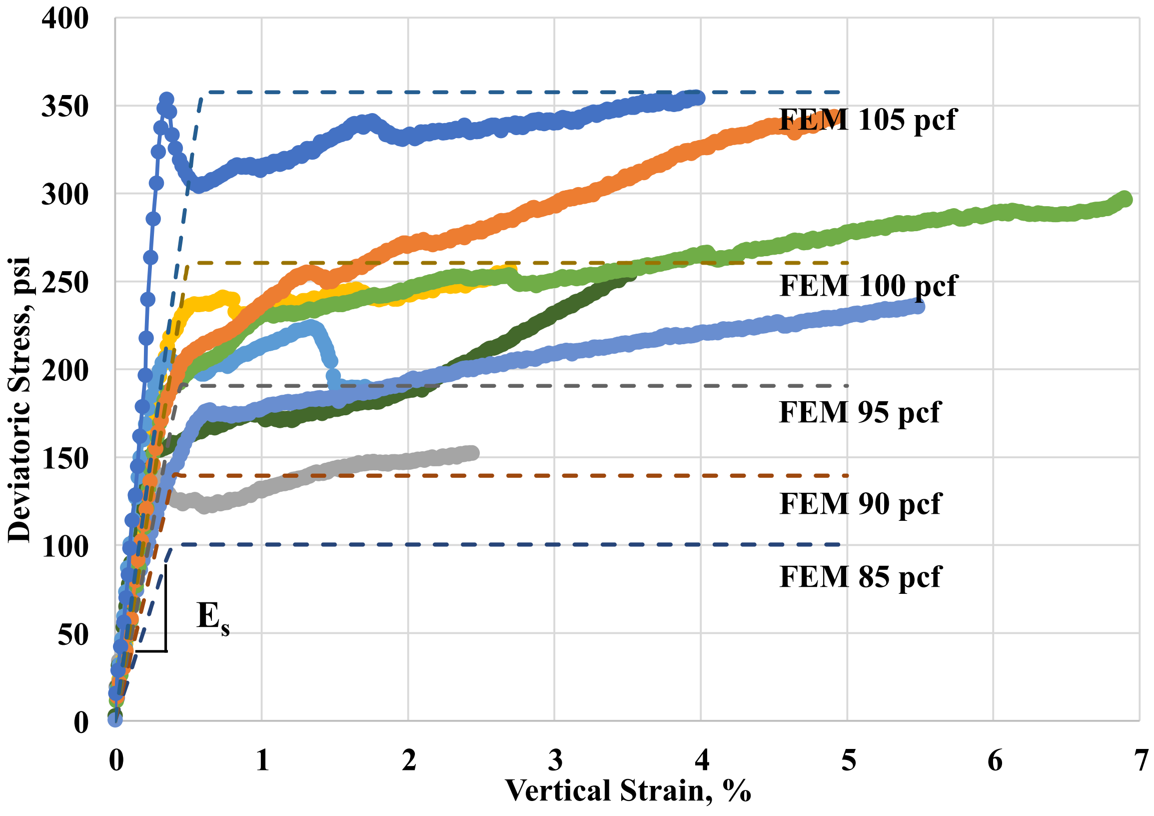
Overview of Miami Limestone and Ocala Limestone (Phase 1)

Stress-Strain Relationship: Secant Modulus

Miami Limestone, **50 psi** triaxial tests, from 91 pcf to 122 pcf

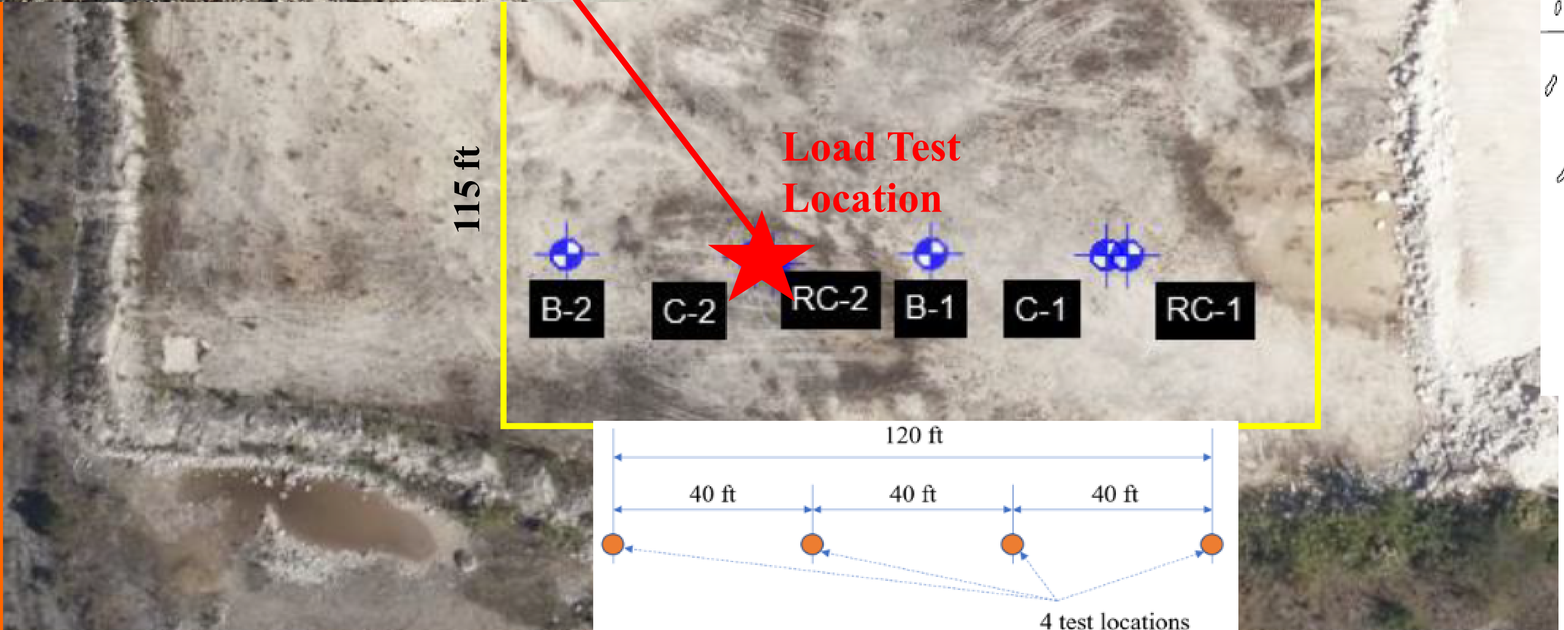


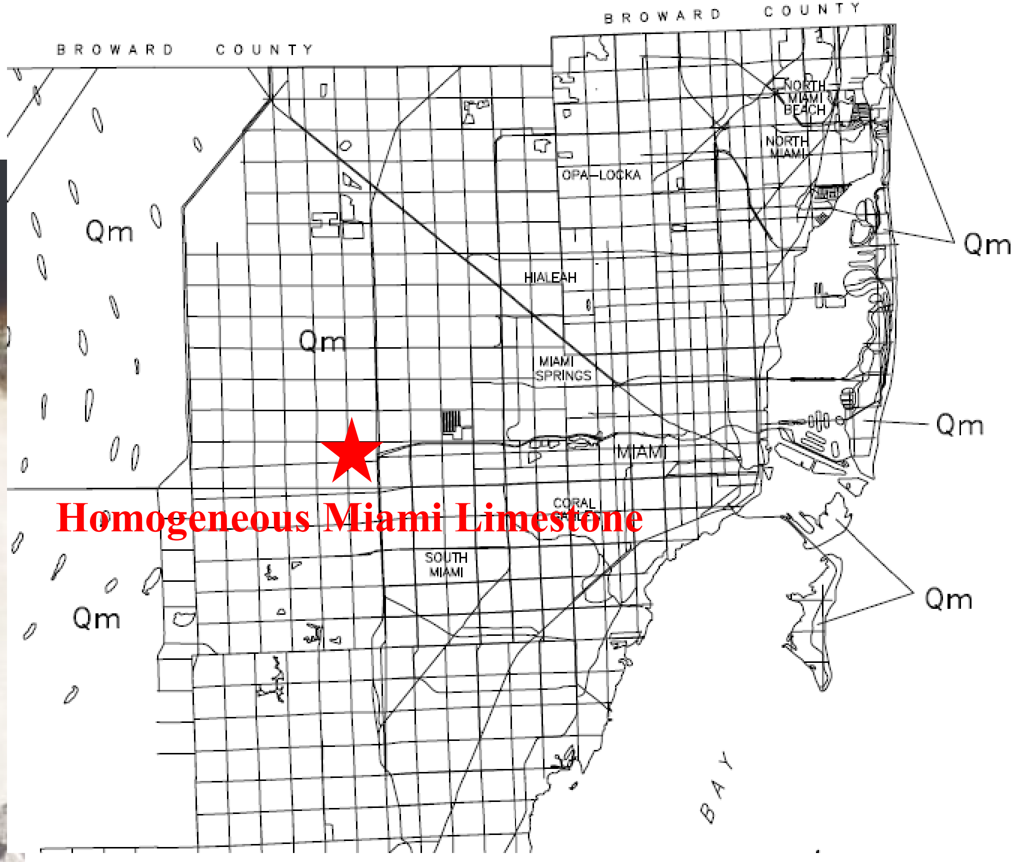
Ocala Limestone, **50 psi** triaxial tests, from 86 pcf to 102 pcf



Load Test 1: Cemex Site, Miami Site Investigation

25°46'59.0"N 80°26'25.6"W

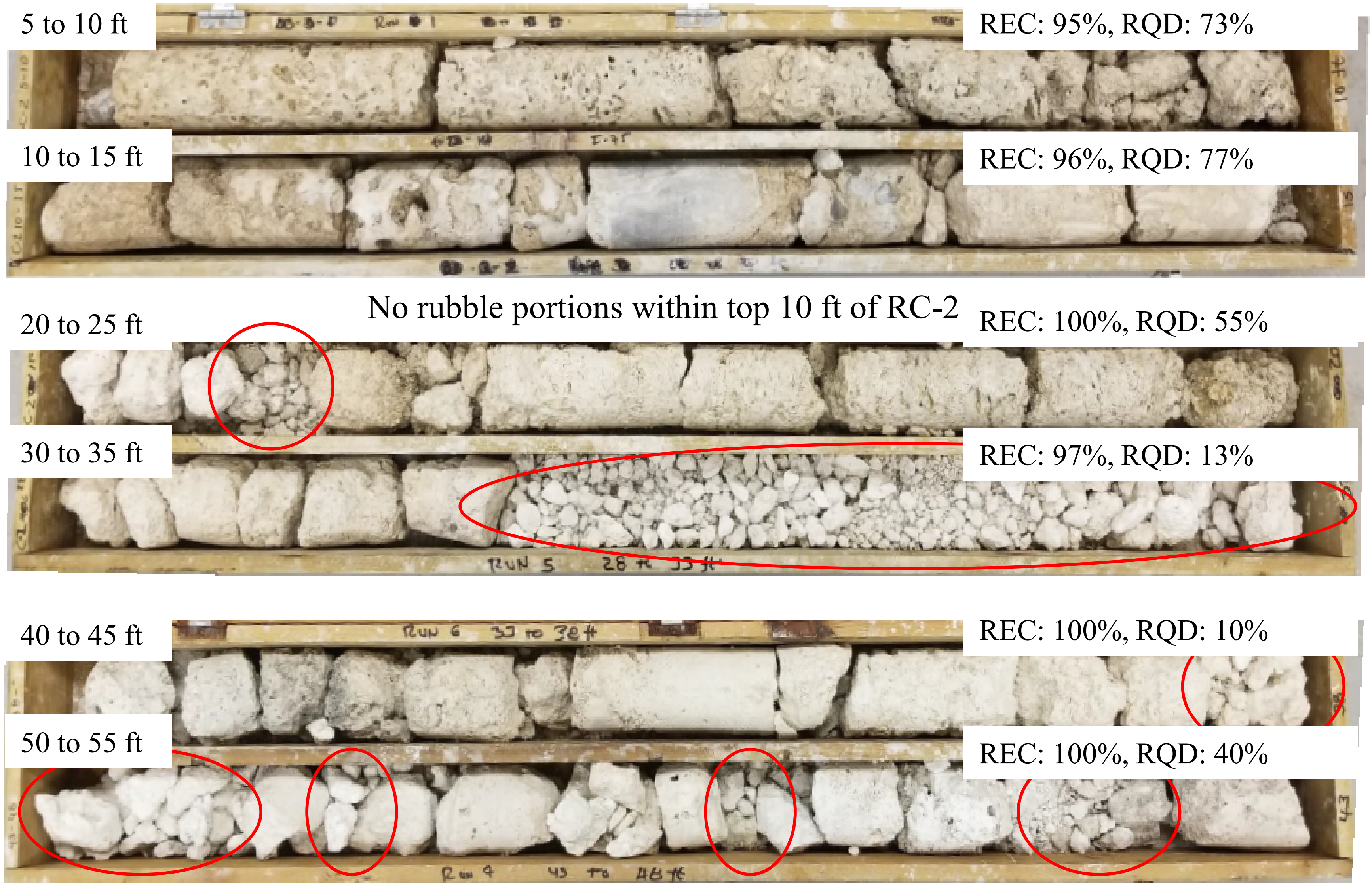




Geologic Map of Dade County, Florida

Load Test 1: Cemex Site, Site Investigation

RC-2 (Terracon)



Adjusted-REC:

95%

96%

94%

32%

86%

63%

Rubble portions can not retain the cylindrical shape for strength tests, it's necessary to count its effect. To evaluate the rock mass strength envelope:

- Weight-adjusted Strength Envelope
- Recovery-adjusted Strength Envelope

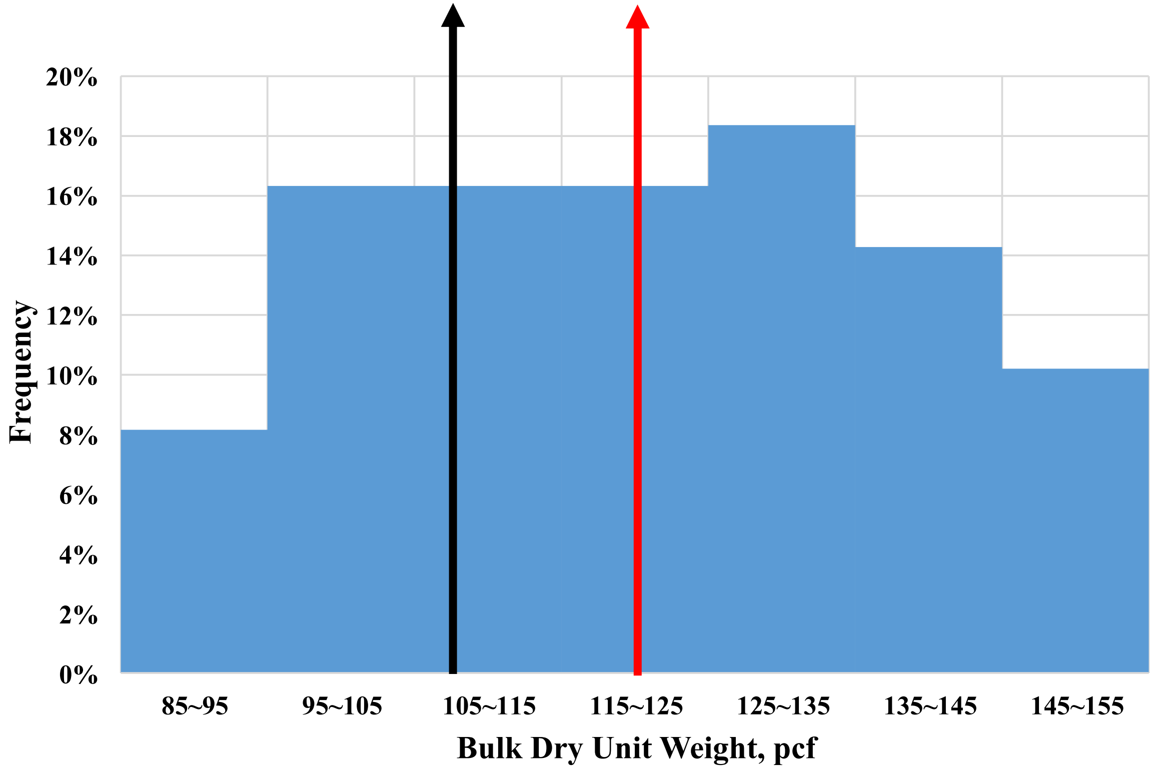
Load Test 1: Cemex Site, Site Investigation

Count the rubble portion as uncoreable material, **New REC: 72%**

Borings	Core Runs	REC, %	RQD, %	New REC, %
RC-1	Run 1	100	36	60
	Run 2	100	60	78
	Run 3	95	20	48
	Run 4	100	60	94
	Run 5	100	11	47
	Run 6	100	47	71
	Mean	99	39	66
RC-2	Run 1	95	73	95
	Run 2	96	77	96
	Run 3	100	55	94
	Run 4	97	13	32
	Run 5	100	10	86
	Run 6	100	40	63
	Mean	98	44	78
Site Mean		99	42	72

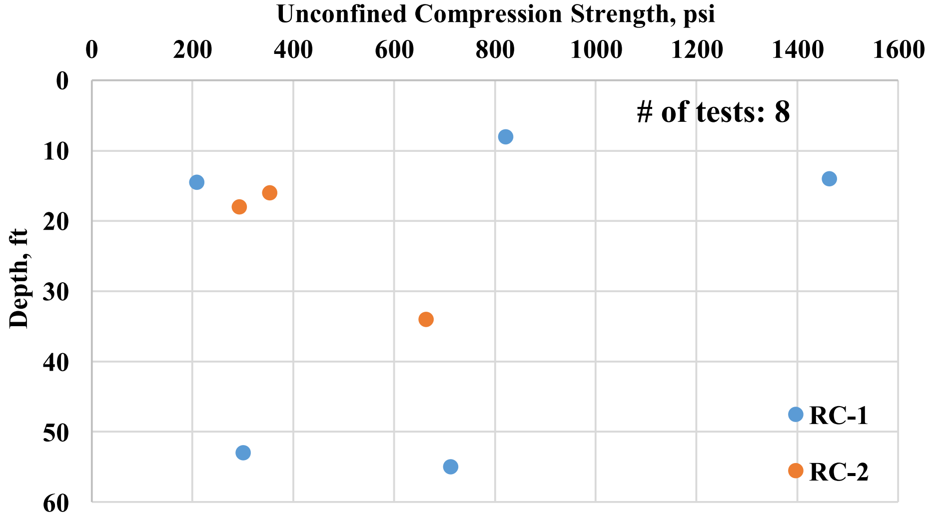
Geomean at Top 2B Depth at RC-2 (Footing Location): 107 pcf

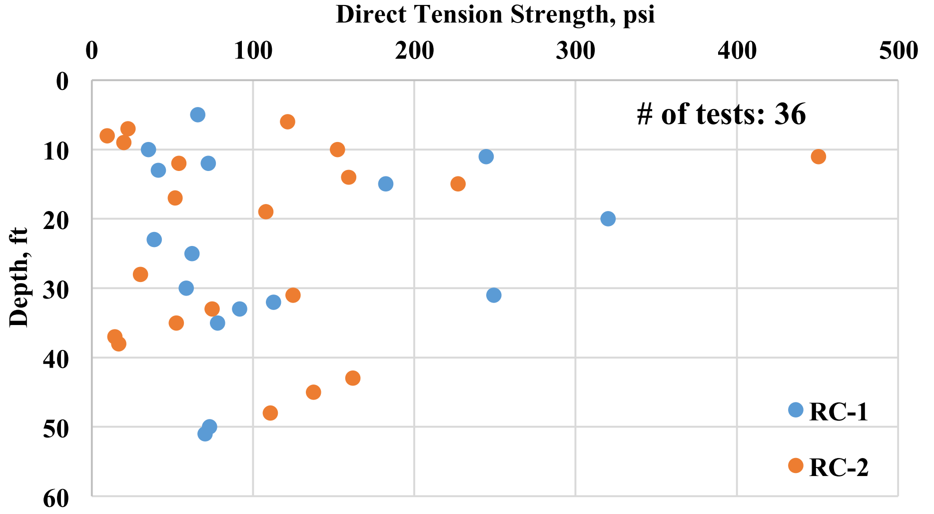
Mean at Cemex Site: 119.3 pcf
Median at Cemex Site: 119.6 pcf



Load Test 1: Cemex Site, Site Investigation

Strength Assessment



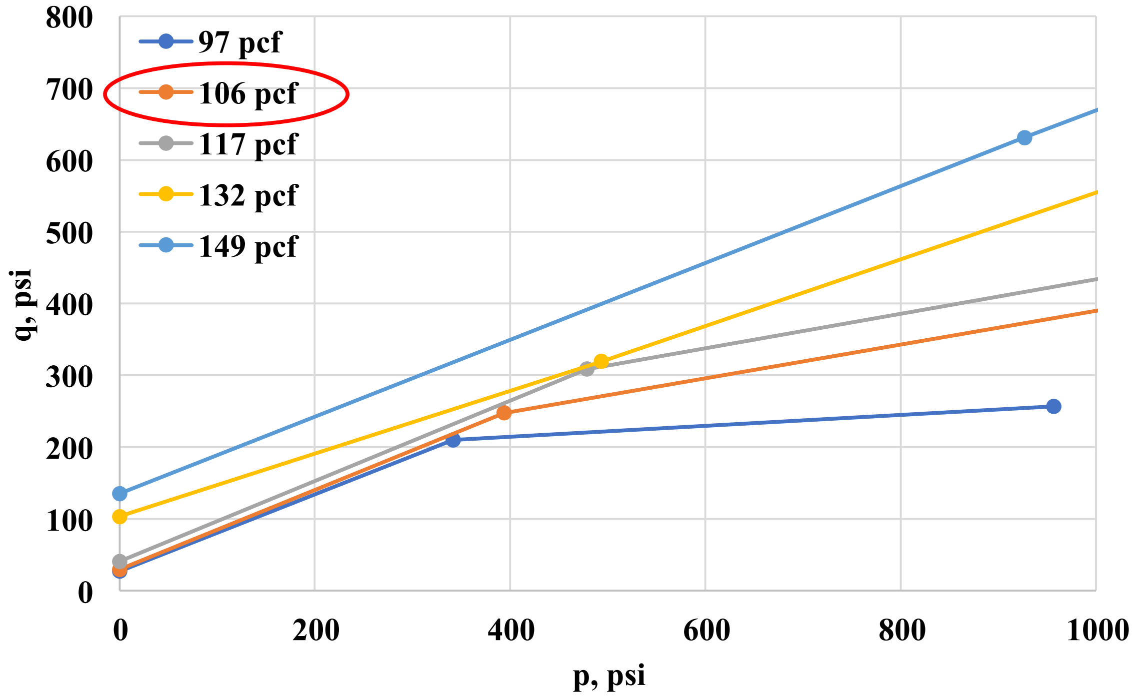


Strength (q_u & q_{dt}) versus Depth for Cemex Site ($q_{dt} = 0.7 \times qt$, Perras, M. A., etc., 2014)

Depth versus Dry Unit Weight and Statistical Summary, RC-2

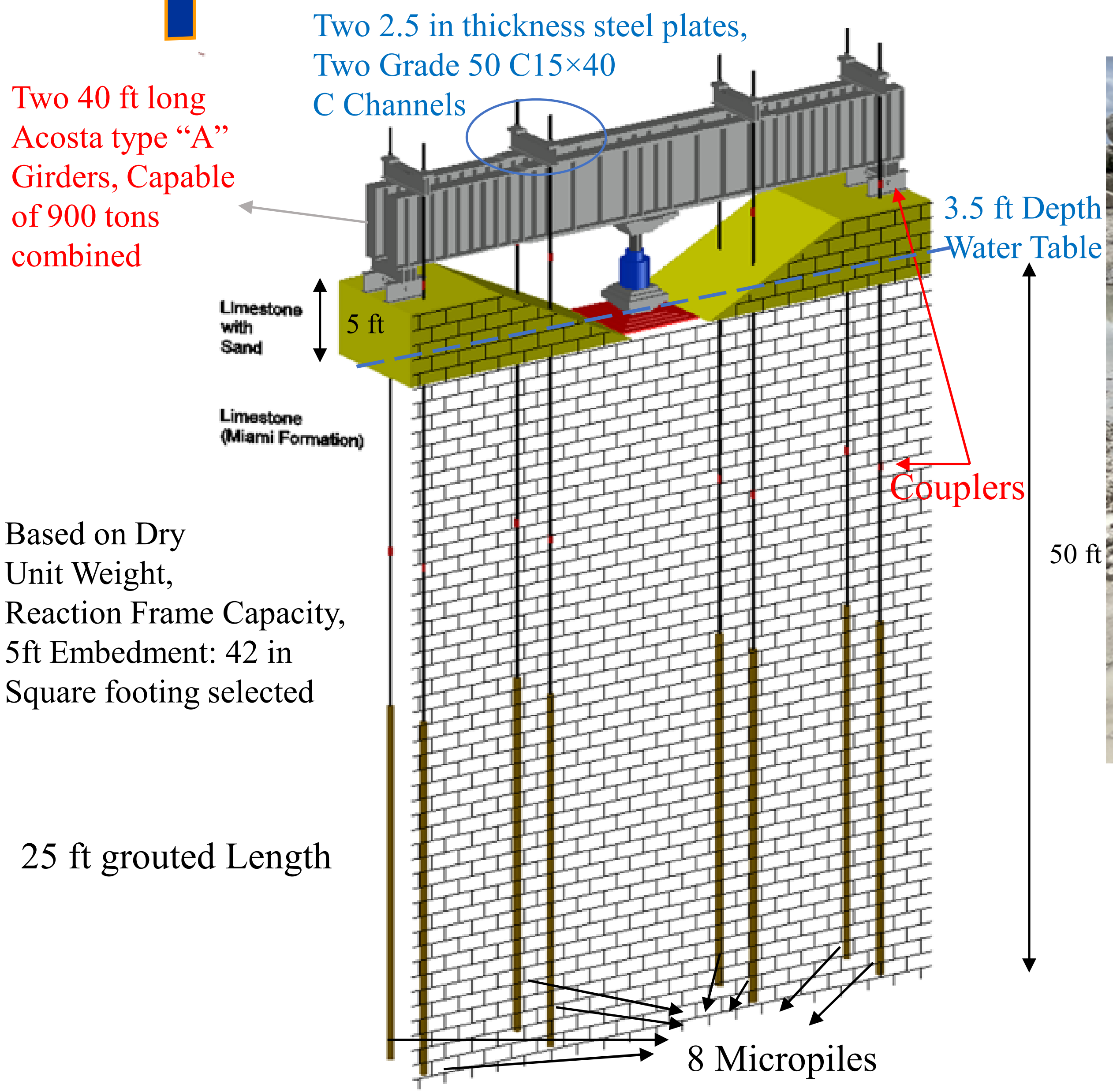
Depth, ft	Dry Unit Weight, pcf
5	106
	100
	97
	88
	94
	85
	134
	145
12 (2B Depth)	126

COUNT	9
MEDIAN	100
MEAN	108
GEOMEAN	107
std	20



Bi-linear Strength Envelope for Cemex Site, with adjusted-REC = 72%

Load Test 1: Cemex Site, Micro-piles Installation and Load Test



Based on Dry
Unit Weight,
Reaction Frame Capacity,
5ft Embedment: 42 in
Square footing selected

Drilling and Micro-piles Installation (H2R)

$$f_s = REC \times \frac{1}{2}\sqrt{q_u \times q_{dt}} = 104 \text{ psi}$$

$$\text{Reaction} = 104 \times 25 \times 12 \times 6 \times 3.14 \times 8 \,/\, 2000 = 2351 \text{ tons}$$

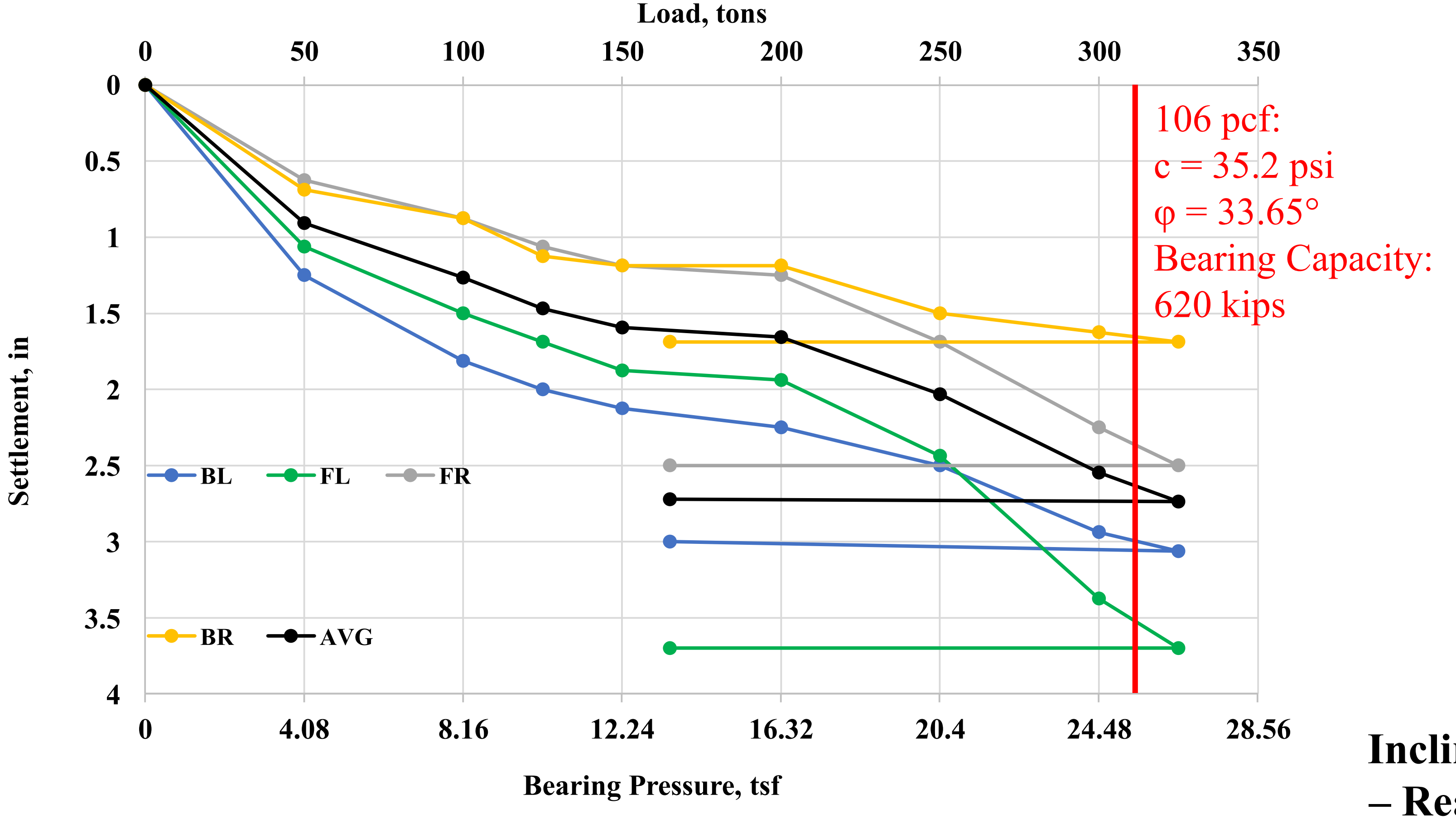
$$\text{Factor of safety} = 2351/900 = 2.6$$

Load Test 1: Cemex Site, Micro-piles Installation and Load Test

Excavation for placement of footing: competent rock at 5 ft depth

Construction for load test: Anchor Installation, Hydraulic Jack & Load Cell, load test.

Load Test 1: Cemex Site, Load Test Results: Bearing Capacity

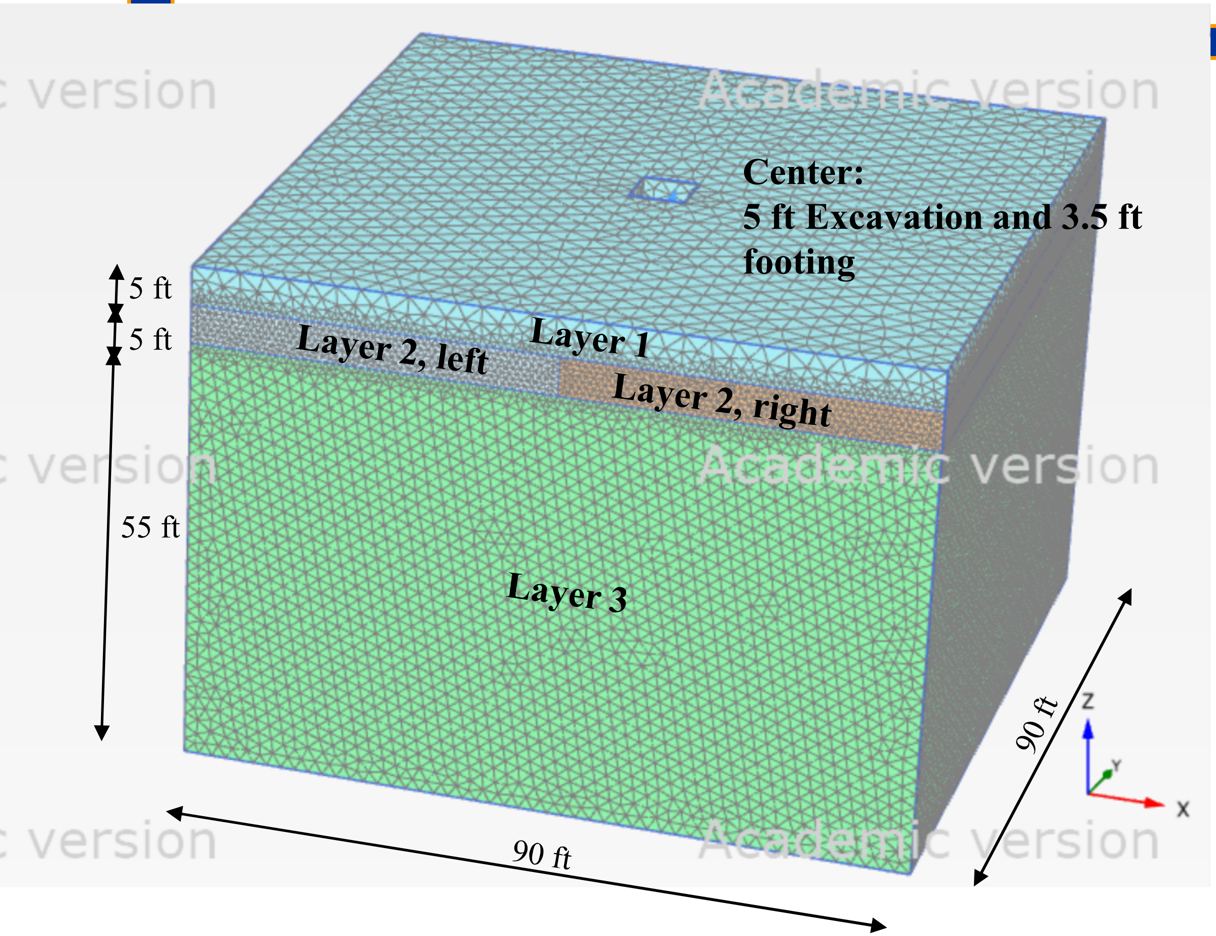


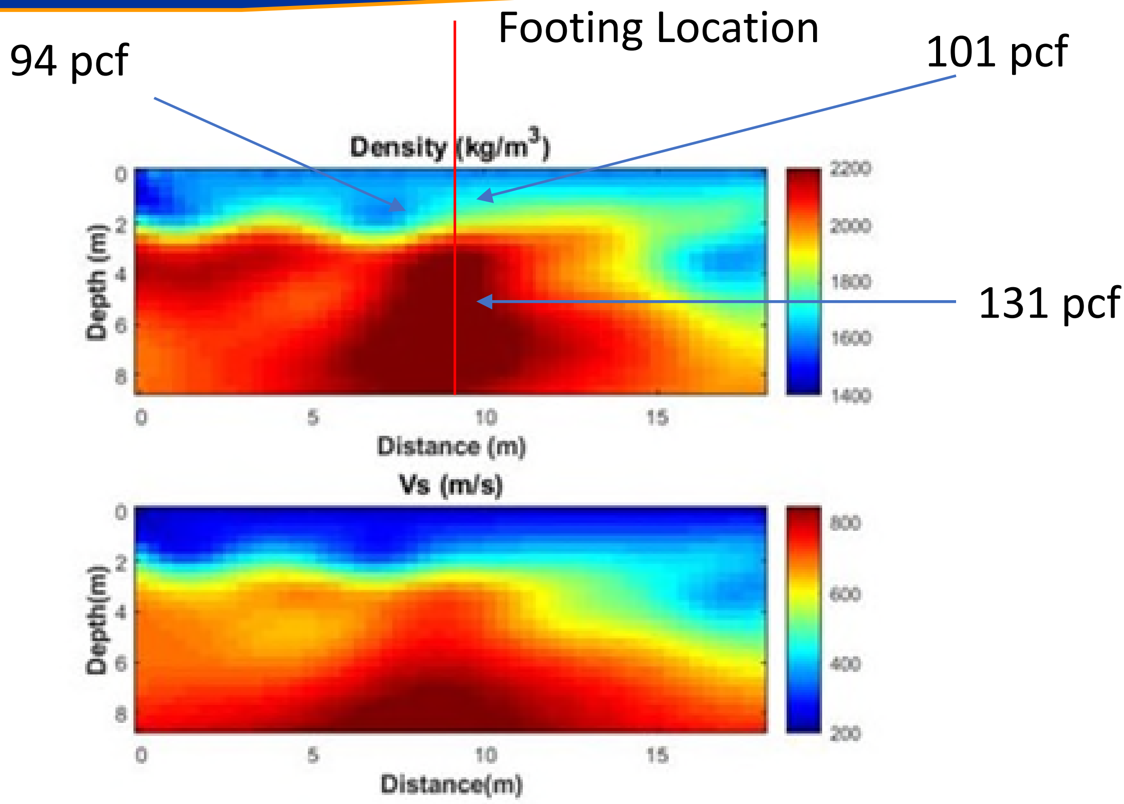
Measured and Predicted: Bearing Capacity for Cemex Site

Settlement Measurement: Tape with Total Station

Inclination of the Loading Jack and Load Cell – Reach Bearing Failure - Load Test ends 2.74 in settlement at 650 kips

Load Test 1: Cemex Site, Seismic Results



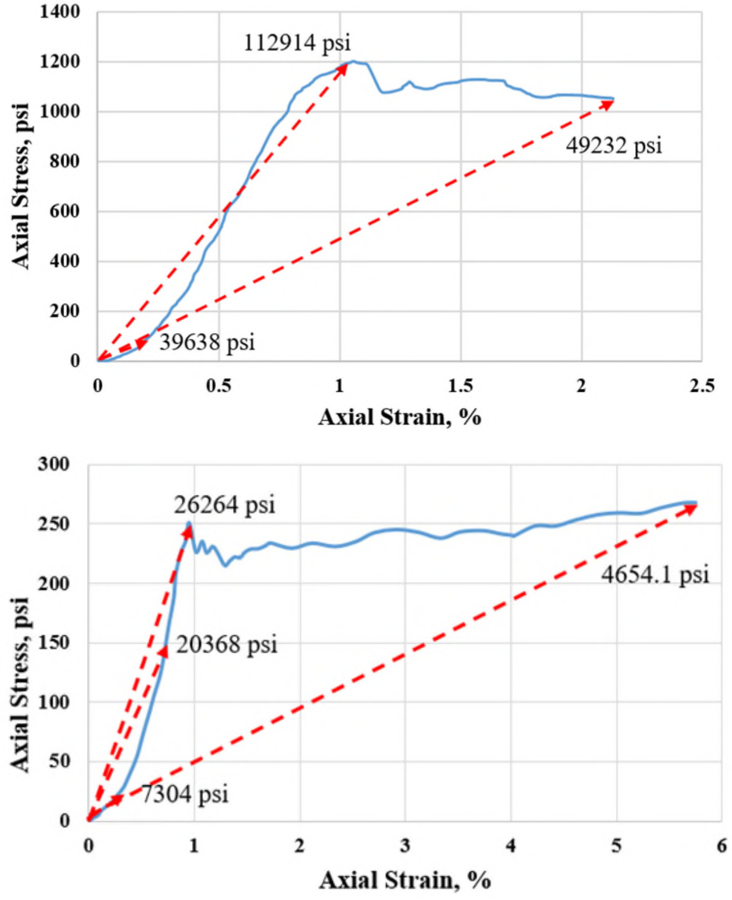


Seismic Results at Cemex Site

Layer	γ_{dt}, pcf	Material Model	c, psi	φ, °	μ	Young's Modulus, psi
1	97	Mohr-Coulomb	32	32.4	0.05	10,514
2, Left	90	Mohr-Coulomb	13.5	31.7	0.01	1,000 ~ 21,636
2, Right	97	Mohr-Coulomb	32	32.4	0.05	1,500 ~ 31,542
3	130	Mohr-Coulomb	103.5	32.5	0.2	50,000

Load Test 1: Cemex Site, Load Test Results: Settlement

FEM – Plaxis 3D – Nonlinear Solution

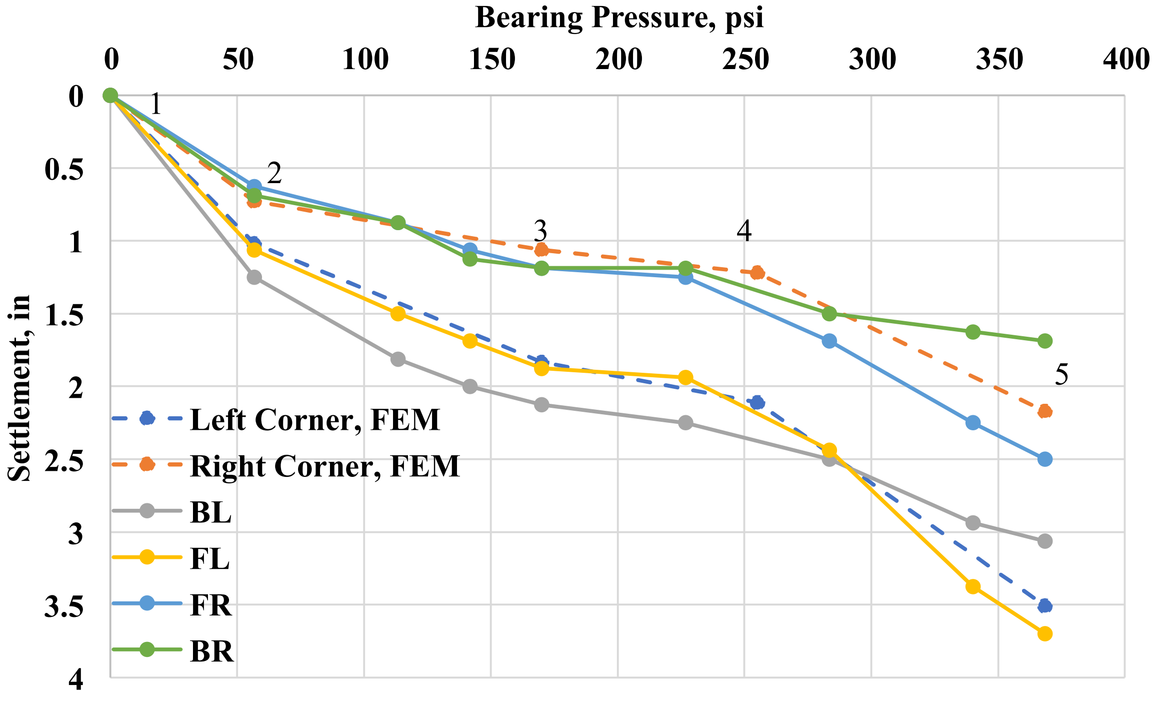


Stress-Strain Curve of Unconfined Compression Test

Secant Moduli vary factor of 3 to 6 between initial loading, failure and yielding.

Stress State, Young's Modulus and Poisson's Ratio of Layer 2 in each Loading Stage

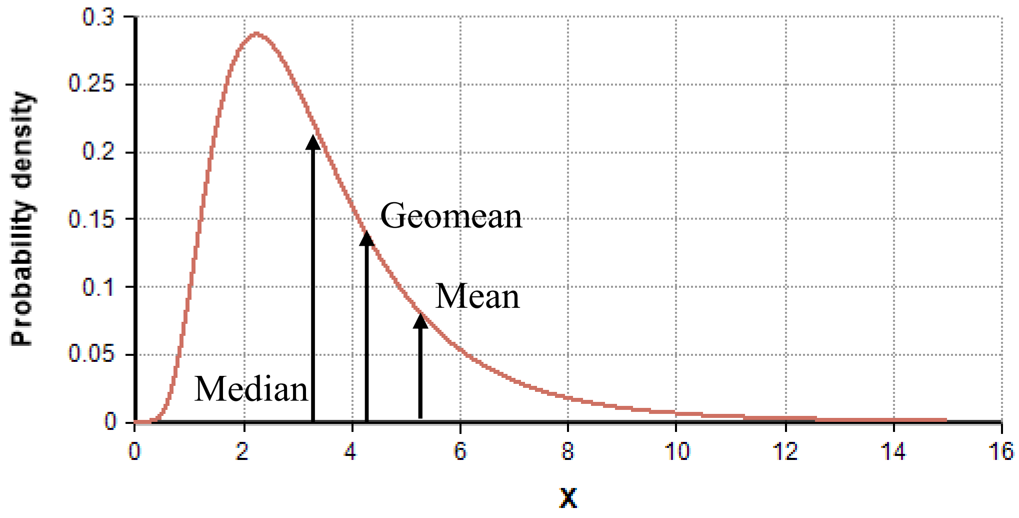
Loading Step	Load, tons	Layer 2							
		Left: 90 pcf				Right: 97 pcf			
		σ1', psi	σ3', psi	Elastic Modulus, psi	Poisson's Ratio	σ1', psi	σ3', psi	Elastic Modulus, psi	Poisson's Ratio
1	0	2.90	2.47	1000	0.01	2.56	2.17	1500	0.05
2	50	19.71	-0.31	1000	0.01	24.74	-0.40	1500	0.05
3	150	71.89	7.26	6500	0.01	105.87	-1.53	9000	0.05
4	225	117.16	21.34	21636	0.01	190.24	22.31	31542	0.05
5	325	195.78	45.79	6500	0.01	263.81	44.55	9000	0.05



Measured and FEM: Load-Settlement Response

Mass Modulus of Heterogenous Rock

Variability of Florida Limestone



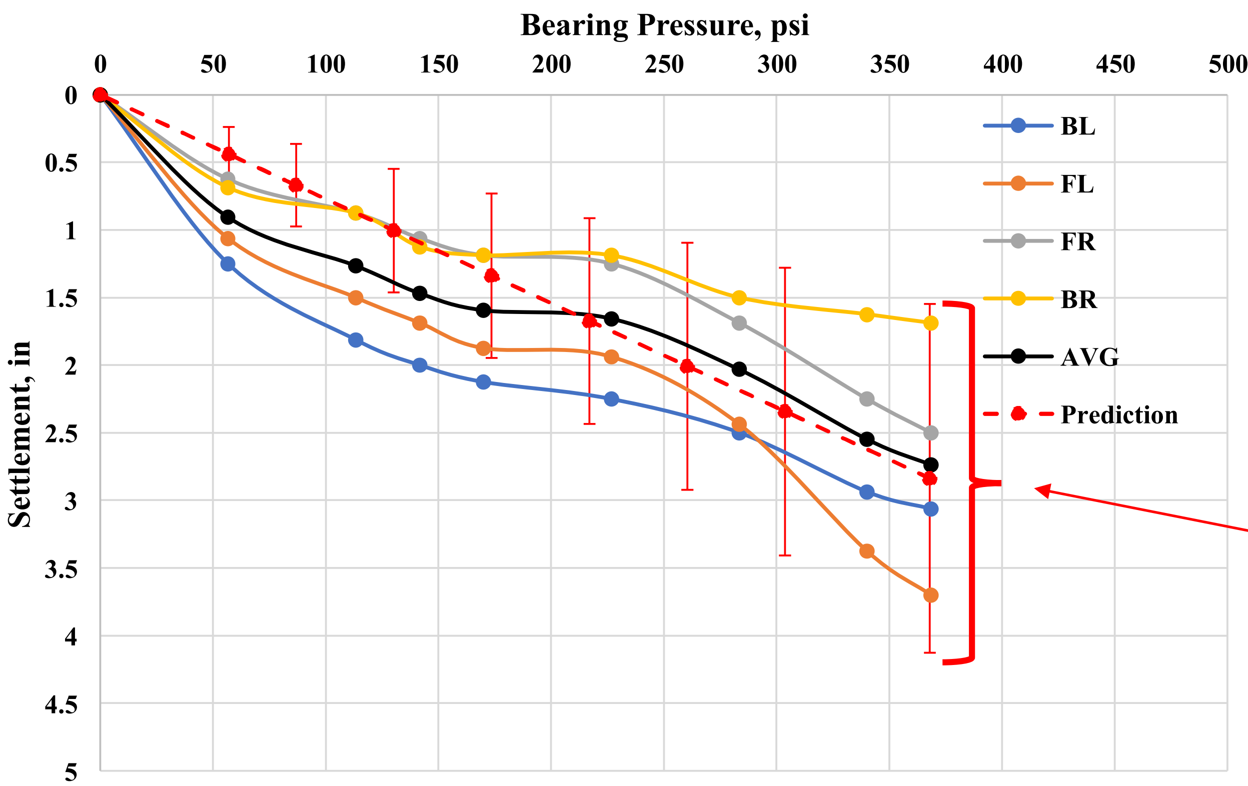
- Strength, Dry Density and Modulii of Florida Limestone is **lognormally distributed** which may be characterized with Mean, Median and Geometric mean;

- Instead of the mean value, Fenton & Griffiths (2002) suggest the **Geometric mean** modulus be used for the heterogenous mass modulus when estimating the footings mean settlement;

- For heterogeneous case of rock over sand with localized failure (right – punching shear), **median** in the footprint is suggested

Load Test 1: Cemex Site, Load Test Results: Predicted Hand Settlement

Probability Measure of a Single-Footing Deformation, Fenton & Griffiths Method (2002) – Linear Solution



Measured and Fenton & Griffiths method: Load-Settlement Response

Settlement at BC (q_s):

$$\rho = \Delta q_s \frac{W_f}{E_g} 1.12(1 - \mu^2)$$

Where,

μ = Poisson's Ratio

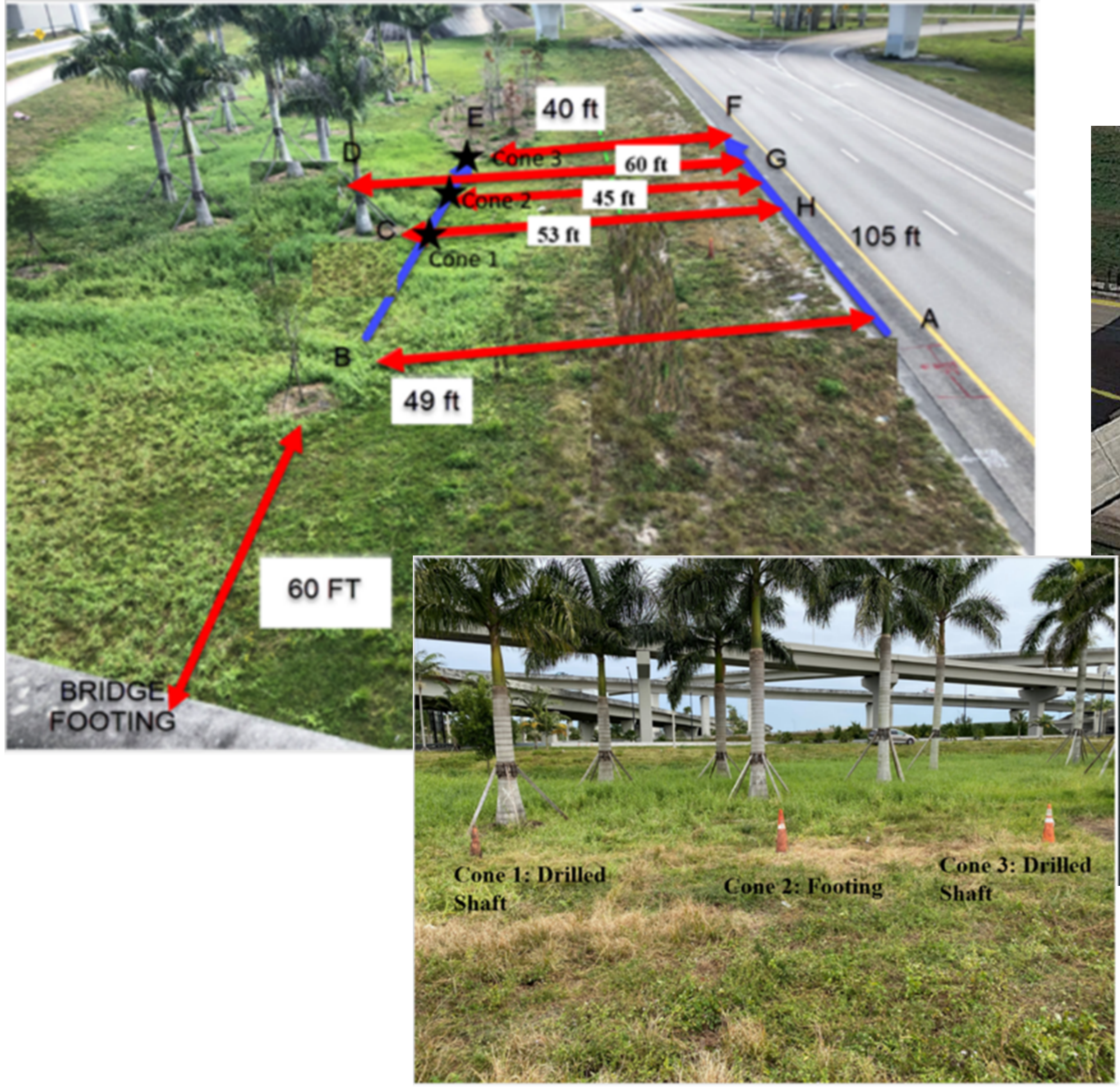
Δq_s = Bearing Pressure

Using a **Geometric Secant Modulus** = 11,104 psi, Secant Modulus Standard Deviation = 11,659 psi based on CV = 1.05,

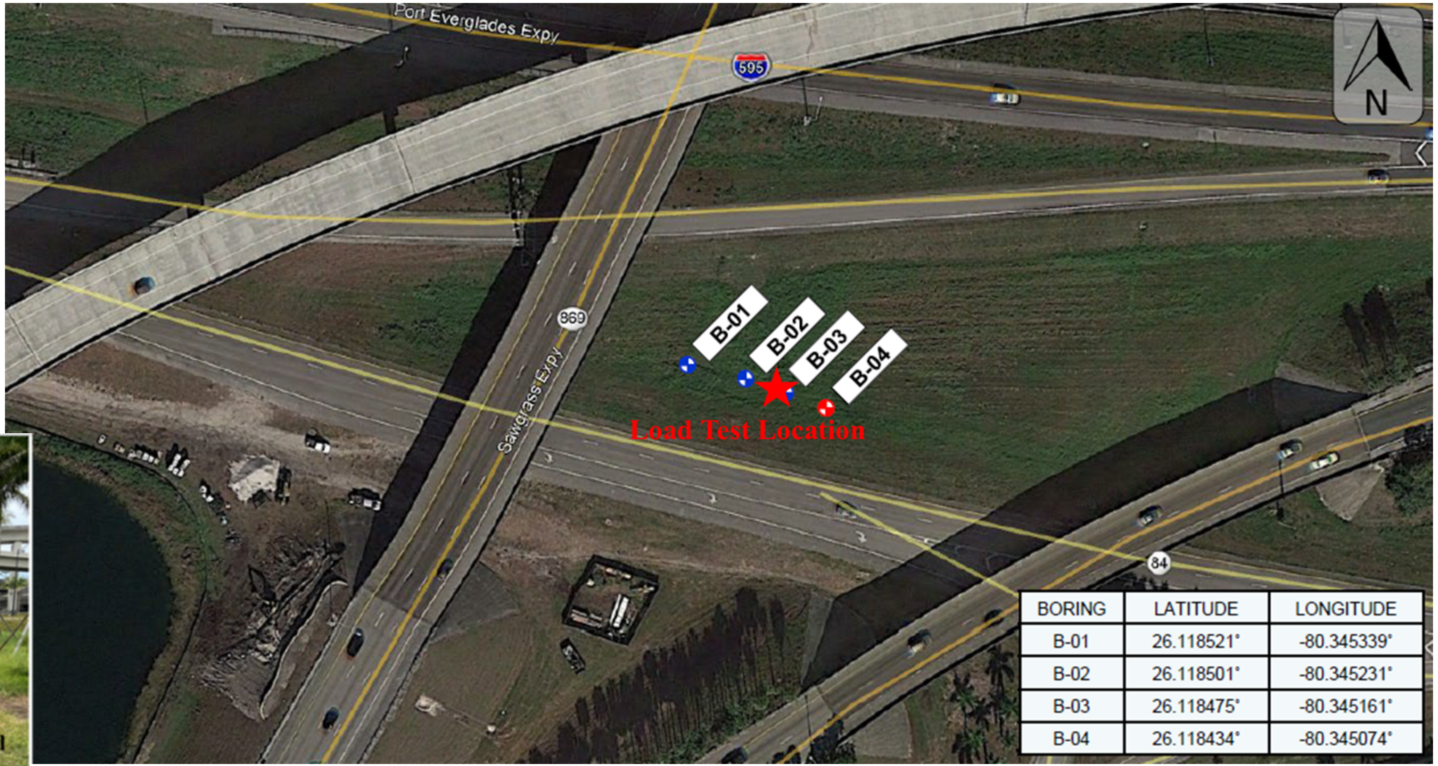
A mean settlement of 2.84 in and a differential settlement: 2.58 in (Fenton & Griffiths Method)

vs Measured mean settlement = 2.74 in and differential settlement = 2.01 in

Load Test 2: SR 84 Site, Site Investigation



Footing & Drilled Shaft Location



BORING	LATITUDE	LONGITUDE
B-01	26.118521°	-80.345339°
B-02	26.118501°	-80.345231°
B-03	26.118475°	-80.345161°
B-04	26.118434°	-80.345074°

Load Test Location: B3

Load Test 2: SR 84 Site, Site Investigation (PSI)

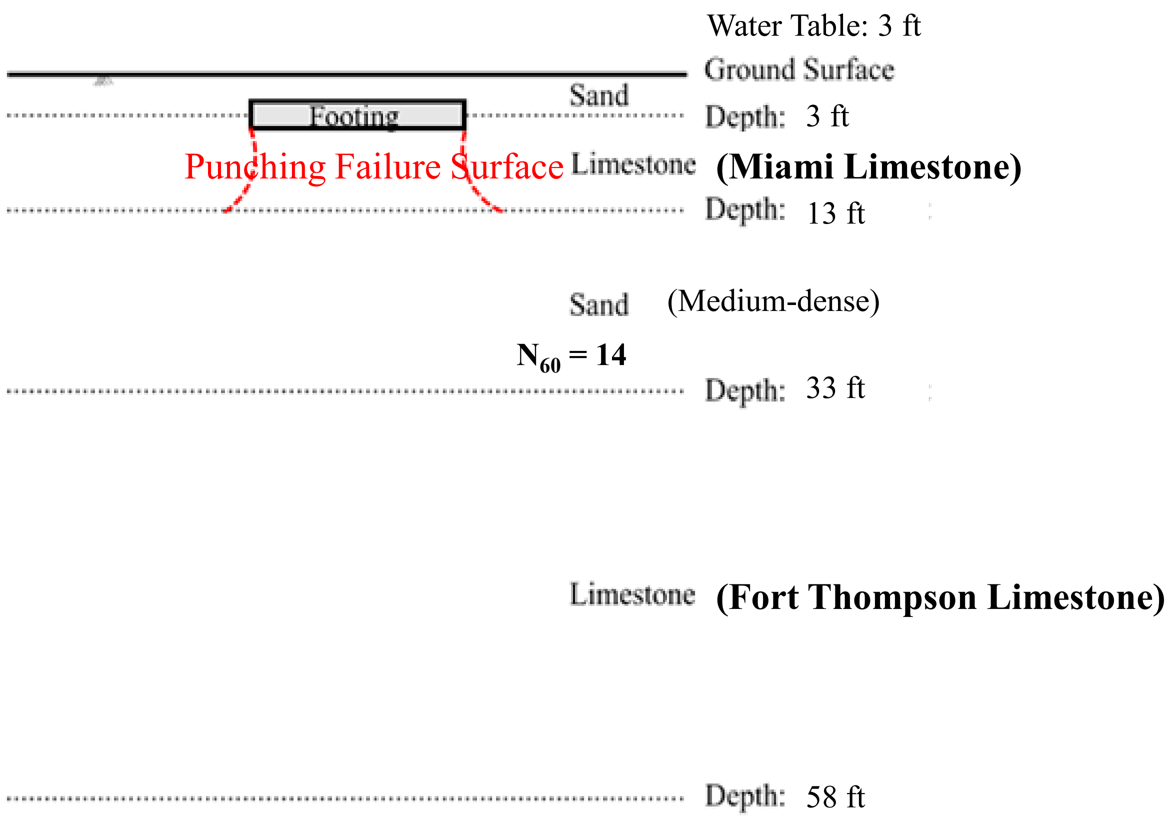
Similar to Cemex Site, the rubble portion was ignored and the Recovery was adjusted: **78% for Miami Limestone & 70% for Fort Thompson Limestone**

Load Test 2: SR 84 Site, Site Investigation

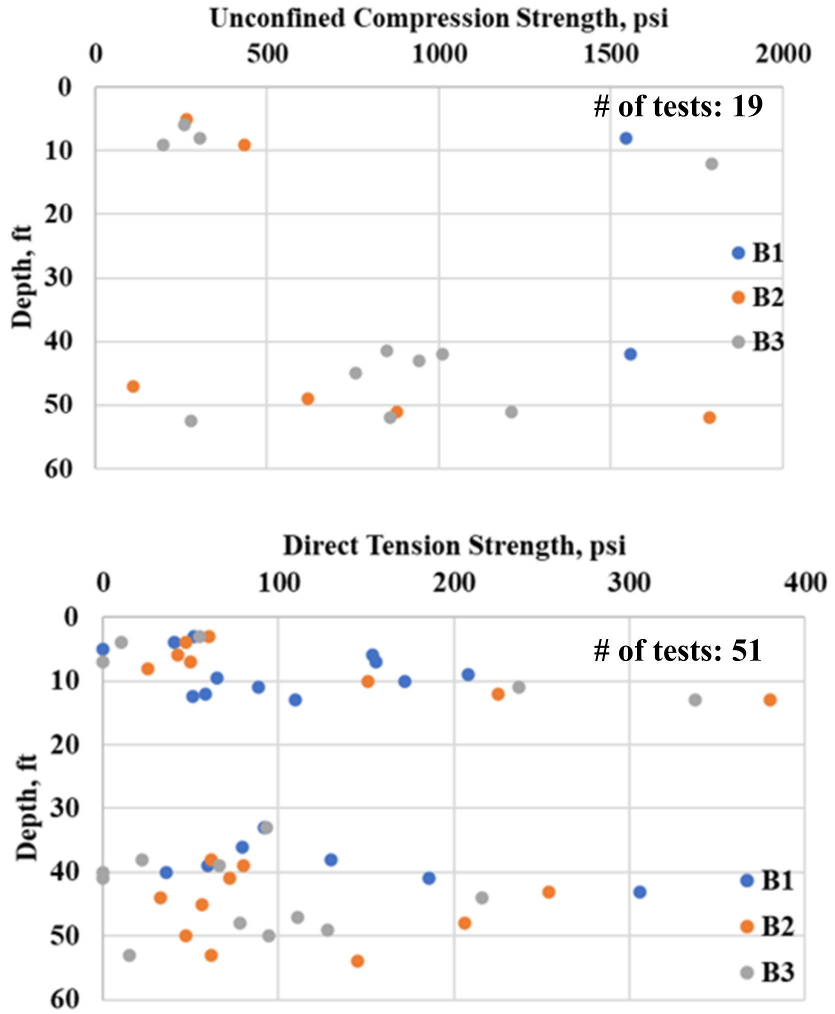
Strength Assessment and Spatial Variability Evaluation

For Miami Limestone layer (3' to 13'), CV = 1.06, Correlation Length: 3 ft

SR 84 Site



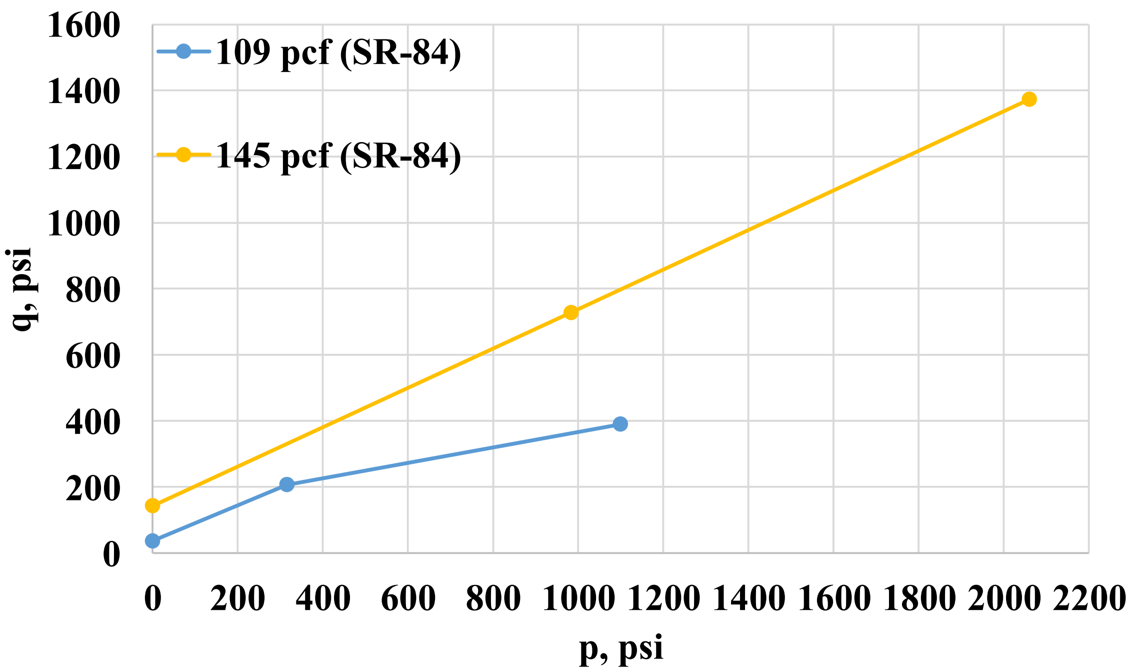
Subsurface layering based on rock coring and SPT



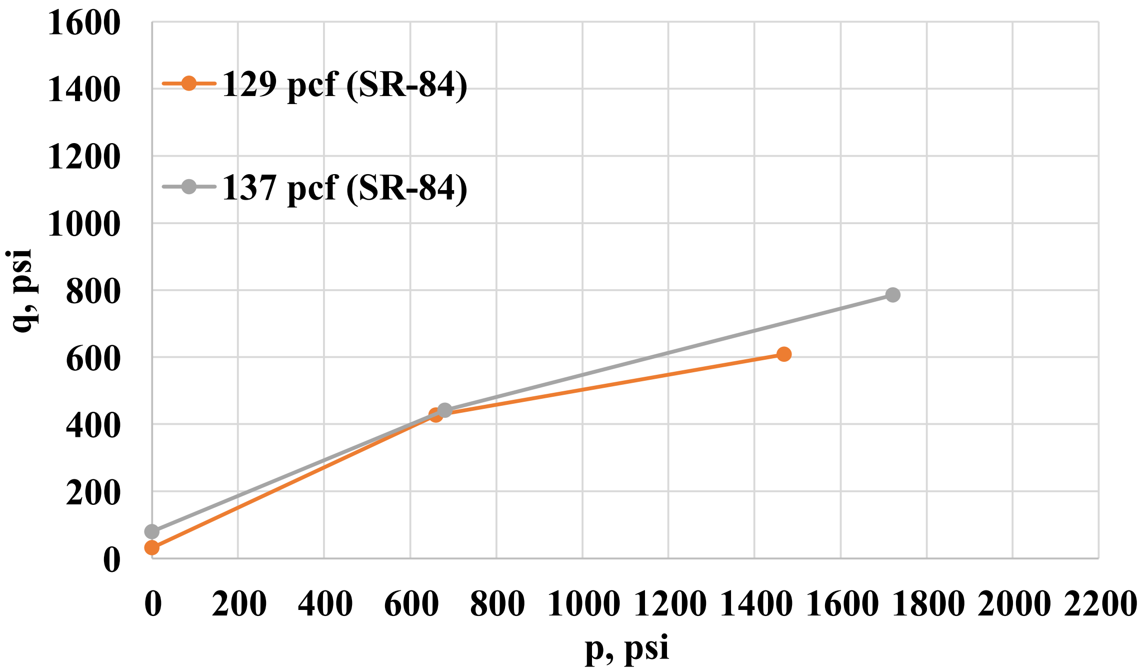
Strength (q_u & q_{dt}) versus Depth for SR 84 Site ($q_{dt} = 0.7 \times qt$, Perras, M. A., etc., 2014)

Load Test 2: SR 84 Site, Site Investigation

Strength Envelopes



Strength Envelope for Miami Limestone at SR 84 Site



Strength Envelope for Fort Thompson Limestone at SR 84 Site

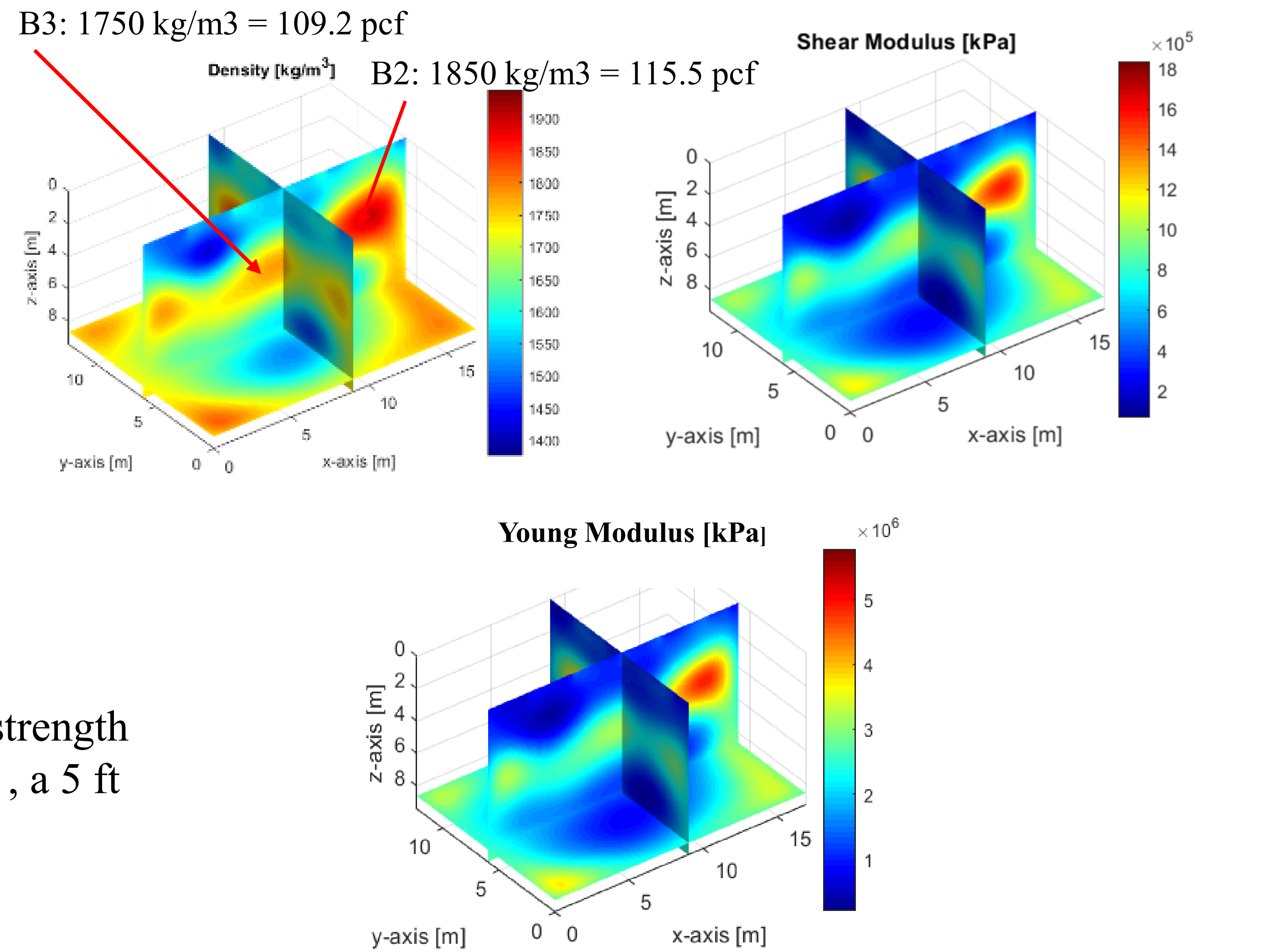
Load Test 2: SR 84 Site Properties & Seismic Shear

Footing Location: B-3

Dry Unit Weight Summary

Boring Number	B-1	B-2	B-3
Count	13	10	10
Median, pcf	127	114	110
Mean, pcf	126	122	118
Geomean, pcf	125	121	117
Std, pcf	15	15	16
Recovery (neglecting rubble portion), %	78	75	82
Competent Fort Thompson Limestone to provide reaction (33 to 55 ft depth)	No	Yes	Yes

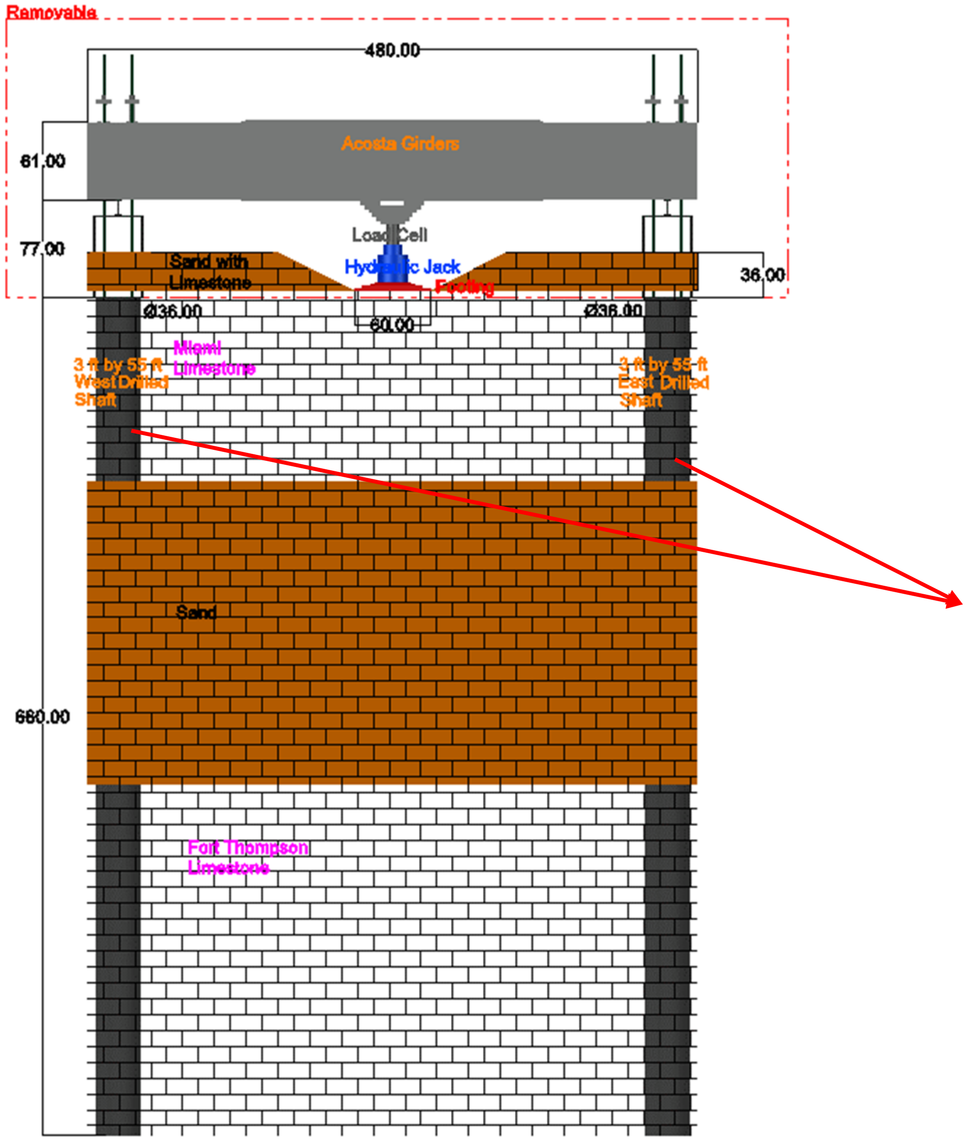
Using the Bearing Capacity equations with strength from B-3 (unit weight 110 pcf) Q_u = 335 psi , a 5 ft x 6 ft rectangular footing was selected



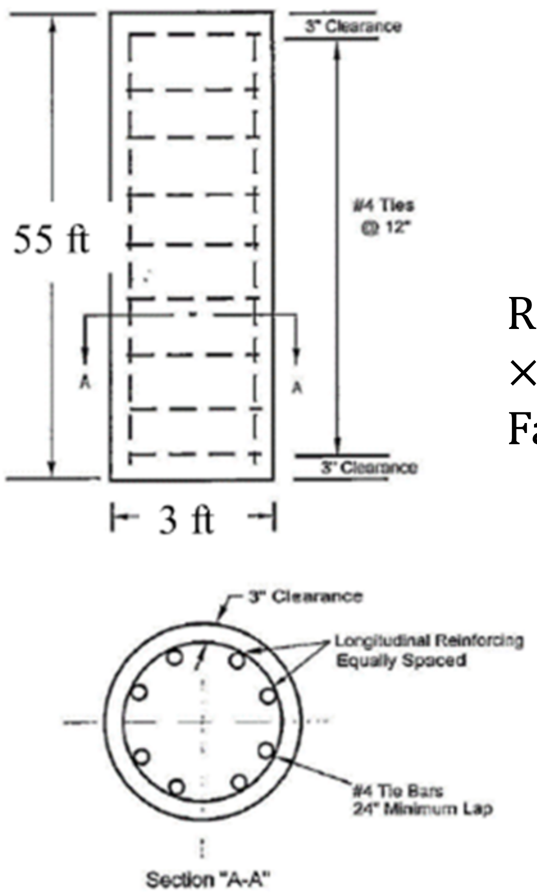
Distribution of Density, Shear Modulus, and the Young's Modulus for the SR-84 Site (Deliverable 5)

Load Test 2: SR 84 Site, Drilled Shaft Installation and Load Test

It was decided to use the drilled shaft to provide the reaction force based on the budget, time and available quotes



Schematic of Load Test at SR-84 Site



Design of the drilled shaft at SR-84 Site

$$f_s = REC \times \frac{1}{2}\sqrt{q_u \times q_{dt}} = 98\ psi$$

Reaction = 98 × 25 × 12 × 36 × 3.14 × 2 / 2000 = 3323 tons

Factor of safety = 3323/900 = 3.7

Load Test 2: SR 84 Site, Drilled Shaft Installation and Load Test

Fasting Setting Concrete for footing placement

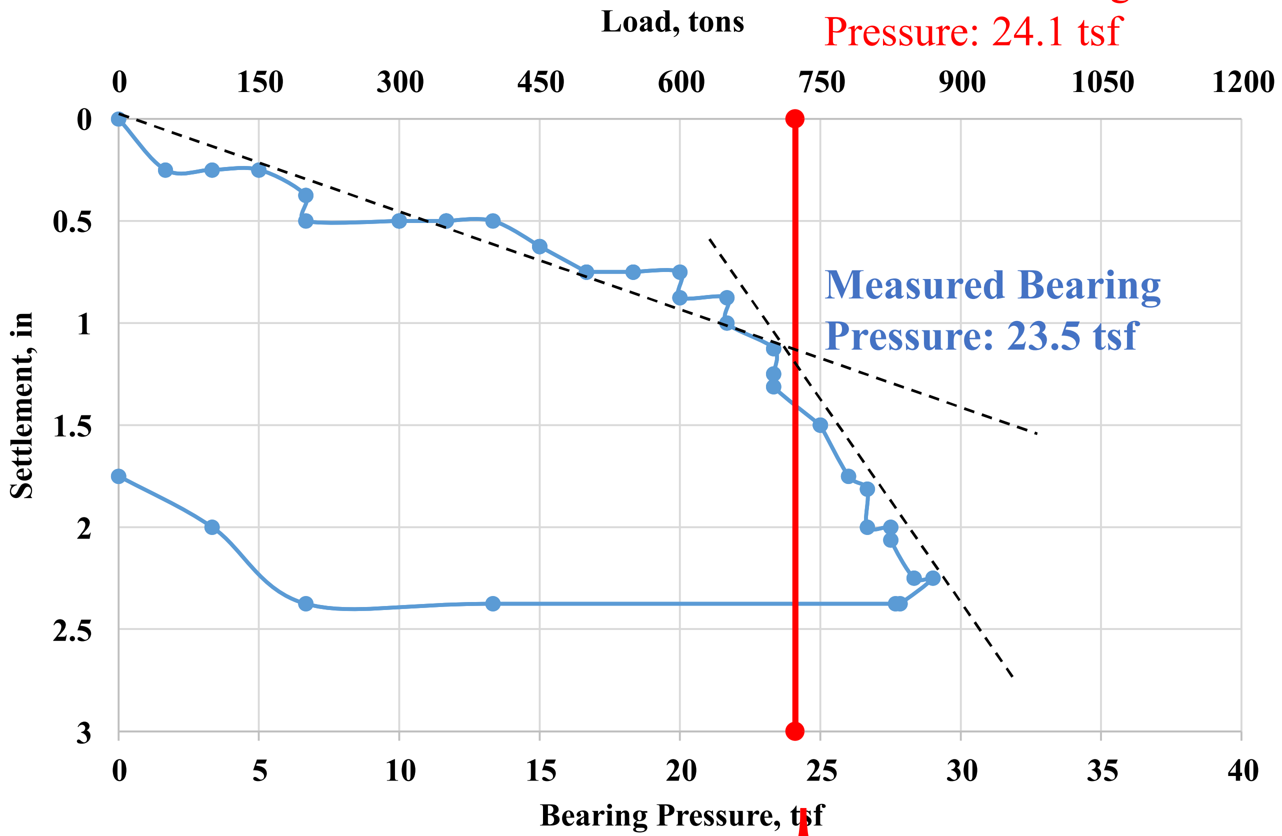

Installation of Girders

Measuring System


Hydraulic Jack and Load Cell Setup

Load Test 2: SR 84 Site, Load Test Results 5 ft x 6 ft Footing

110 pcf:
c = 44 psi
φ = 35.4°
Predicted Bearing Pressure: 24.1 tsf



Bearing Failure of Rock – Punching Shear

A rock over sand reduction factor, N_R = **1.195** was obtained based on the geometry (rock thickness) and elastic modulus ratio of layers

$Q_u = \min(Q_{u1}, Q_{u2}) * \xi / N_R = 24.1$ tsf

N_R = Rock thickness reduction factor

$N_R = 0.86 * R^{-0.25}$ if $R < 0.3$

$N_R = 1.2 - 0.1R$ if $R \geq 0.3$

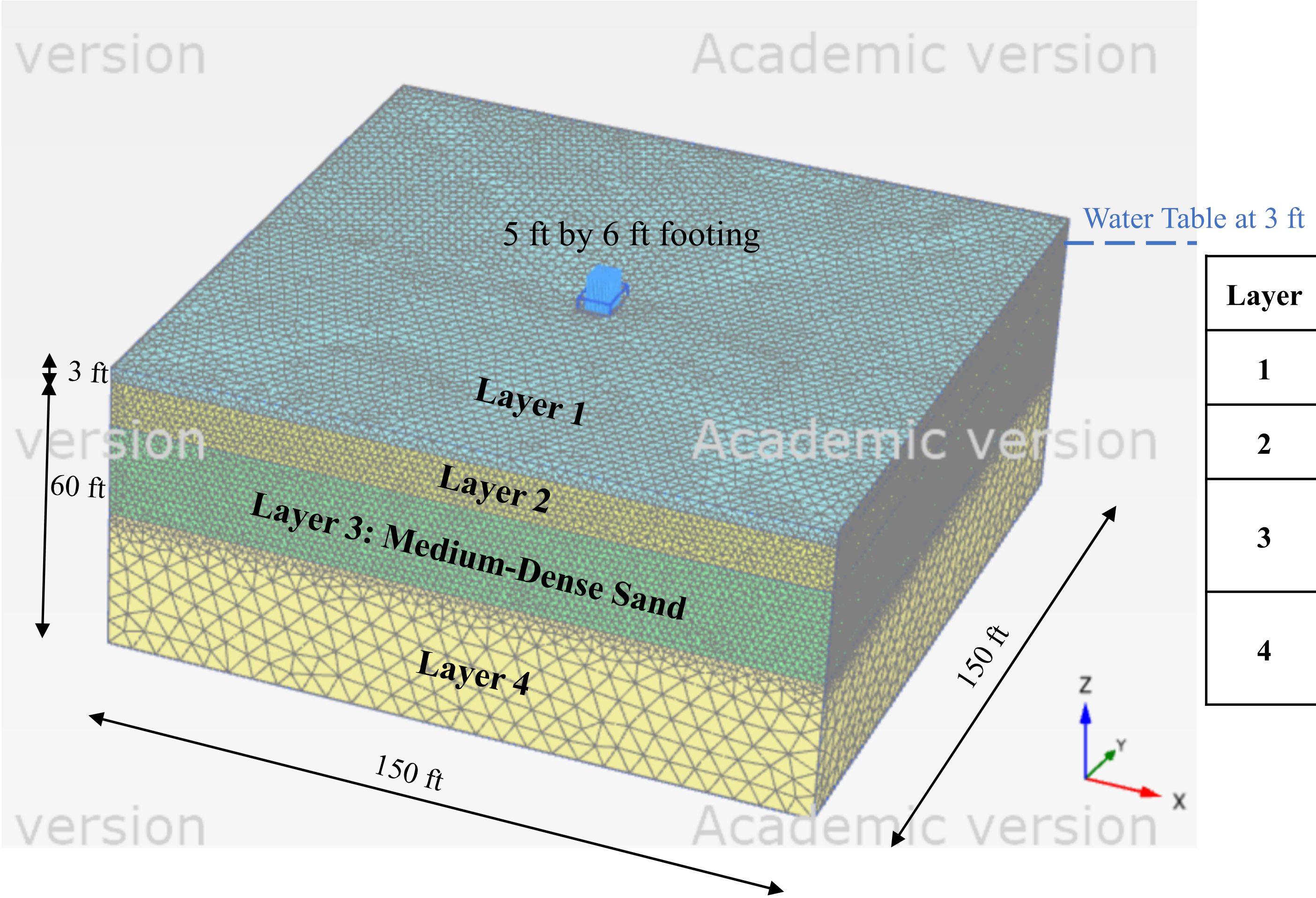
$R = 0.093T^2 (E_{soil} / E_{rock})$, limit R to 2.0

T = Rock thickness in feet, 5 ft {if T is in m, then $R = T^2 (E_{soil} / E_{rock})$}

E_{soil} / E_{rock} (1,087/38,000) = Modulus ratio of soil and rock layers

Load Test 2: SR 84 Site, Load Test Results: Settlement

FEM – Plaxis 3D - Nonlinear Solution



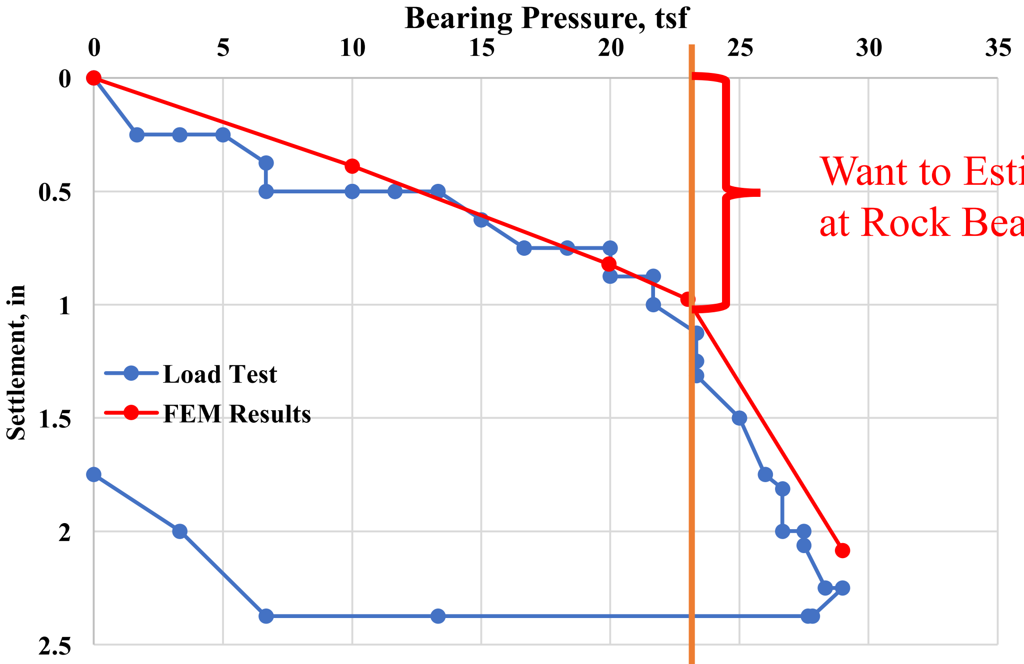
Material Model, Strength and Stiffness Parameters of Different Layer

Layer	γ_{dt}, pcf	Material	Material Model	c, psi	φ, °	ψ, °	μ	Young's Modulus, psi
1	90	Miami Limestone	Mohr-Coulomb	25	31	0	0.1	50,000
2	110	Miami Limestone	Mohr-Coulomb	44	35.4	0	0.1	38,000 ~ 19,000
3	108	Medium-Dense Sand	Mohr-Coulomb	0	32	32	0.3	1,087 ~ 500
4	136.5	Fort Thompson Limestone	Mohr-Coulomb	93.3	32.2	0	0.1	60,000

Load Test 2: SR 84 Site, 3D FEM Results

Stress State, Young's Modulus and Poisson's Ratio of Layer 2 in each Loading Stage

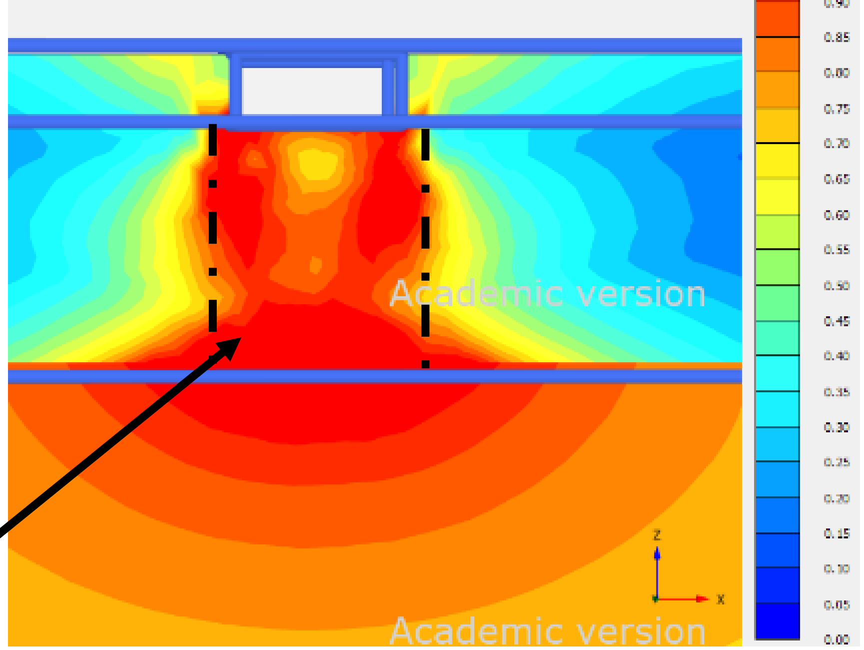
Loading Step	Bearing Pressure, tsf	Layer 2 110 pcf			
		σ1', psi	σ3', psi	Elastic Modulus, psi	Poisson's Ratio
1	0	3.7	1.5	38,000	0.1
2	10	2.7	1.6	38,000	0.1
3	19.95	40.5	-3.5	38,000	0.1
4	23	91.7	-7.0	38,000	0.1
5	29	112.2	-3.9	19,000	0.1



Measured versus FEM: load-settlement response

Want to Estimate Settlement at Rock Bearing failure

Bearing Failure of Rock – Punching Shear

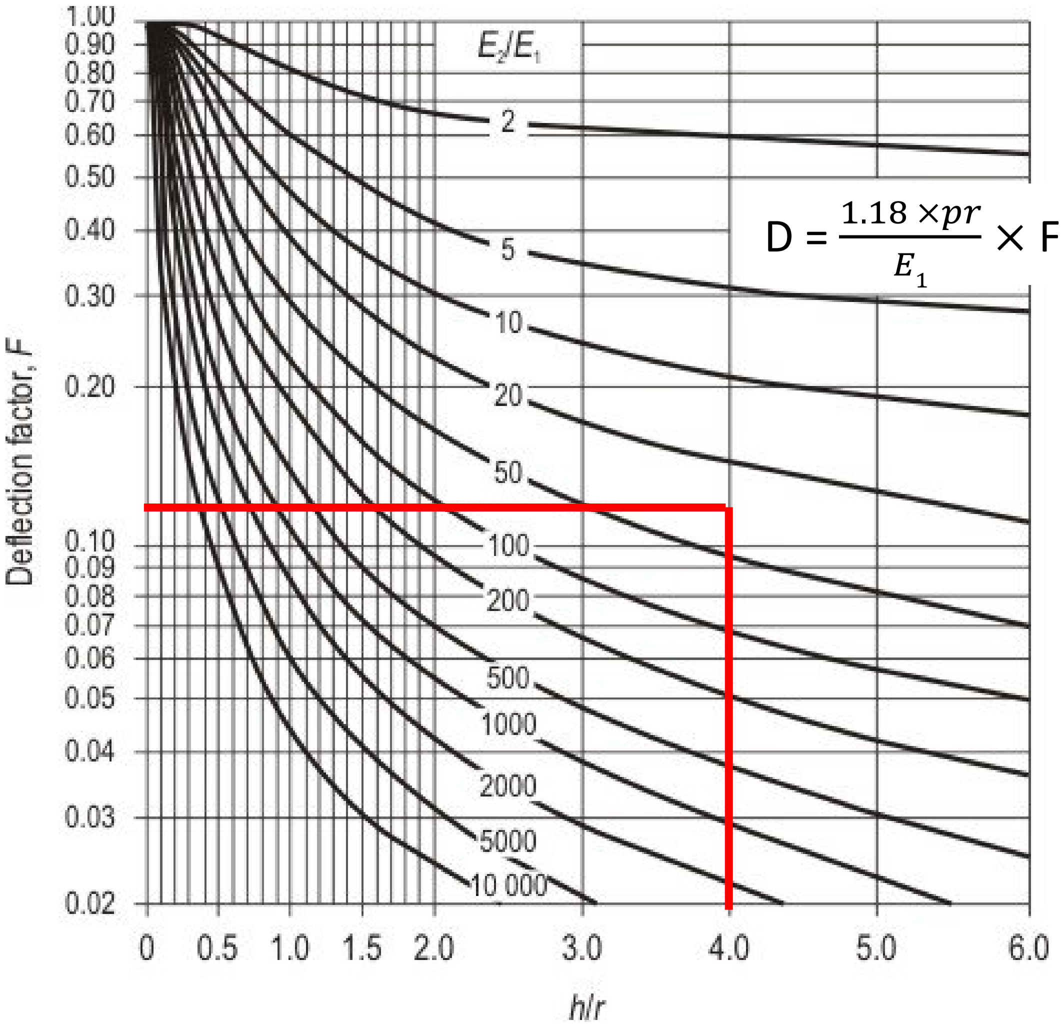


Contours of relative shear stresses (τ_{rel}) at SR-84 Site, Rock Failure

$$\tau_{rel} = \frac{\tau_{mob}}{\tau_{max}}$$, Plaxis 3D

Load Test 2: SR 84 Site, Estimated Settlement at Rock Punching Failure

Burmister Solution (1958) – settlement for two-layered – Linear Solution



E_2 = Upper layer elastic modulus (**median**) = 38,000 psi
E_1 = Lower layer elastic modulus = 1,087 psi
r = width of footing = 2.5 ft
h = thickness of upper layer = 10 ft
F = Deflection Factor = 0.11
P = bearing pressure = 23.5 tsf

To predict the settlement at Punching Shear Failure occurs

$$D = \frac{1.18 \times Pr}{E_1} \times F = 1.14 \text{ in}$$

Using **Secant Modulus** – At Punching Failure
FEM: 0.98 in vs. Burmister's Method: 1.14 in

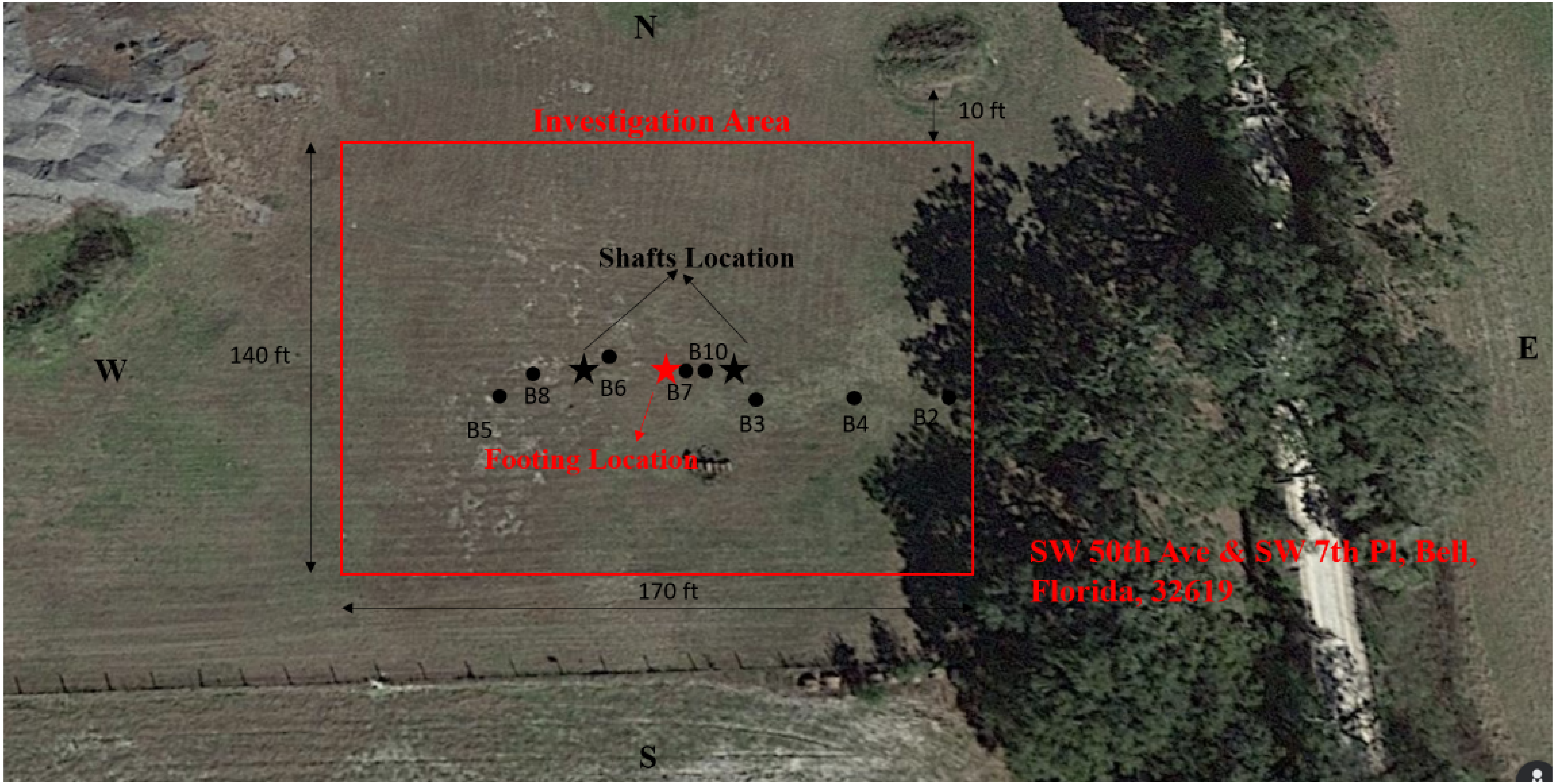
Measured Qu = 326 psi bearing pressure with 1.125 in of settlement at SR-84 Site.

Comparison of Bearing Pressure with near bridge pier spread footings – SR-84 Site

Locations	Design Method	Footing Geometry			Rock Thickness, ft	Rock Strength			Nominal Bearing Pressure, ksf
		B', ft	L', ft	D_f, ft		γ at Df, pcf	c, psf	φ_f, °	
Load Test at SR-84	FL Bearing Capacity Equations	5	6	3	10	110	6336	35.4	48
Pier 4 Spread Footing	AASHTO LRFD Bridge Design	17.49	21.5	8	6	130	0	32	36.4
Pier 5 Spread Footing	AASHTO LRFD Bridge Design	16.7	23	8	9	130	0	32	37.8
Pier 6 Spread Footing	AASHTO LRFD Bridge Design	17.8	23.51	8	6	130	0	32	34.6
Piles used for Pier 7 and Pier 8									
Pier 9 Spread Footing	AASHTO LRFD Bridge Design	16.8	25.3	9	9	130	0	32	45.4
Pier 10 Spread Footing	AASHTO LRFD Bridge Design	15.6	25.8	8	12	130	0	32	57.5
Pier 11 Spread Footing	AASHTO LRFD Bridge Design	18.8	24.1	8	8	130	0	32	35.8
Pier 12 Spread Footing	AASHTO LRFD Bridge Design	19.6	20.4	10	10	130	0	32	50.3
Pier 13 Spread Footing	AASHTO LRFD Bridge Design	17.48	24.66	8	11	130	0	32	43.2
Pier 14 Spread Footing	AASHTO LRFD Bridge Design	19.6	22.8	8	9	130	0	32	40.7
Pier 15 Spread Footing	AASHTO LRFD Bridge Design	19.3	24.7	8	12	130	0	32	44.6
Pier 16 Spread Footing	AASHTO LRFD Bridge Design	17.4	23.3	8	11	130	0	32	45.2

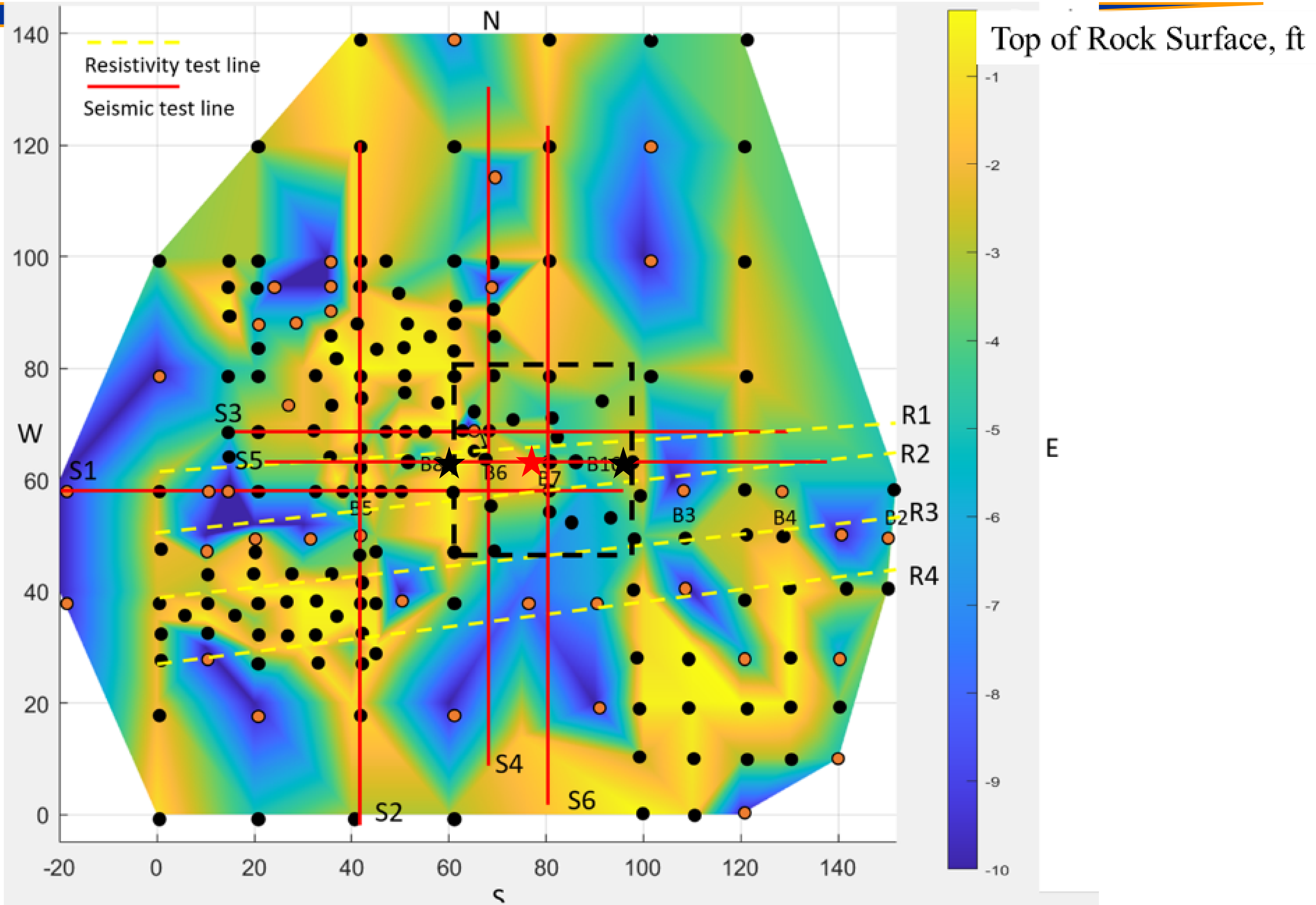
Load Test 3: Bell Site, Site Investigation

B2 & B3: SPT, B4 & B5 & B10: Rock Coring, B7 & B8 & B10: MWD, B6: no rock down to 10 ft

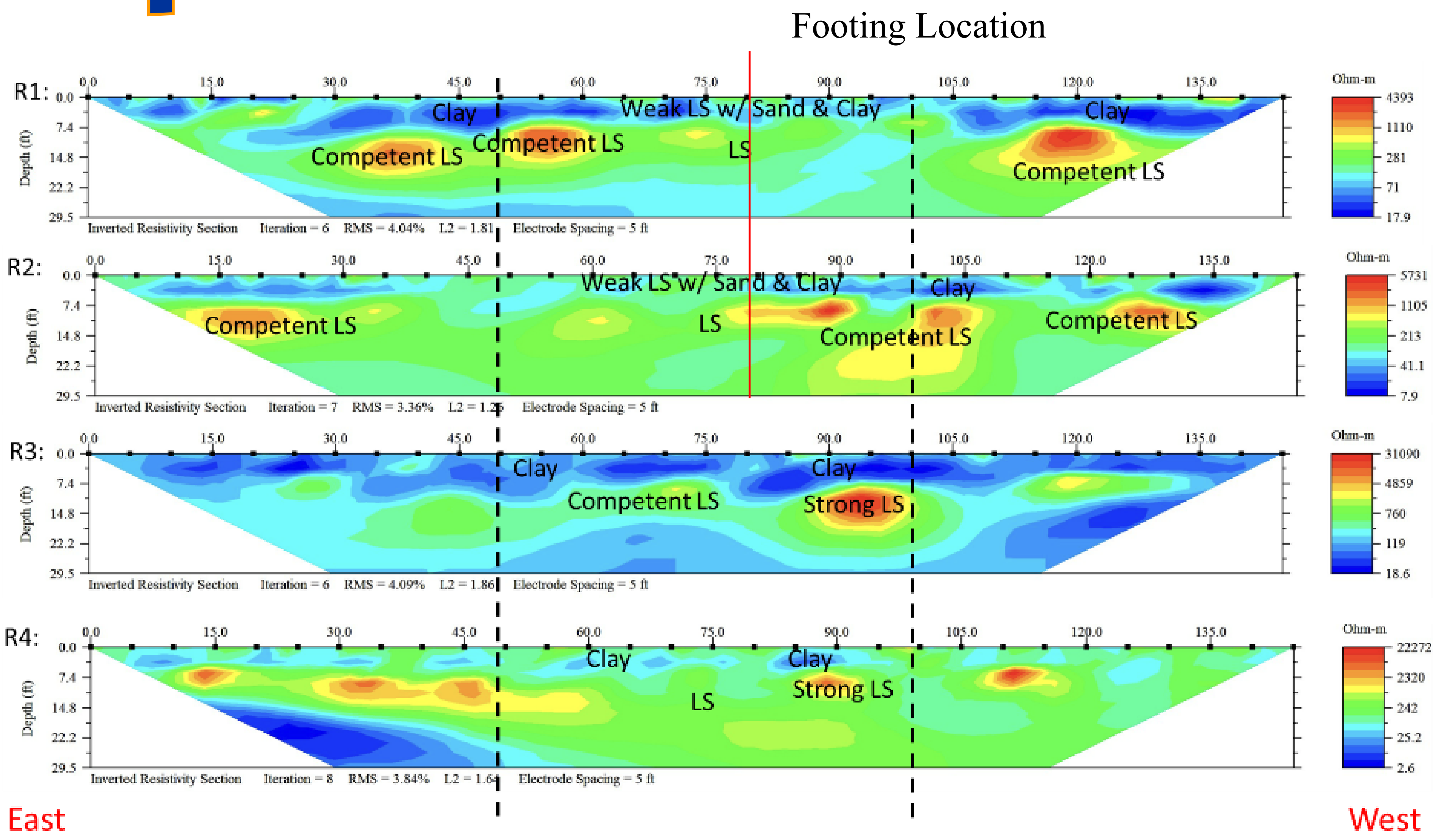


Load Test 3: Bell Site, Site Investigation (200 Hand Auger Holes)

- ● Borings have Top of Rock Surface within 5 ft depth from GSE
- ● Borings have Top of Rock Surface below 5 ft depth from GSE
- ★ Shafts Location
- ★ Footing Location
- Influence Zone



Load Test 3: Bell Site, Site Investigation (Resistivity Test)



Sand: 70 ~ 500 Ohm-m

Clay: 1 ~ 150 Ohm-m

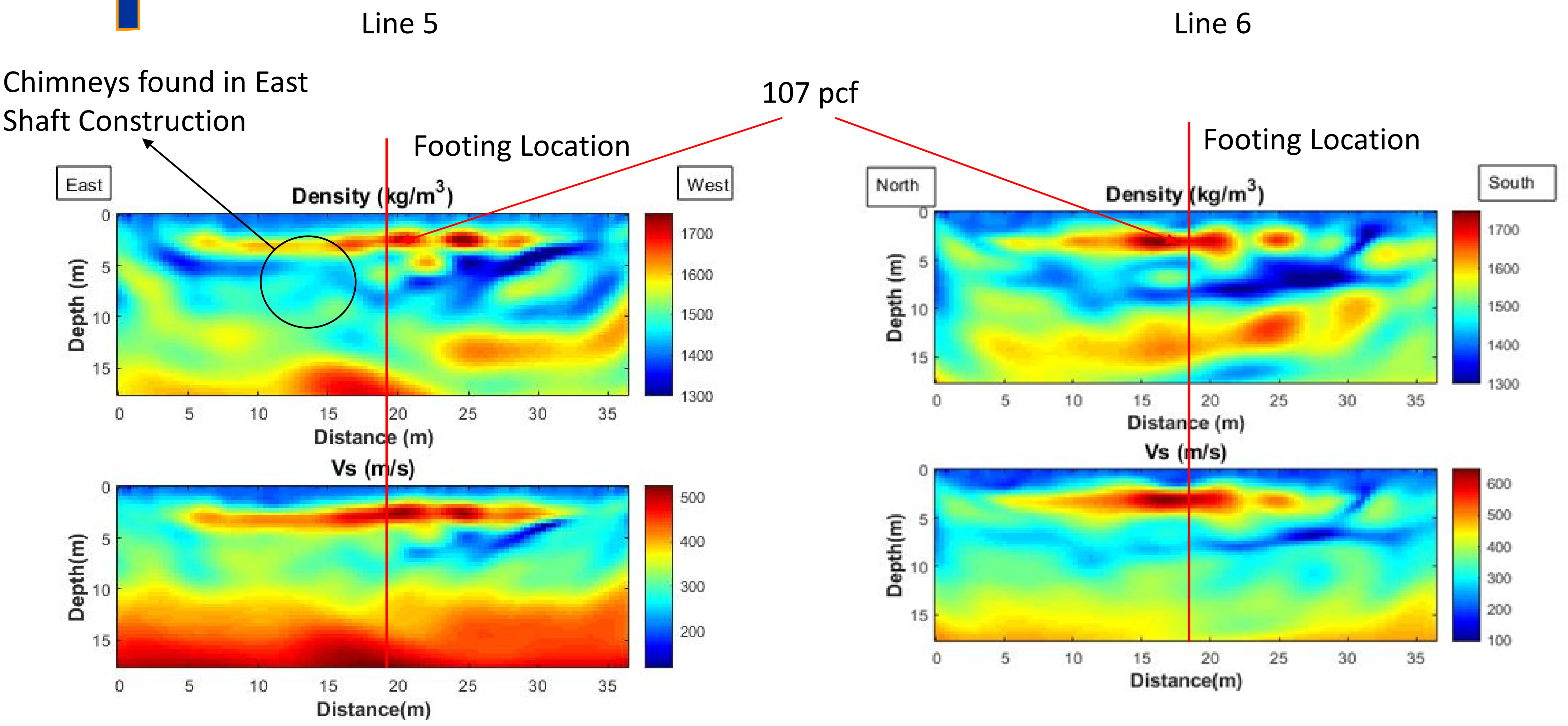
Weathered Limestone: 50 ~ 200 Ohm-m

Competent Limestone: > 700 Ohm-m

No **Dry Unit Weight**, Cannot differentiate the **layering**.

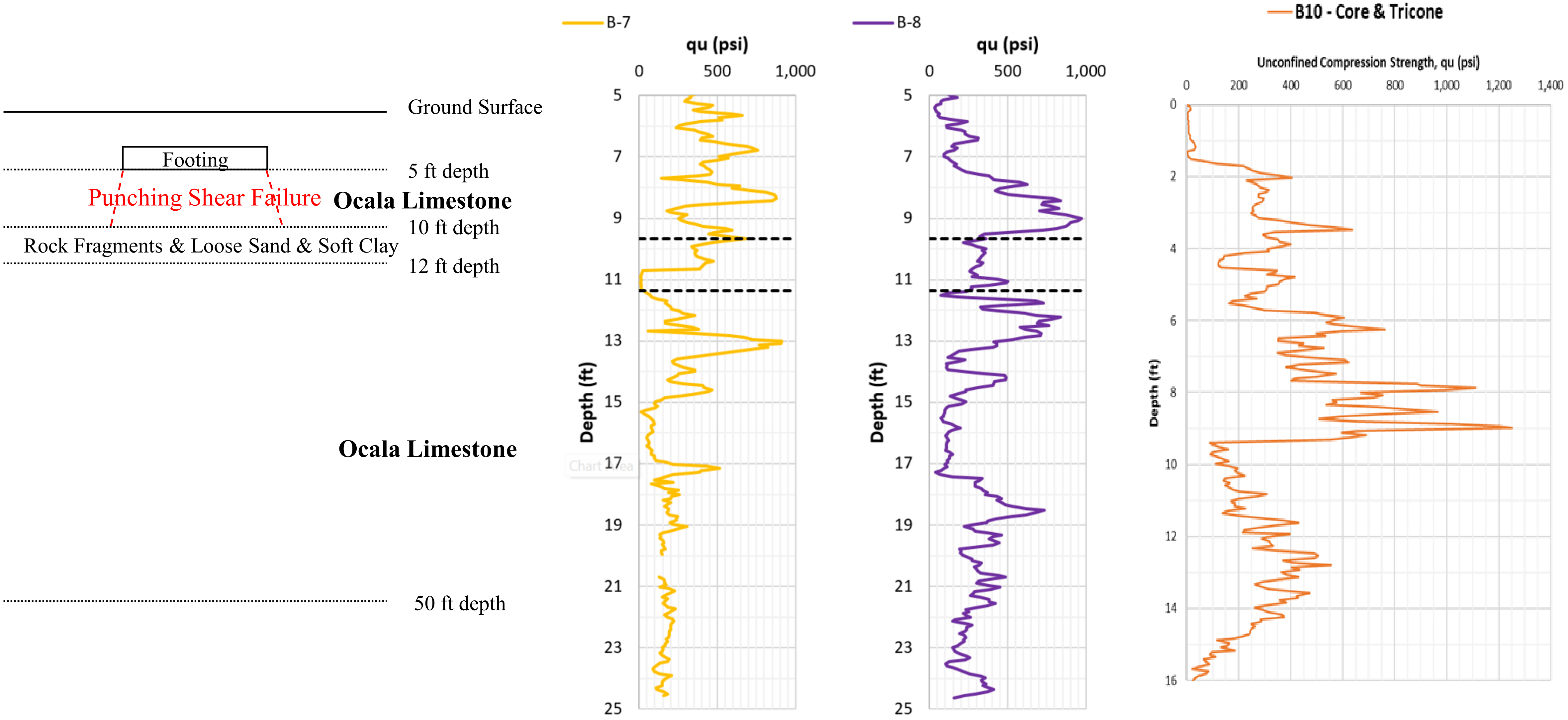
Imaging of Deep Sinkholes Using the Multi-electrode Resistivity Implant Technique Case Studies in Florida (David Harro, Henok Kiflu)

Load Test 3: Bell Site, Site Investigation, Seismic Results

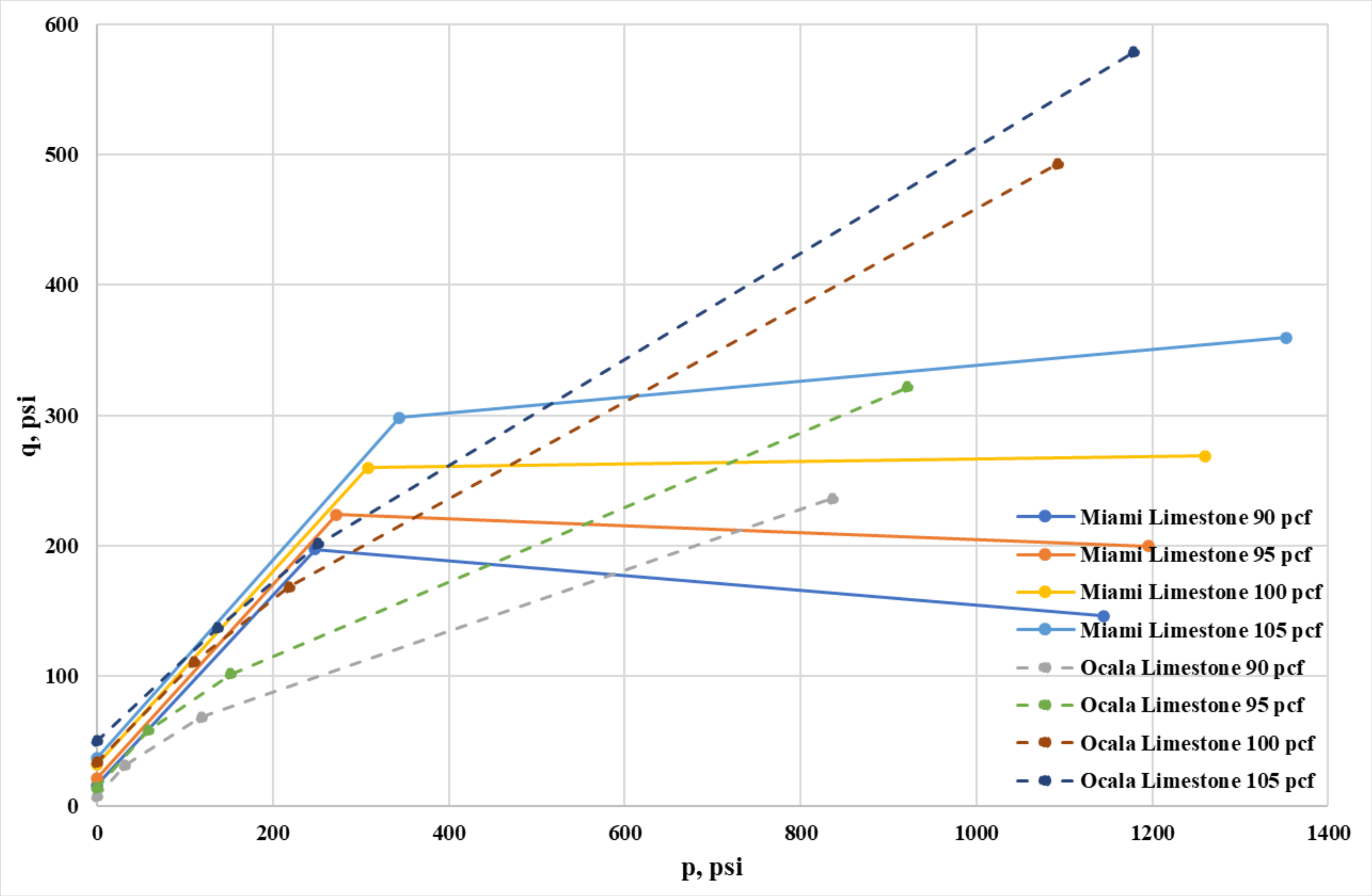


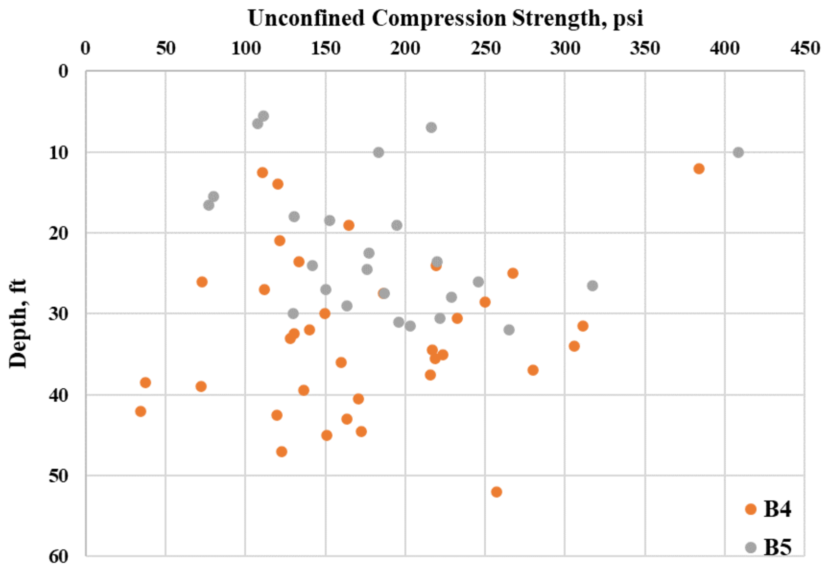
Based on the Dry unit weight (107 pcf), the **105 pcf** Ocala Limestone strength envelope is used, a **5 ft by 5 ft (14 tsf bearing pressure) footing** is selected by using the FL Bearing Capacity Equations

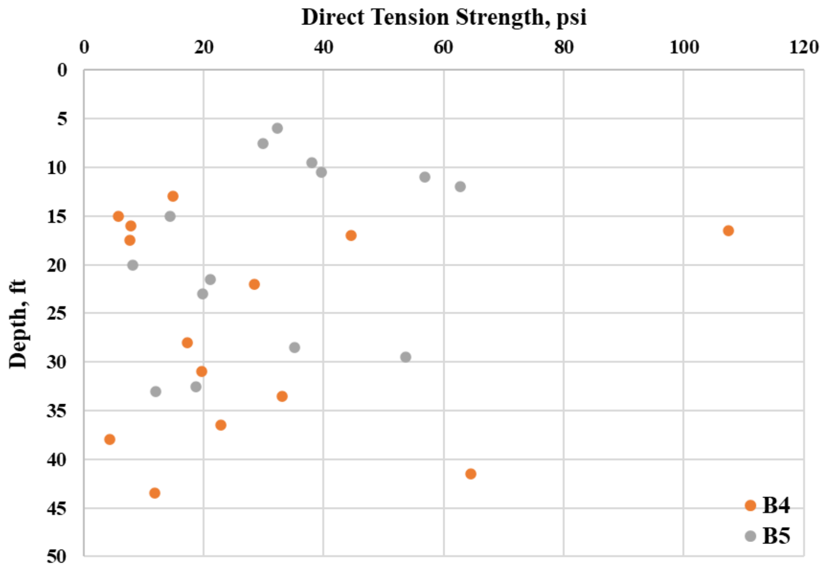
Load Test 3: Bell Site, Site Investigation



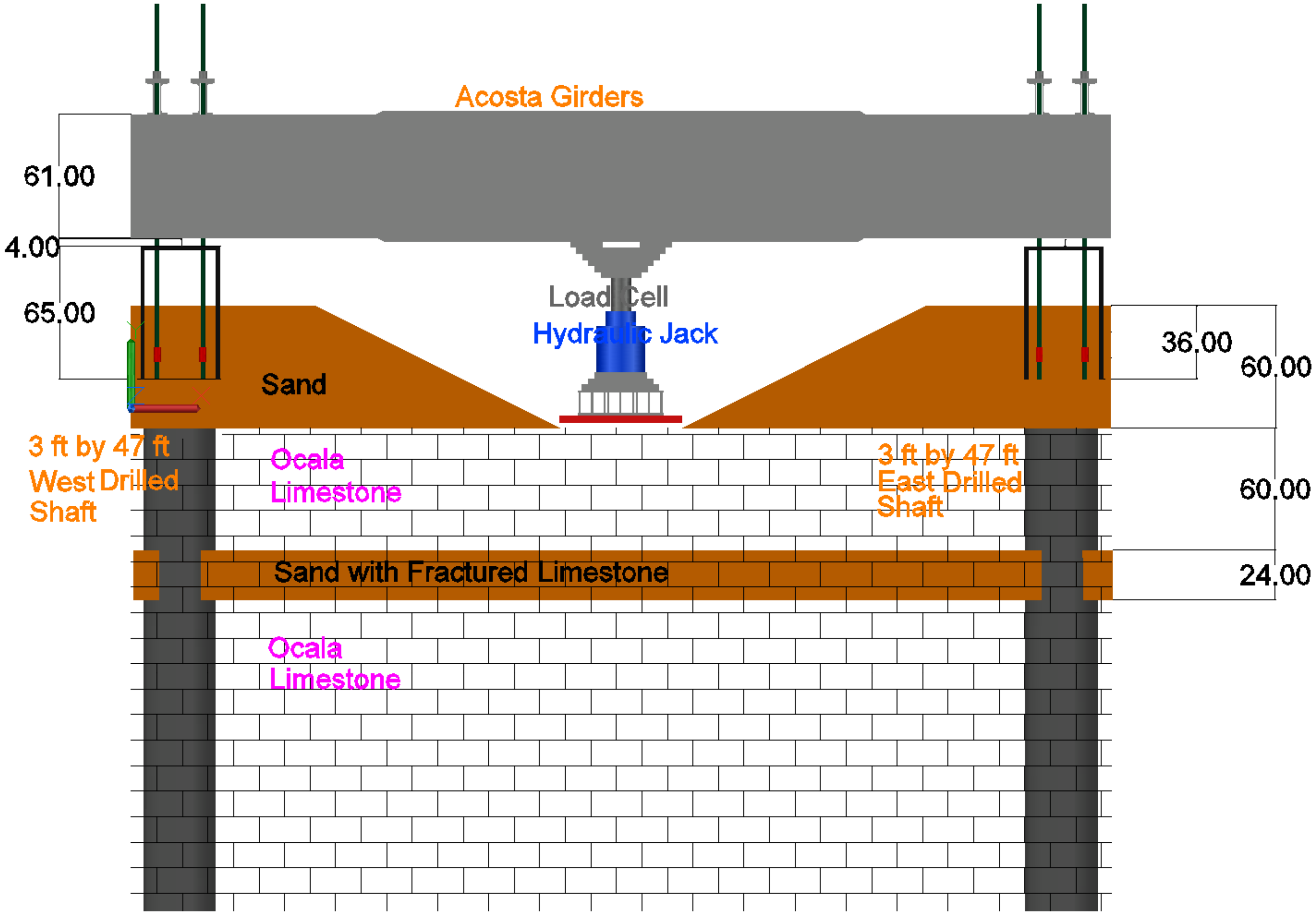
Load Test 3: Bell Site, Site Investigation







Load Test 3: Drilled Shaft Installation and Load Test



$$f_s = REC \times \frac{1}{2}\sqrt{q_u \times q_{dt}} = 26\ psi$$

Reaction $= 26 \times 47 \times 12 \times 36 \times 3.14 \times 2\ /\ 2000 = 1658$ tons

Factor of safety $= 1658/900 = 1.84$

Load Test 3: Drilled Shaft Installation and Load Test

West Shaft

Casing Placement

Bailing Bucket to clean the hole

Rebar Cage Installation

Load Test 3: Drilled Shaft Installation and Load Test

West Shaft

Rebar Cage Installation

Concrete Pumping

Load Test 3: Drilled Shaft Installation and Load Test

East Shaft

Drill down to 50 ft, Sinkhole shows up

Fill the drilling hole and sinkhole with asphalt

Load Test 3: Drilled Shaft Installation and Load Test

East Shaft

Chimney found in the drilling hole

Rebar Cage Placement

Leveling Shafts

Load Test 3: Drilled Shaft Installation and Load Test

Based on the Voids Area, REC = 83%

Top of Rock Surface at 5 ft depth

Leveled Concrete at Footing Location

Load Test 3: Drilled Shaft Installation and Load Test

Load Test

Threaded Rods and Stands Placement

Load Test

Load Test 3: Drilled Shaft Installation and Load Test

Load Test

Measuring System

Auto Level

1: Front Left (FL)
2: Front Right (FR)
3: Hydraulic Jack, Middle (Courtesy of AFT)
4: Rear Left (RL)
6: Rear Right (RR)

Load Test 3: Bearing Capacity

A rock over sand reduction factor, $\mathbf{N_R = 1.66}$ was obtained based on the geometry (rock thickness) and elastic modulus ratio of layers

$Q_u = \min(Q_{u1}, Q_{u2}) * \xi / N_R = \mathbf{14\ tsf}$

N_R = Rock thickness reduction factor

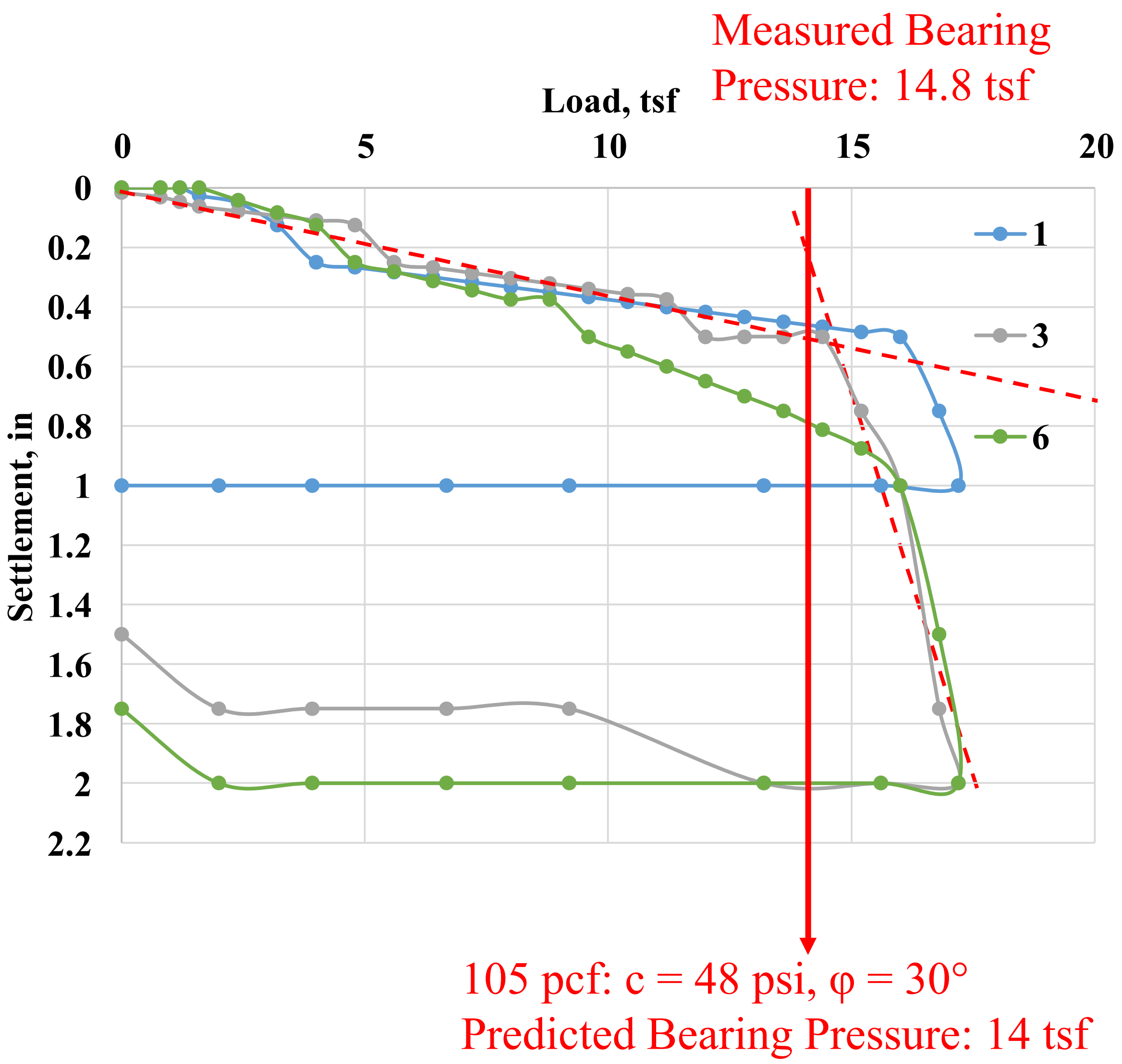
$N_R = 0.86 * R^{-0.25}$ if $R < 0.3$

$N_R = 1.2 - 0.1R$ if $R \geq 0.3$

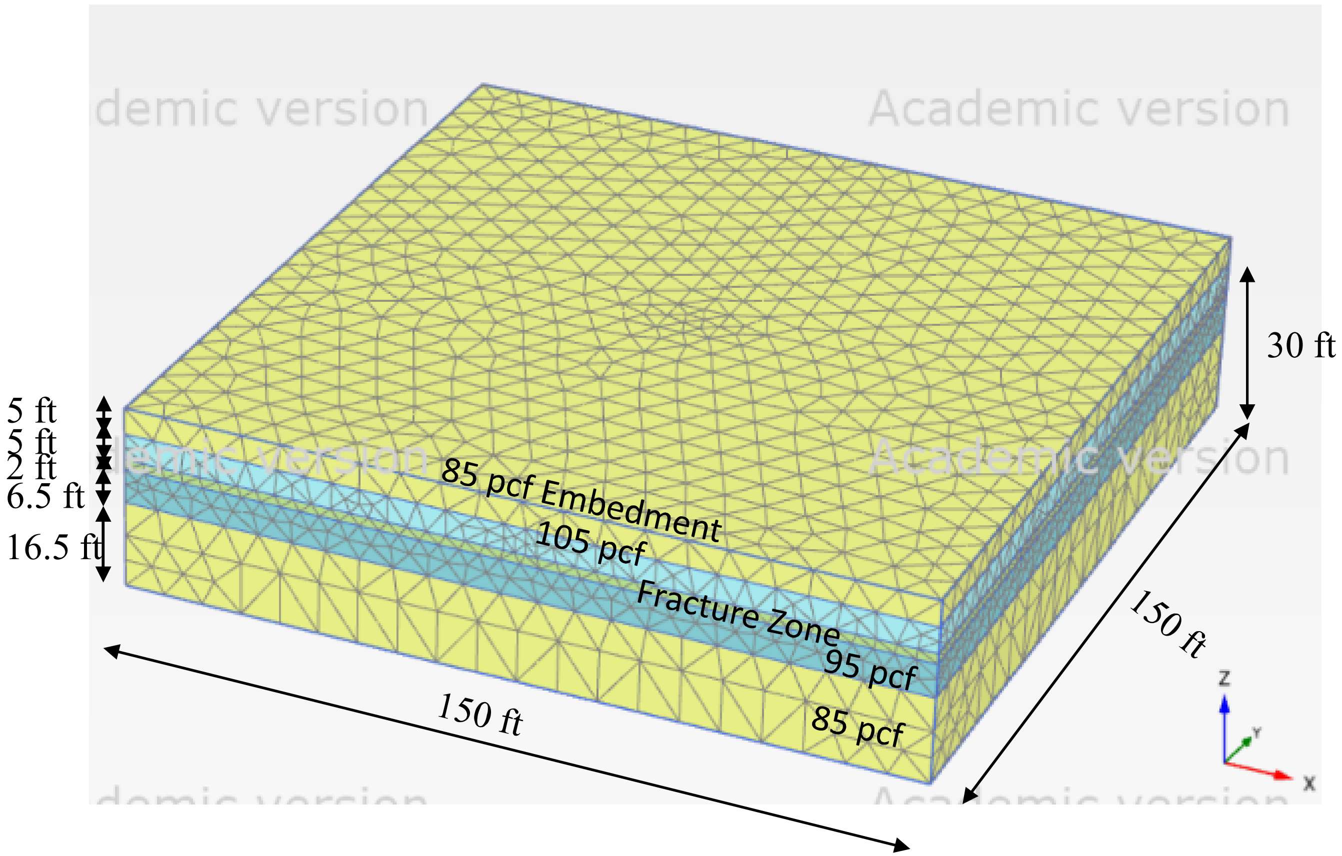
$R = 0.093T^2 (E_{soil} / E_{rock})$, limit R to 2.0

T = Rock thickness in feet (5 ft) {if T is in m, then $R = T^2 (E_{soil} / E_{rock})$}

E_{soil} / E_{rock} (1,500/48,787) = Modulus ratio of soil and rock layers

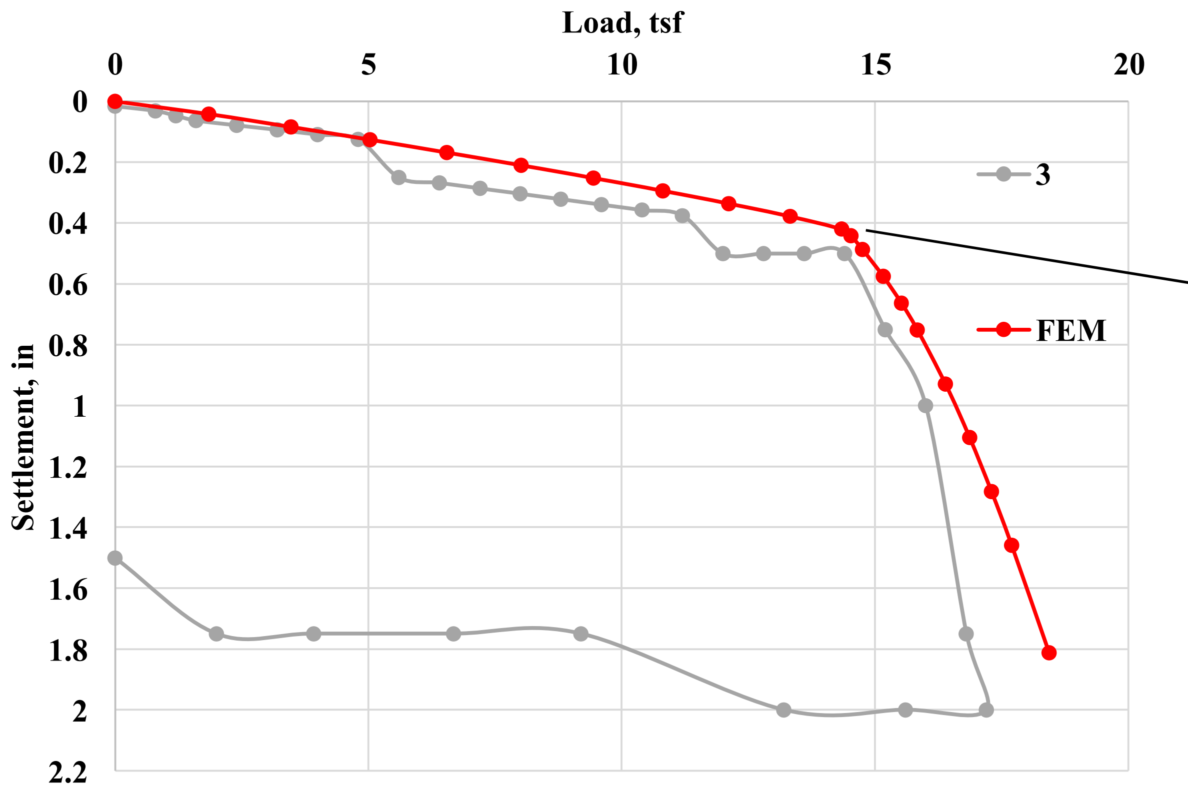


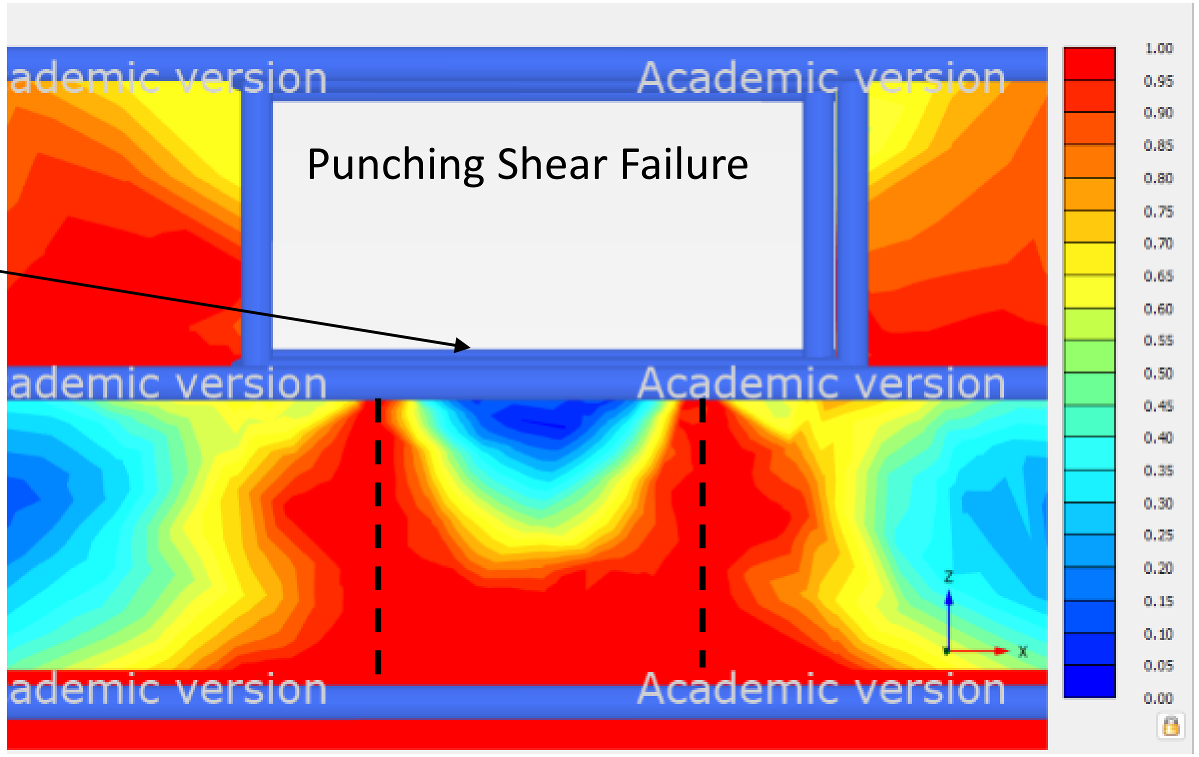
Load Test 3: Settlements



Layer	γ_{dt}, pcf	Material	Material Model	c, psi	φ, °	ψ, °	μ	Young's Modulus, psi
1	85	Ocala Limestone	Mohr-Coulomb	6.6	19.8	0	0.1	20,967
2	105	Ocala Limestone	Mohr-Coulomb	48	30	0	0.1	48,786 ~ 13,889
3	80	Fracture Zone	Mohr-Coulomb	0	20	0	0.2	1,500 ~ 500
4	95	Ocala Limestone	Mohr-Coulomb	19.5	24	0	0.1	34,877
5	85	Ocala Limestone	Mohr-Coulomb	6.6	19.8	0	0.1	20,967

Load Test 3: Settlements

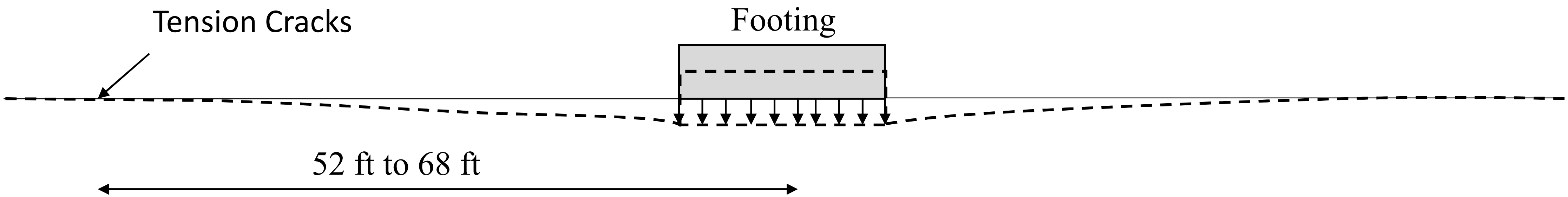




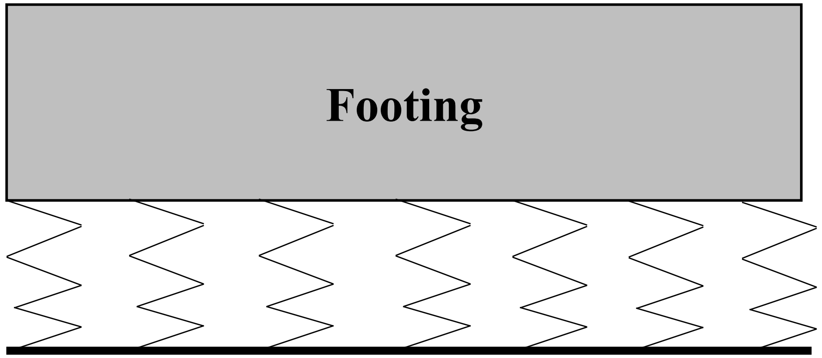
Contours of relative shear stresses (τ_{rel}) at SR-84 Site, Rock Failure

$\tau_{rel} = \frac{\tau_{mob}}{\tau_{max}}$, Plaxis 3D

Load Test 3: Bell Site, Tension Cracks



Methods to Predict Footing Settlement



Spatial Distributed Nonlinear Springs in FB-Multipier

Finite Element Method

Linear or Nonlinear Load-Settlement response, and Multiple Layers

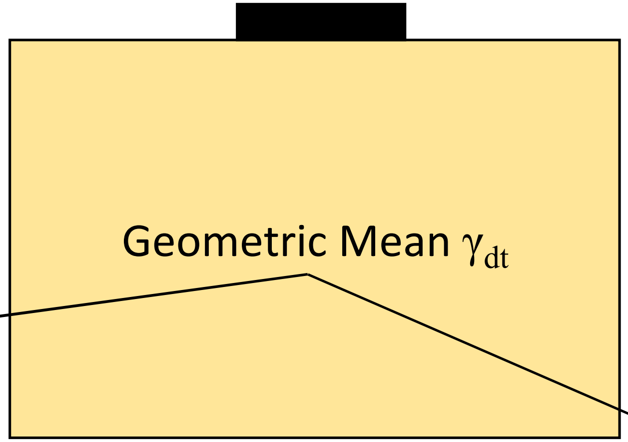
Burmister Method with FL Bearing Capacity Equation

Used for Rigid Footing, only predict the mean settlement up to bearing failure of rock using the secant modulus, Linear Load-Settlement response

Beam on Nonlinear Winkler Foundation (BNWF) Model with FL Bearing Capacity Equation (FB-Multipier)

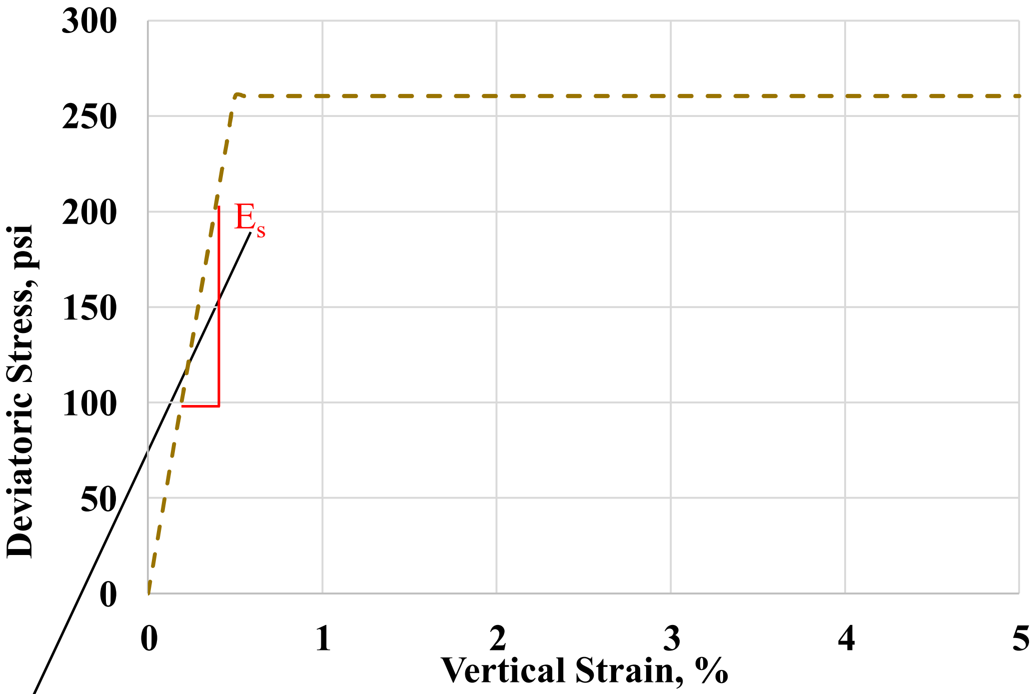
Predict the settlement along the footing's length, provide shear and moment distribution within the footing (Bridge Pier Design), nonlinear load-settlement response by changing secant modulus (function of stress/strain level)

BNWF Model – Single Layer

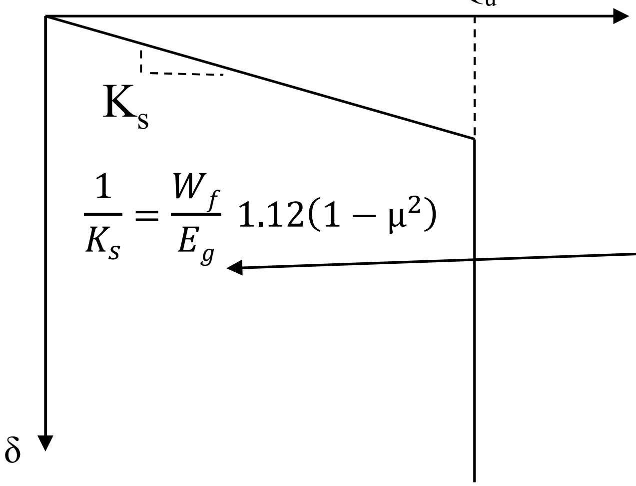


FL Bearing Capacity Equations: Q_u

Mean Settlement by Elastic Solution using the Secant Modulus: δ



Q_u from FL Bearing Capacity (Phase 1)



$$\frac{1}{K_s} = \frac{W_f}{E_g} 1.12(1 - \mu^2)$$

E_g: **Geometric Mean** of the elastic modulus values over the region of influence

$$E_g = \exp\{\frac{1}{W_f H} \sum \ln E(x, y)\}$$

where,
W_f = footing width
H = overall depth of soil layer

Differential Settlement by Fenton & Griffiths method (2002): Δδ. **Note, the CV of typical Florida Limestone is around 1, larger than the values (0.2) found in literatures.**

BNWF Model - Two Layer System

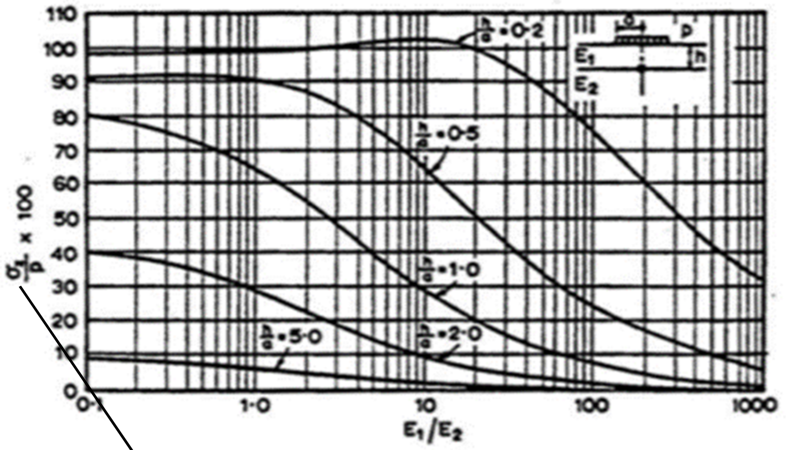
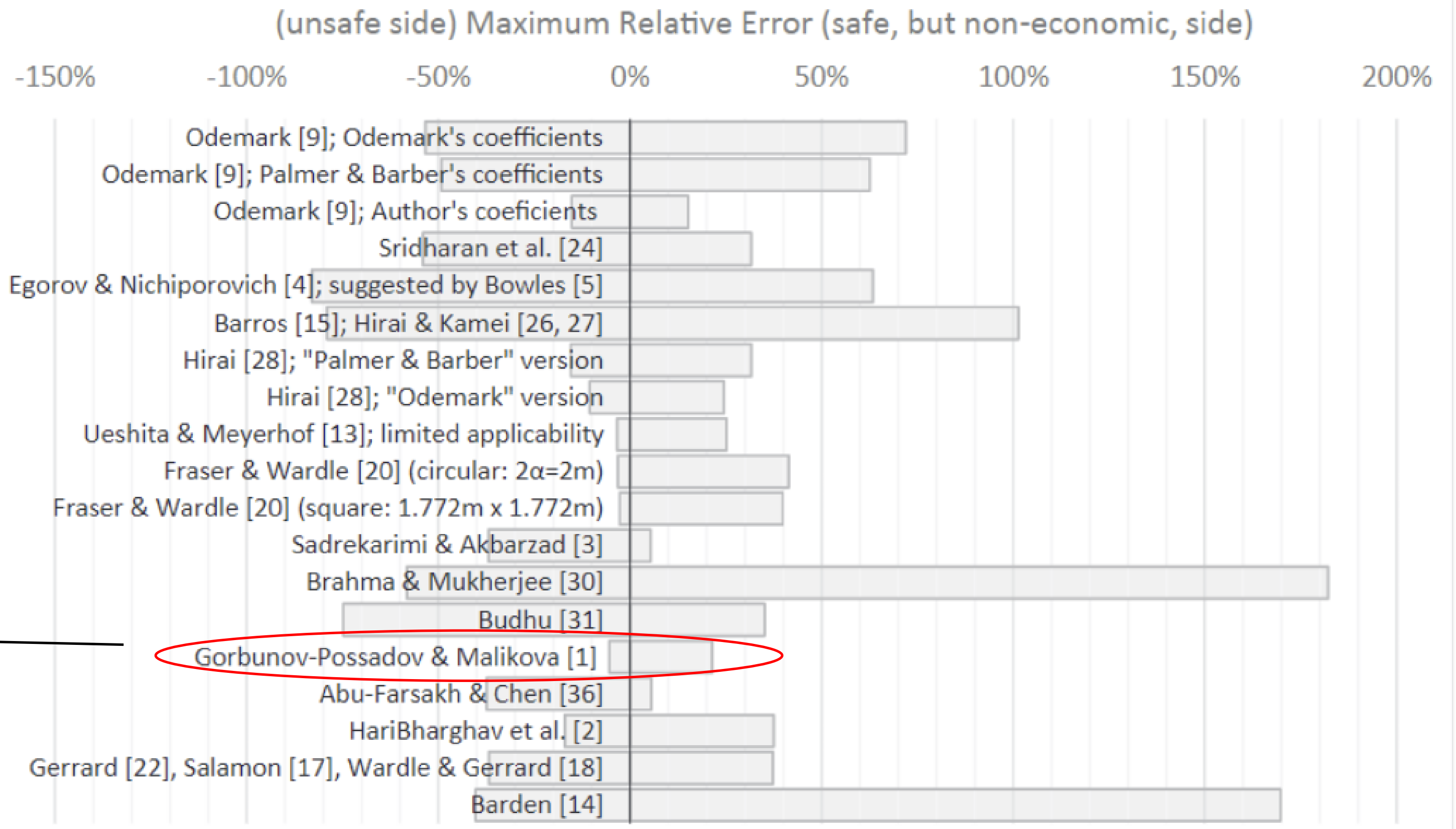


FIG.6.3 Vertical interface stress on axis (Fox, 1948a).

$$E_{eq} = \frac{\sum h_i \bar{\sigma}_{zi}}{\sum \frac{h_i \bar{\sigma}_{zi}}{E_i}}$$

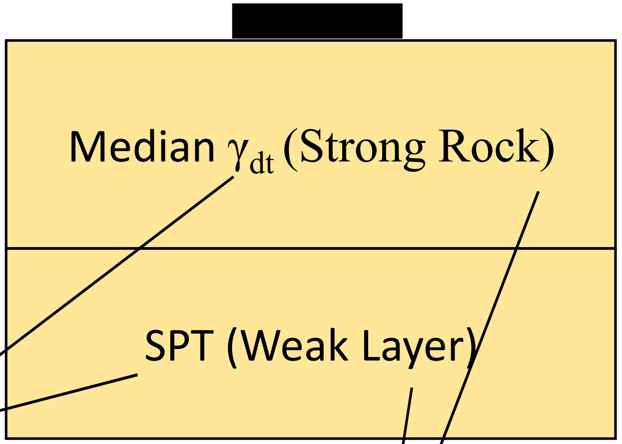
Weighted Harmonic Mean, Gorbunov-Possadov and Malikov (1973)



Maximum Relative Error between Equivalent Modulus Methods

The equivalent modulus of elasticity of layered soil mediums for designing shallow foundations with the Winkler spring Hypothesis: A critical review (2019)

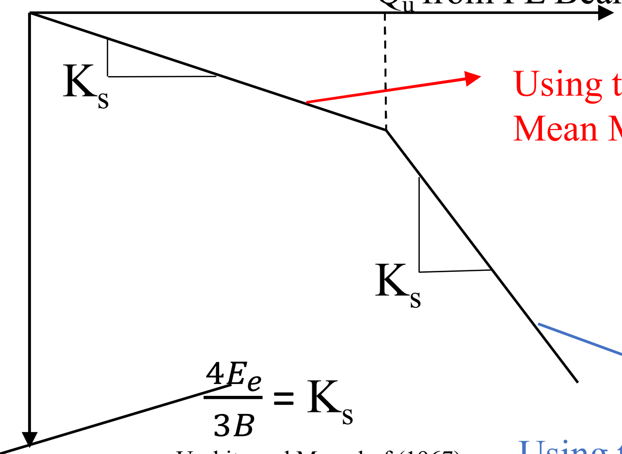
BNWF Model - Two Layer System



FL Bearing Capacity Equations: Q_u, Require the thickness of rock and moduli ratio

E_1 from dry unit weight of rock and soil/E_2 from SPT below count, Generating the Harmonic Mean. After Bearing, the rock modulus become secant failure modulus

Q_u from FL Bearing Capacity (Phase 1)



Using the Weighted Harmonic Mean Modulus before rock failure

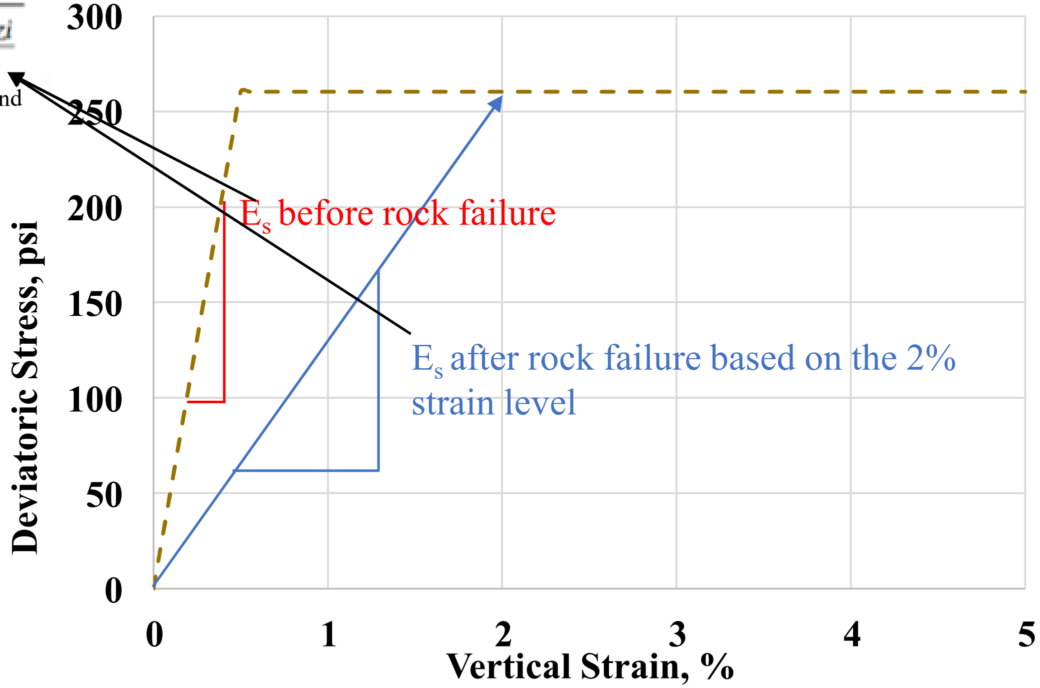
Using the Secant Weighted Harmonic Mean Modulus after rock failure

$$\frac{4E_e}{3B} = K_s$$

Ueshita and Meyerhof (1967)

$$E_{eq} = \frac{\sum h_i \bar{\sigma}_{zi}}{\sum \frac{h_i \bar{\sigma}_{zi}}{E_i}}$$

Gorbunov-Possadov and Malikov (1973)

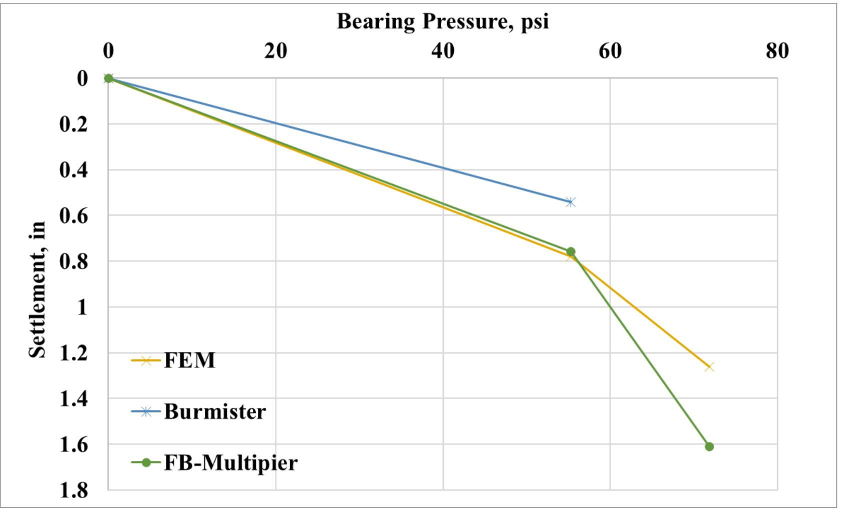


BNWF Model – 2 Layers (5 ft by 6 ft footing, Sand Modulus = 1,000 psi)

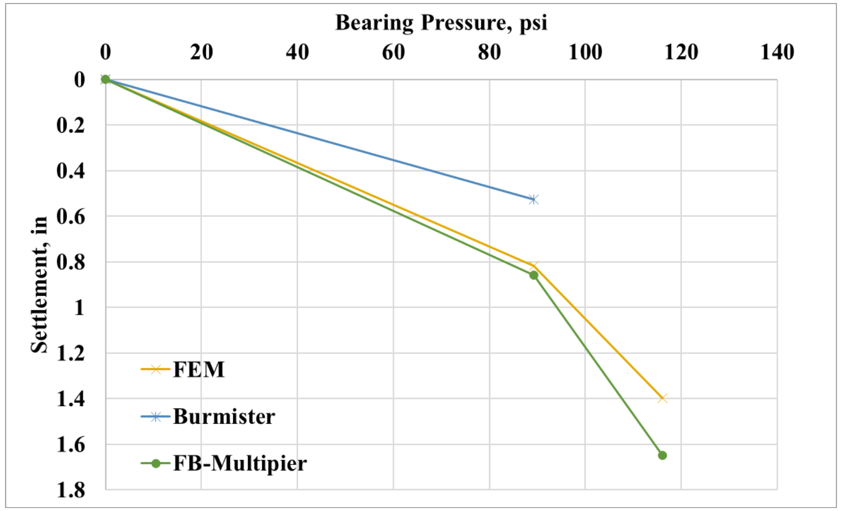
Maximum deviations of settlement: 20%.

Future Research will be expanded to different footing width, L/B ratio, rock thickness and different sand modulus.

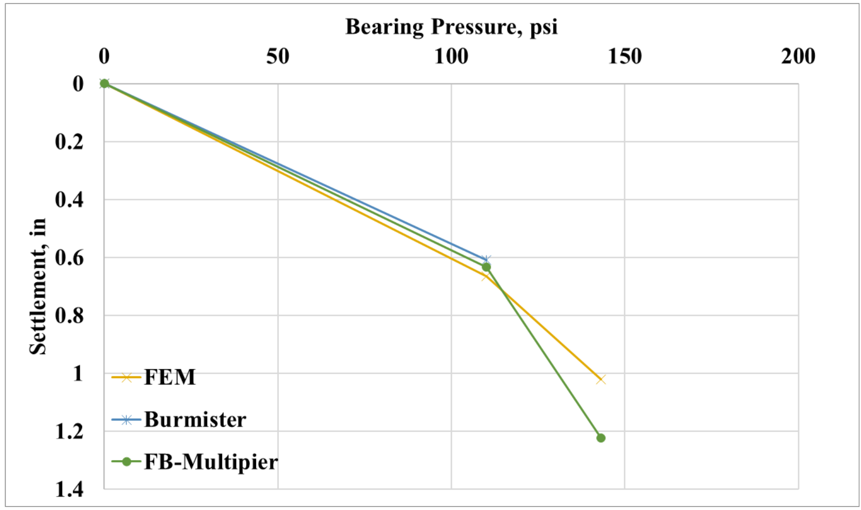
95 pcf, rock thickness: 2.5 ft



95 pcf, rock thickness: 5 ft

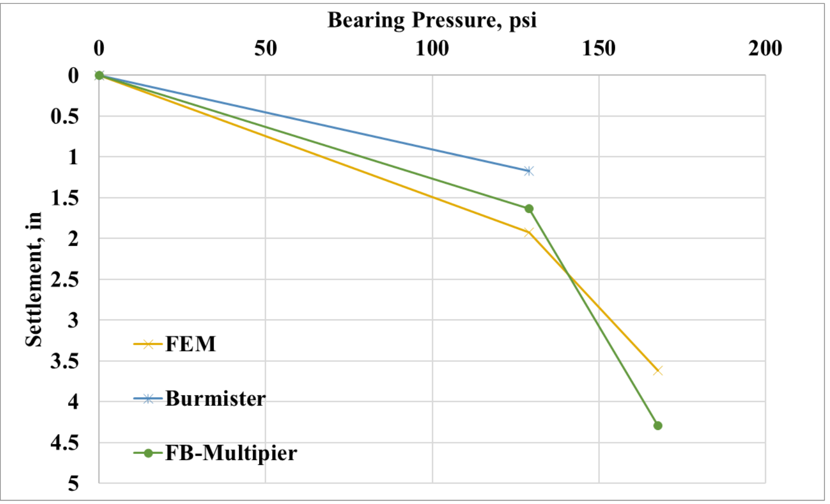


95 pcf, rock thickness: 7.5 ft

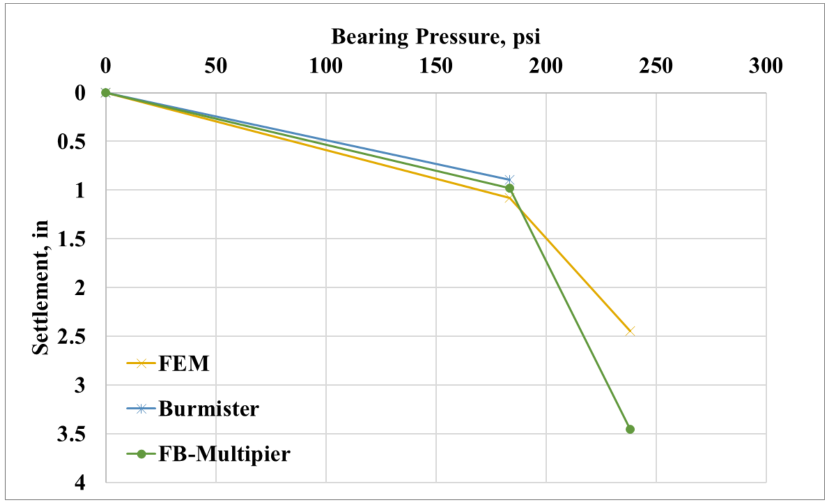


BNWF Model – 2 Layers (5 ft by 6 ft footing, Sand Modulus = 1,000 psi)

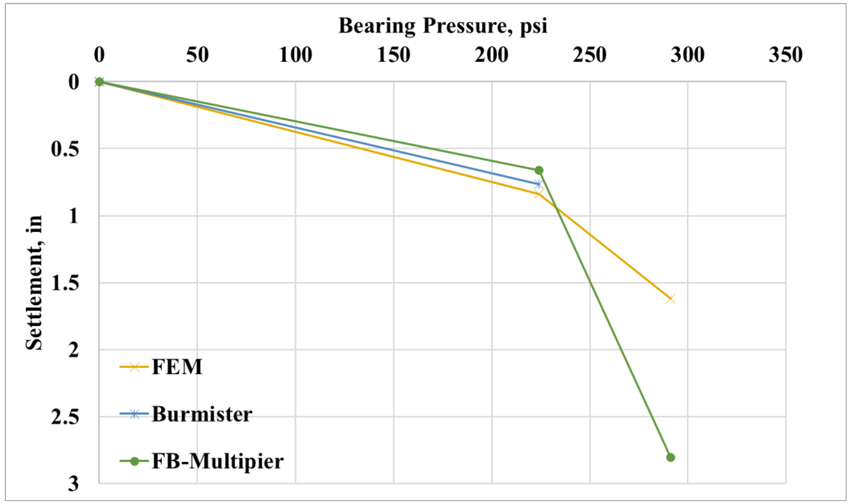
110 pcf, rock thickness: 2.5 ft



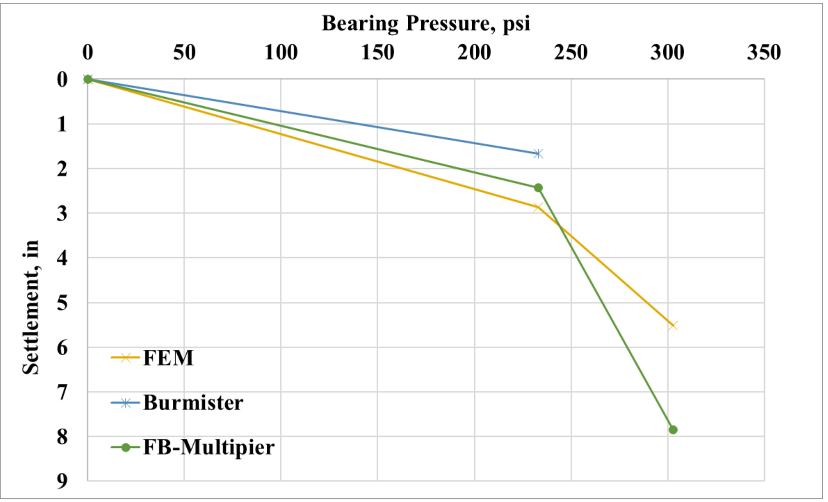
110 pcf, rock thickness: 5 ft



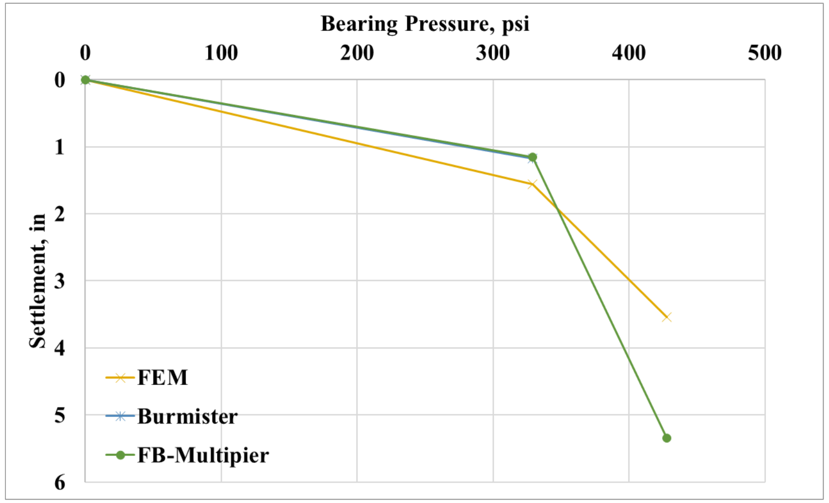
110 pcf, rock thickness: 7.5 ft



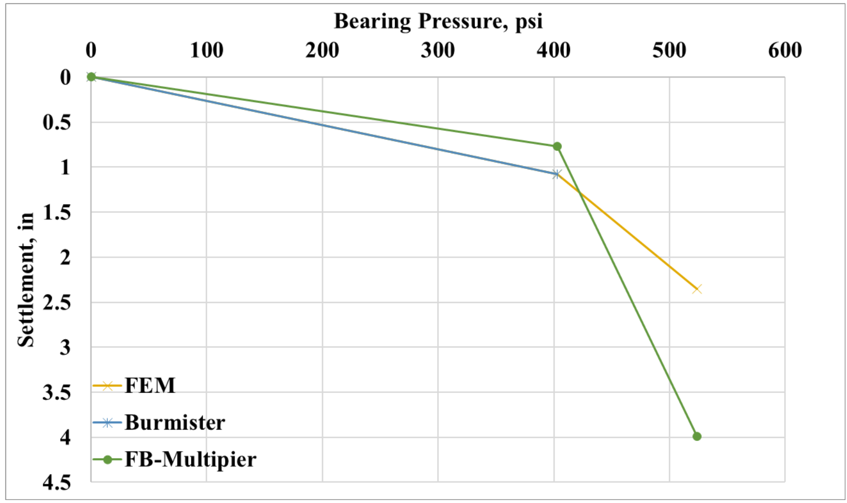
130 pcf, rock thickness: 2.5 ft



130 pcf, rock thickness: 5 ft



130 pcf, rock thickness: 7.5 ft



Preliminary Results and Recommendation

- The **Florida Bearing capacity equations** show good agreement with the load tests for heterogeneous single and 2 layer (rock over sand) for Miami Limestone and Ocala Limestone, the **geomean (single layer) or median (two-layered)** are recommended to characterize the modulus based on rock dry unit weight.

- For footing settlement, the **stress-strain relationship and secant modulus** provides a good estimate. **Fenton & Griffiths Method (2002)** provides a good estimate of mean and differential settlement for single layer of rock; **Burmister's solution (1958)** provides a good estimate of two-layer settlement up to bearing failure of upper rock layer. The **BNWF** model uses the **geometric secant modulus (single layer)** and **harmonic secant modulus (two-layered)** with the **Bearing Capacity** to develop Winkler spring model.

- The **seismic shear tests** (Deliverable 5) shows great promise in characterizing the rock dry density and layering accurately near the ground surface.

- Further **Random FEM** analysis of layered system with a **high CV (1.0)** will be performed to evaluate the differential settlement for a **two-layer systems** (rock over sand) to improve the two-layered **BNWF** model.

Timeline

Deliverable # / Description as provided in the scope (included associated task #)	Completion Date
1.) Load Test 1 Site Investigation	1/2020
2.) Shallow Foundation – Load Test 1	10/2020
3.) Shallow Foundation – Load Test 2	5/2021
4.) Shallow Foundation – Load Test 3	9/2022
5.) Seismic Field Testing to develop Mass Properties of rock	10/2022
6a.) Draft final (Task 6)	12/2022
6b.) Closeout teleconference (Task 6)	12/2022
7.) Final report (Tasks 7)	12/2022

Acknowledgements

The researchers would like to thank:

- Florida Department of Transportation (FDOT) project managers: Rodrigo Herrera, David Horhota.
- District 6 Engineers: Matthew Gisondi, Adrian Viala, etc.
- State Material Office (SMO) folks: Dino Jameson, William Greenwood, Kyle Sheppard, Todd Britton, Bruce Swidarski, Travis Stevens, Tyler Adkins, Michael Risher, Kelly Carmona, Awilda Merced, Bay Pantoja, etc.
- H2R Drilling Crew for micro-piles installation on Cemex site
- R.W. Harris Drilling Crew for drilled shaft installation on SR84 and Bell sites
- Terracon and PSI Rock Coring Crew
- Powell Family Structures & Materials Laboratory: Scott Powell, Caitanya Jivan (CJ) Bhakti
- Weil Hall Structure Lab: Dr. Taylor Rawlinson
- BSI: Dr. Davidson
- Applied Foundation Testing (Loan of Hydraulic Jack and Concrete Mixer)
- Cemex Inc., Aggregates Division

The Research would not be completed without everyone's help!

Thank You!

Q & A