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

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Presentation Outline

- Project Background
- Project Objectives
- Overview of Miami Limestone (2 Load Tests)
  - Bi-linear Strength Envelope and Bearing Capacity Equations
  - Stress-strain Relationship
- Load Test 1: Cemex Site (Miami)
  - Site Investigation
  - Micro-piles Installation and Load Test
  - Load Test Results: Bearing Capacity
  - Load Test Results: Settlements
- Load Test 2: SR 84 Site (Davie, FL: Rock over Sand)
  - Site Investigation
  - Drilled Shaft Installation and Load Test
  - Load Test Results: Bearing Capacity
  - Load Test Results: Settlements
- Planned Load Test 3: Bell Florida (Ocala Limestone) in Gilchrist County
- Current Findings
- Timeline and Acknowledgements

Project Background

Phase I (FDOT BDV31-977-51):

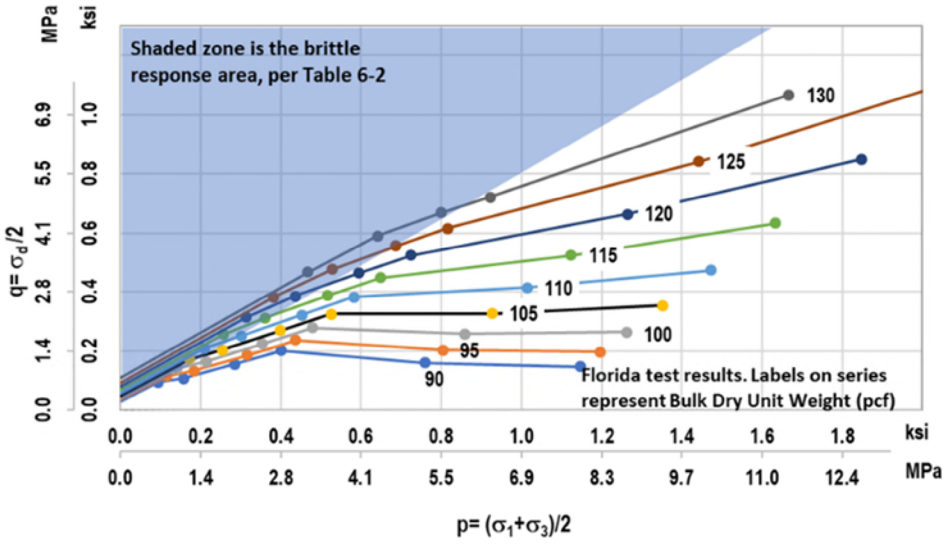
- Investigated the **strength envelope** of several Florida limestone formations near the ground surface – function of dry unit weight of rock and formation (Carbonate)
- Developed **Bearing Capacity Equation**, function of rock strength (homogeneous) and moduli (layered: rock over sand)

Phase II (FDOT BDV31-977-124):

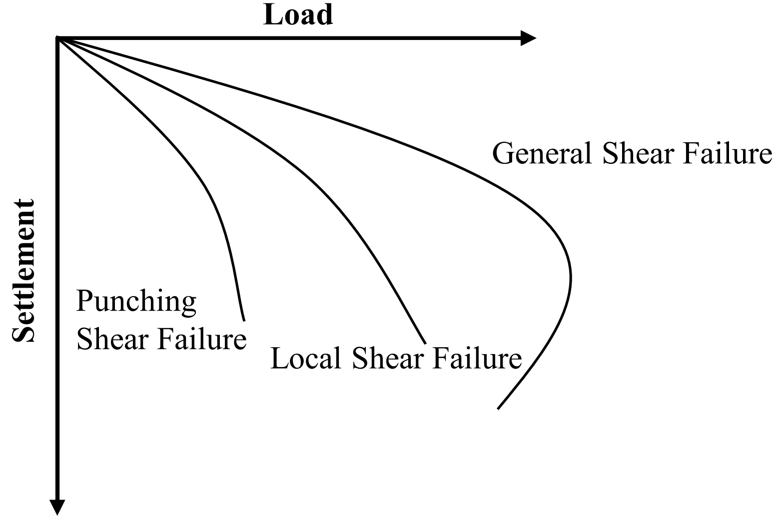
- **3 full scale field shallow foundation load tests** conducted to validate the **Bearing Capacity Equation** and predict the **load-settlement response** at different rock formations and layering (Hand Solution and Numerical Method)

Phase III (Planned):

- Implement the Bearing Capacity Equation and the load-settlement model in **FB-Multipier** for shallow foundation design in Florida



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**.
- Planned Load Test 3: Bell Site, **Ocala Limestone**.

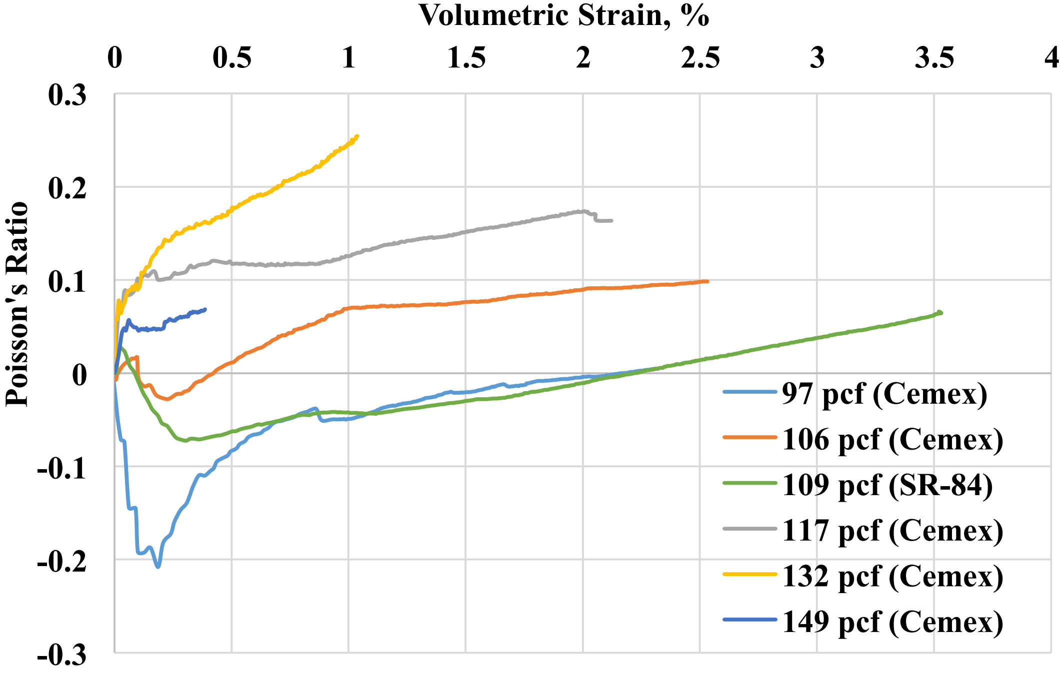
II. Measure and Predict Load versus Settlement for shallow foundation on homogeneous & heterogeneous (rock over sand) Florida Limestone (Deliverable 6 to 7).

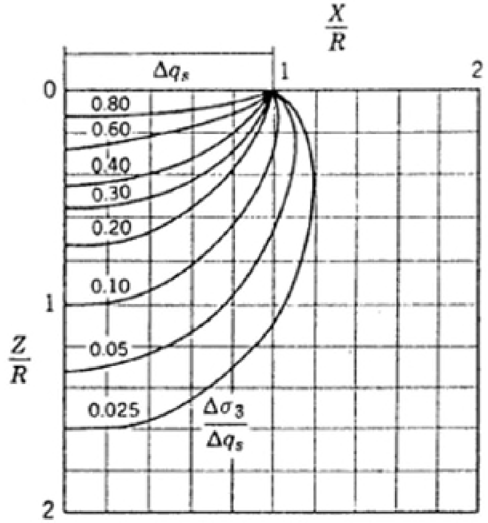
- Heterogeneous –Single Layer: **Fenton & Griffiths Method (2002)**
- Heterogeneous Two Layer (Rock over sand): **Burmister Method (1958)**

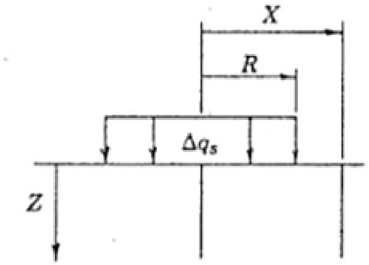
III. For I & II -Assess rock strength, Young's modulus (Secant - E_{secant}) and rock unit weight from laboratory tests (**q_u, q_{dt} and triaxial tests**, Deliverable 1) and in-situ methods – a **newer seismic method** (Deliverable 5)

Overview of Miami Limestone – Poisson Ratio, Lateral Stress

- **Low or Negative Poisson's Ratio** was found for most dry density rock samples in triaxial tests,
- **Due to low Poisson's Ratio, Low Confining Stress ($\Delta\sigma_3$)** was observed from FEM and Associated Stress Path for Miami Limestone.
- 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**

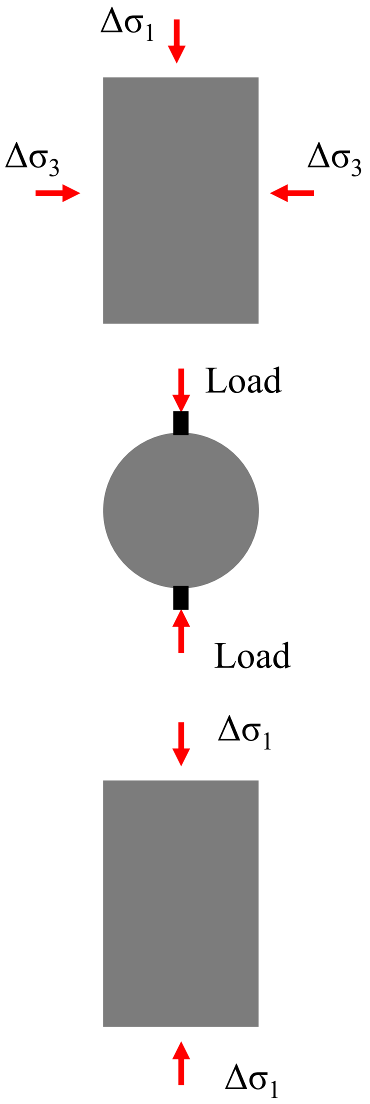


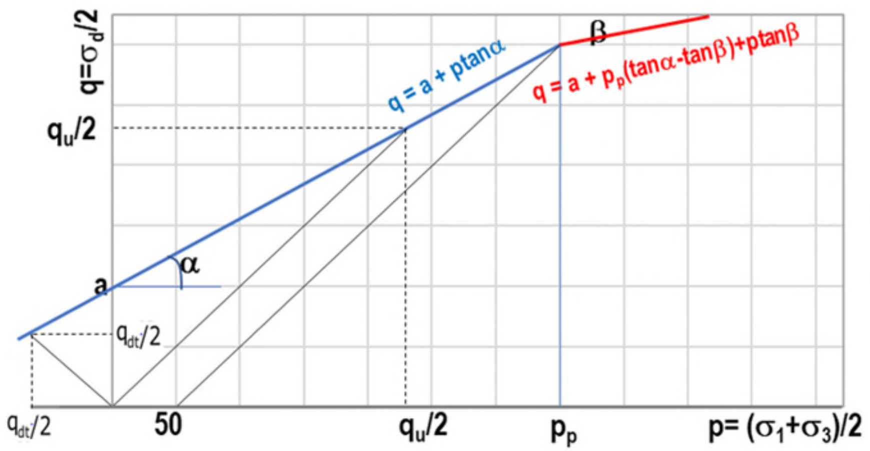




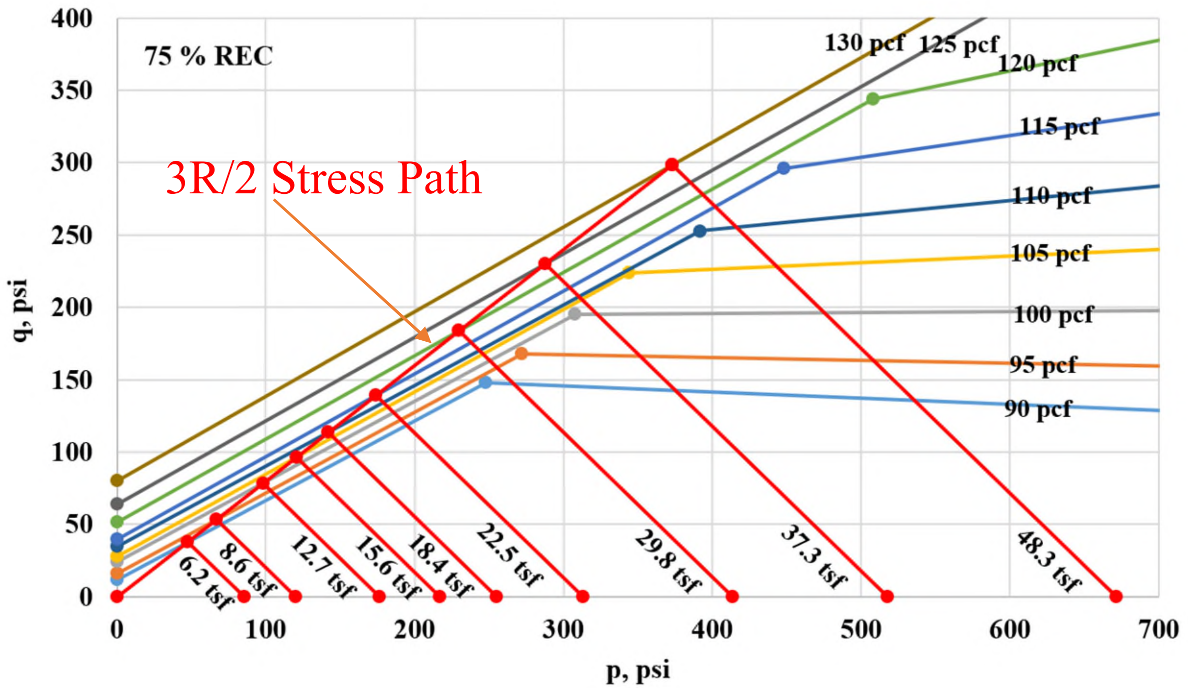
Overview: Strength Envelope and Bearing Capacity

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

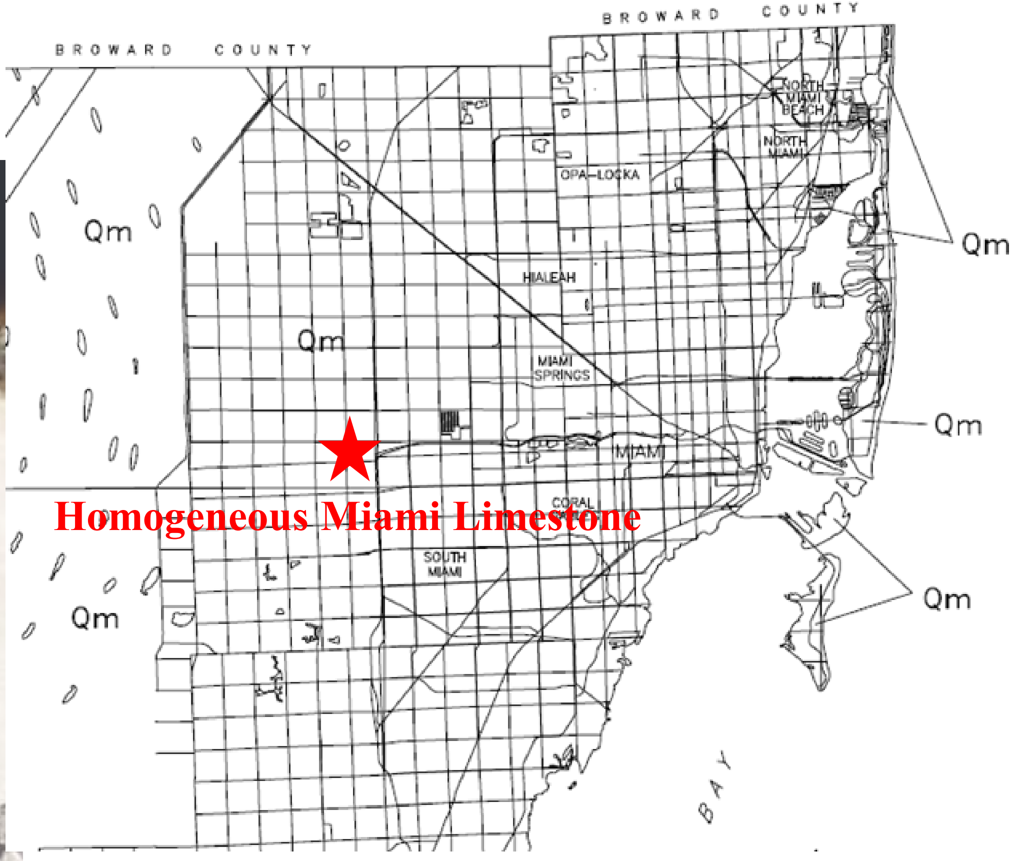
$$Q_{u1} = n*c*N_c + q*N_q$$

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

Florida Bearing Capacity Equations

Load Test 1: Cemex Site, Miami Site Investigation

25°46'59.0"N 80°26'25.6"W



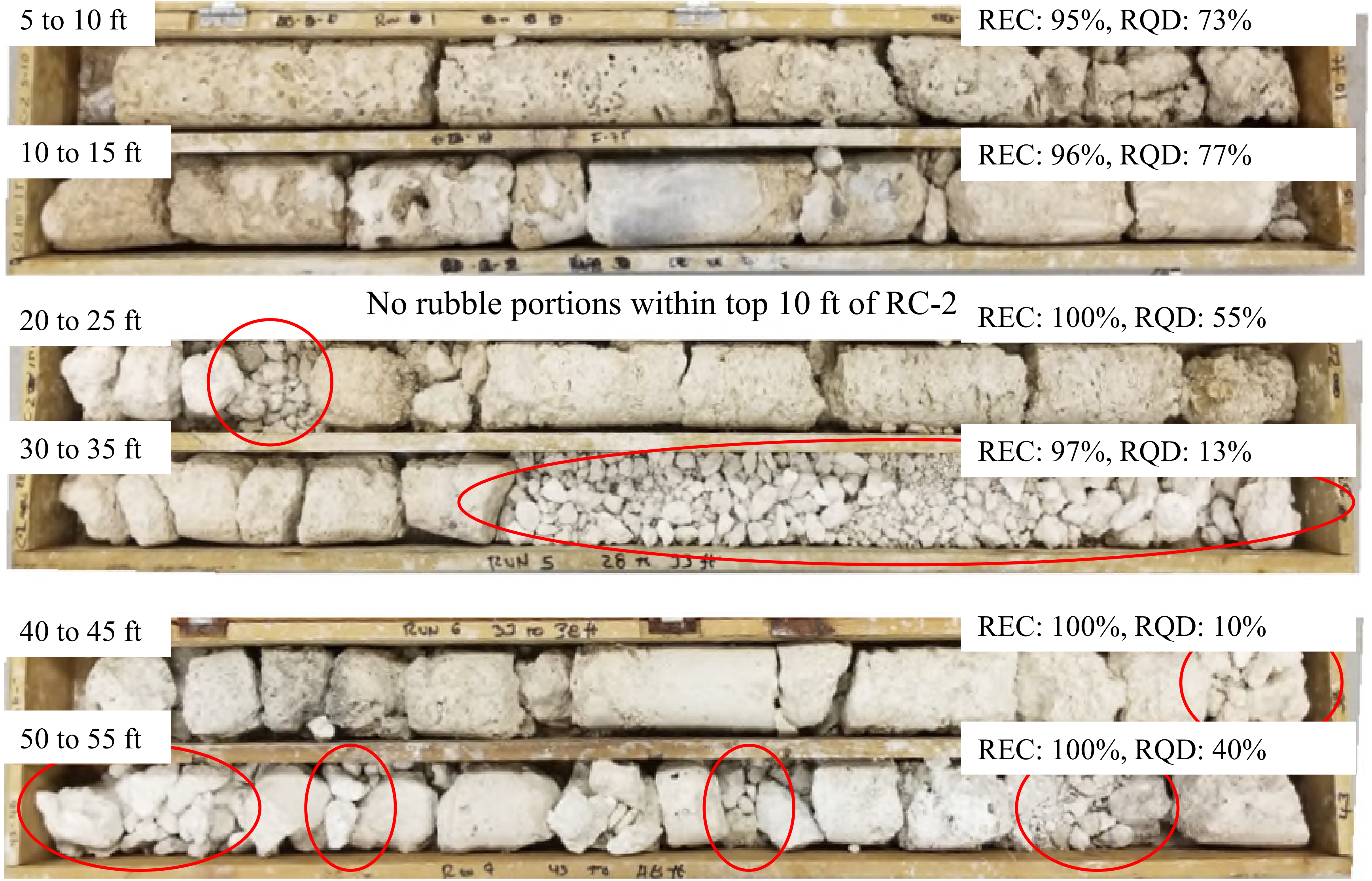
Geologic Map of Dade County, Florida

Load Test 1: Cemex Site, Site Investigation

RC-2 (Terracon)

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



Adjusted-REC:

95%

96%

94%

32%

86%

63%

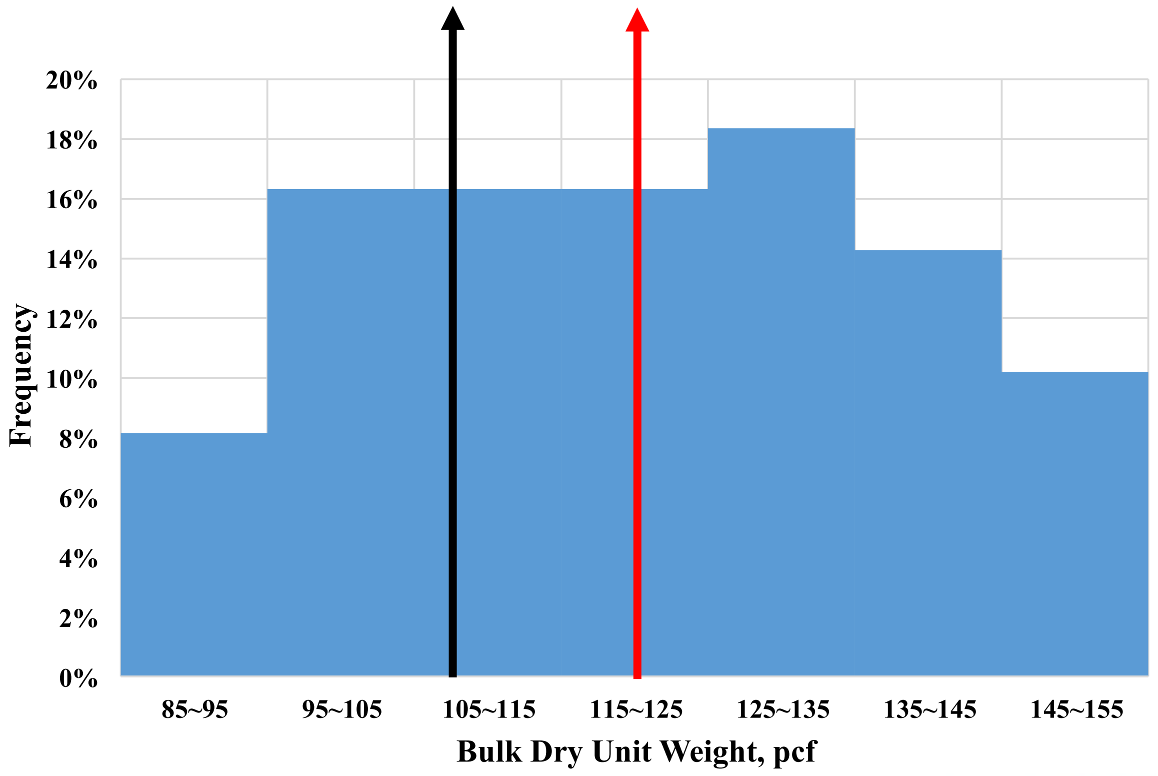
Load Test 1: Cemex Site, Site Investigation

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

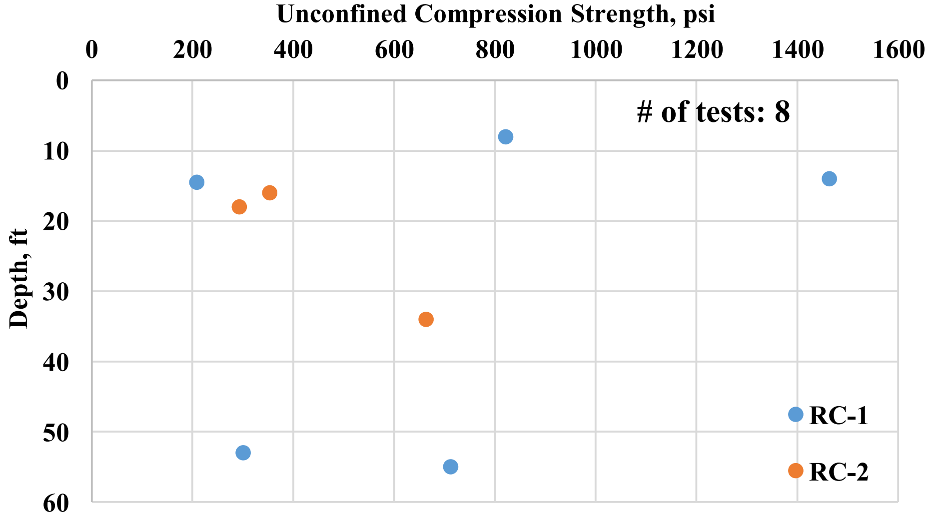
Geomean at Top 2B Depth at RC-2 (Footing Location): 107 pcf

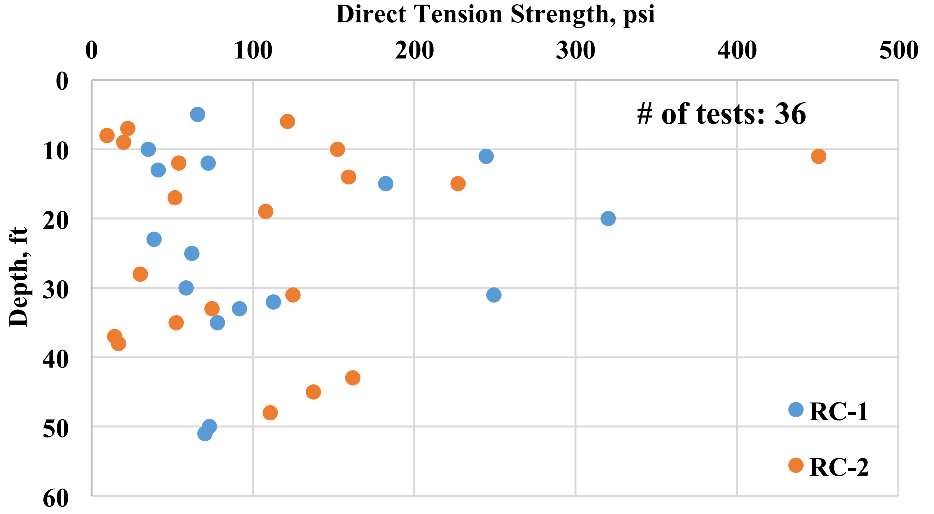
Mean at Cemex Site: 119.3 pcf
Median at Cemex Site: 119.6 pcf



Load Test 1: Cemex Site, Site Investigation

Strength Assessment



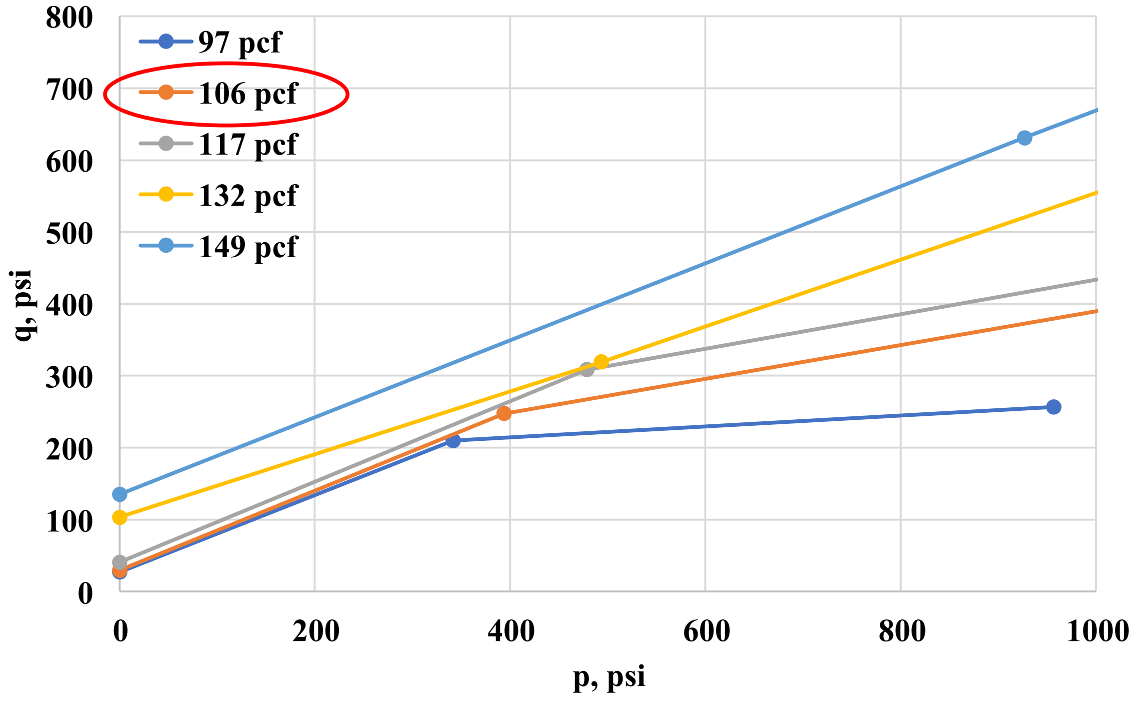


Strength (q_u & q_{dt}) versus Depth for Cemex Site ($q_{dt} = 0.7 \times qt$, Perras, M. A., etc., 2014)

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

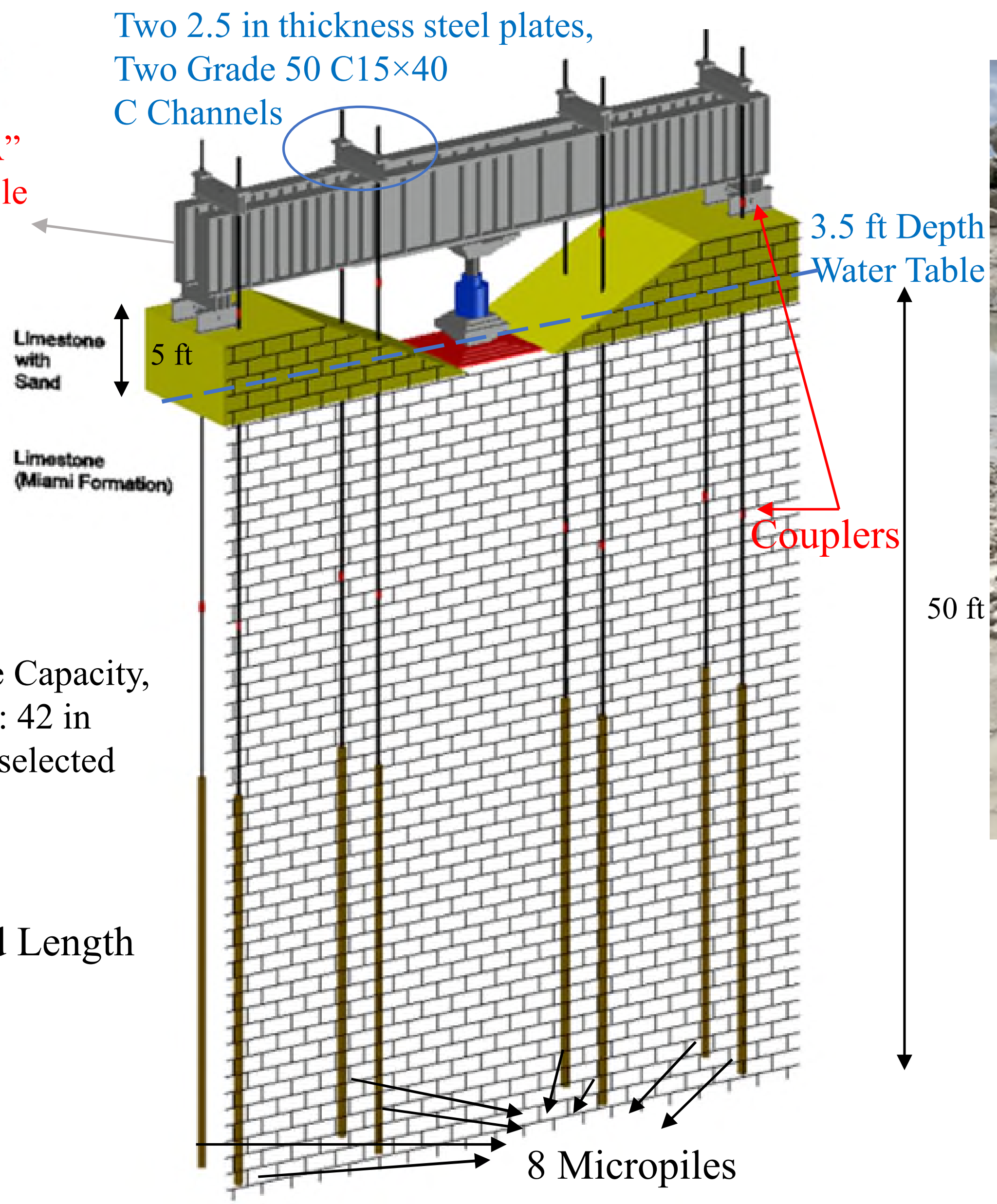
COUNT	9
MEDIAN	100
MEAN	108
GEOMEAN	107
std	20



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

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

Two 40 ft long Acosta type “A” Girders, Capable of 900 tons combined



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

25 ft grouted Length

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 (H2R) and Load Test

Drill with Polymer Mud System – Tight Holes

Mix with ASTM C494 Type D Retarder admixture and ASTM C494 Type F Superplasticizer admixture

ASTM C845 Type K cement

Compressive Strength of Cement Specimens

Sample ID	Break Type	Set Time, Days	Compressive Strength, psi
1st	III	21	4497.3
2nd	III	14	2911.2
3rd	II	14	6731.4
4th	I	15	6166.4
5th	I	14	4483.1
6th	I	7	6219.2
7th	III	8	3687.8
8th	II	7	5590.5

6 in hole/2.25 in GR80 threaded rods = 2.67

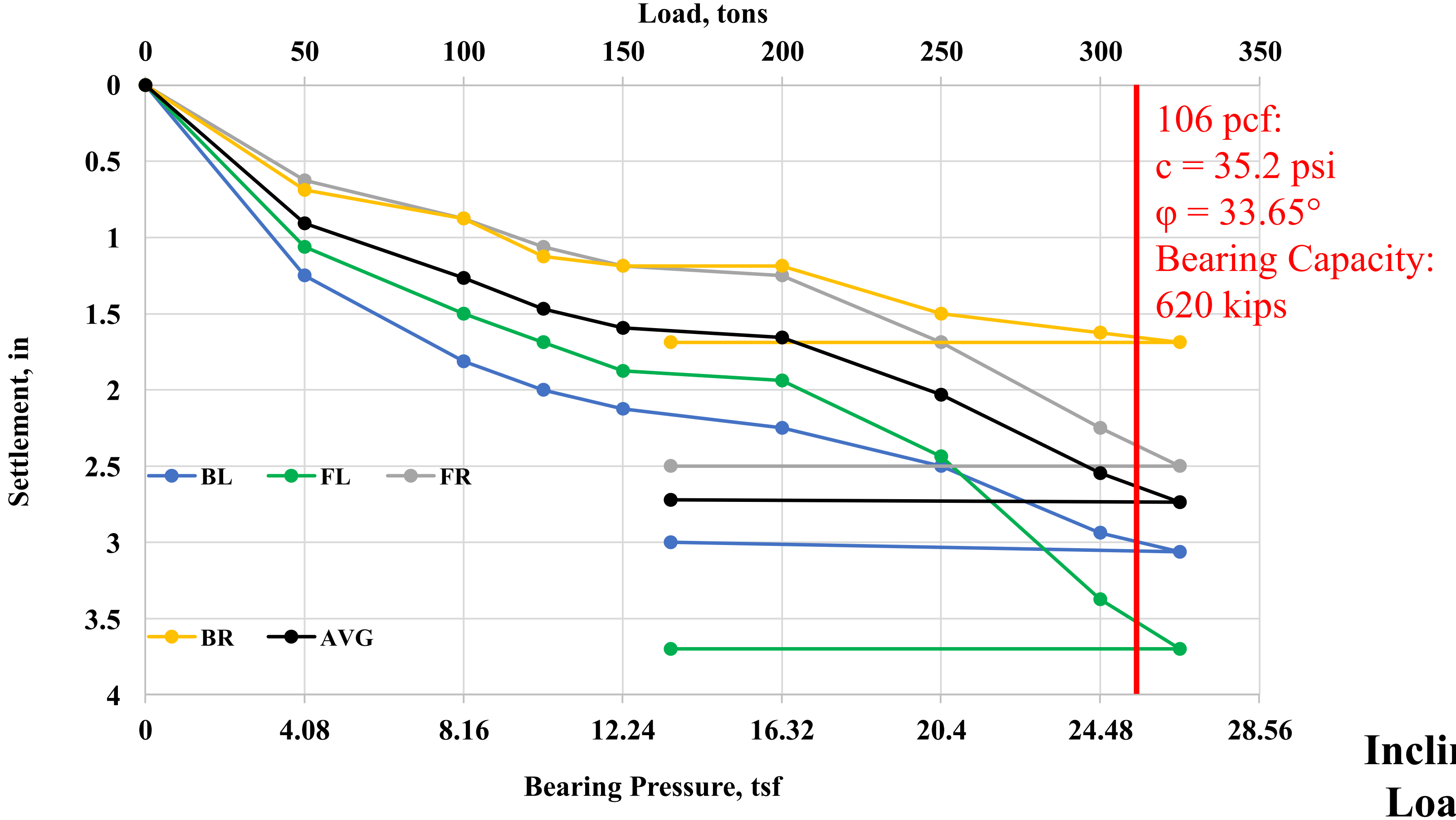
Max q_u = 1463.3 psi

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



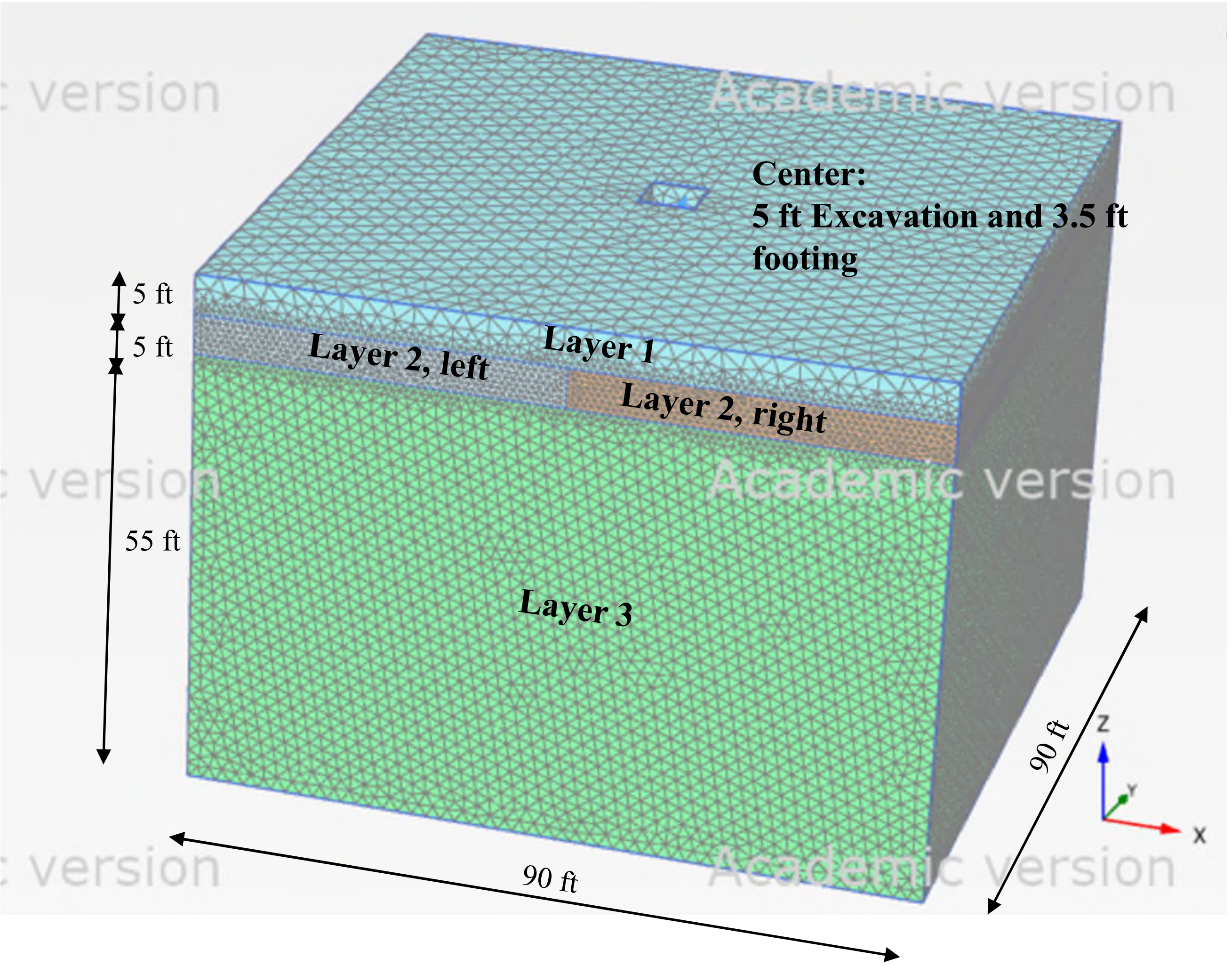
Measured and Predicted: Bearing Capacity for Cemex Site

Settlement Measurement: Tape with Total Station

Inclination of the Loading Jack and Load Cell, Load Test ends 2.74 in settlement at 650 kips

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

FEM – Plaxis 3D

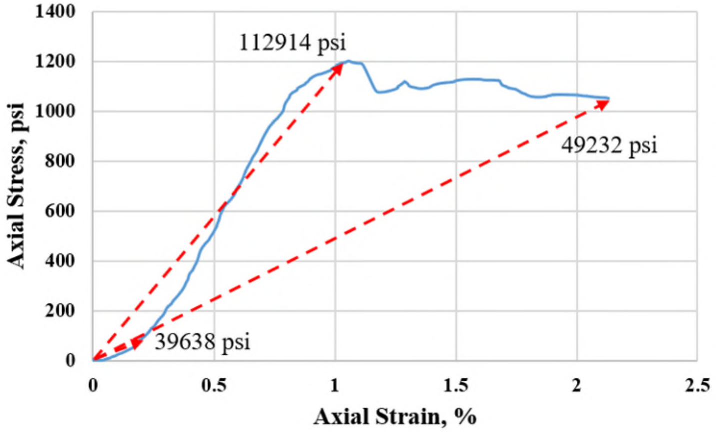


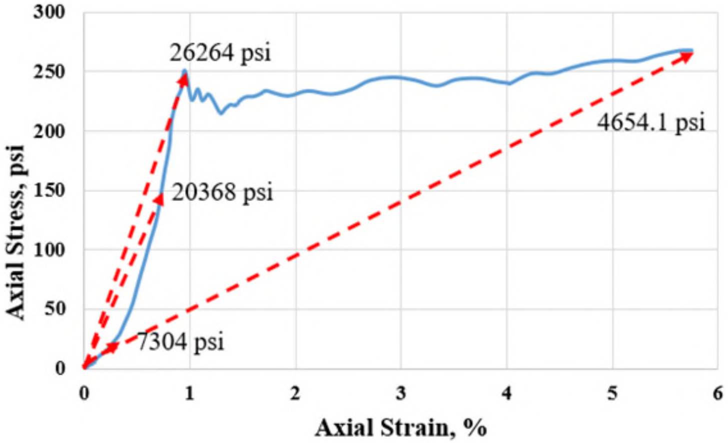
Material Model, Strength and Stiffness Parameters of Different Layer

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



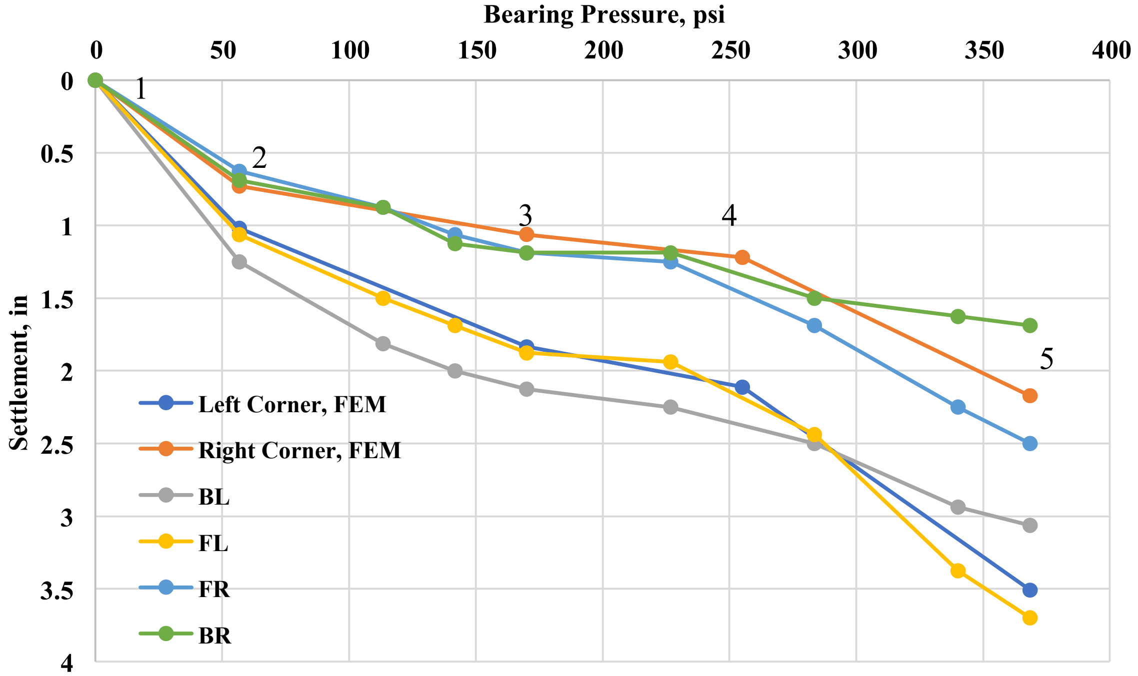


Stress-Strain Curve of Unconfined Compression Test

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

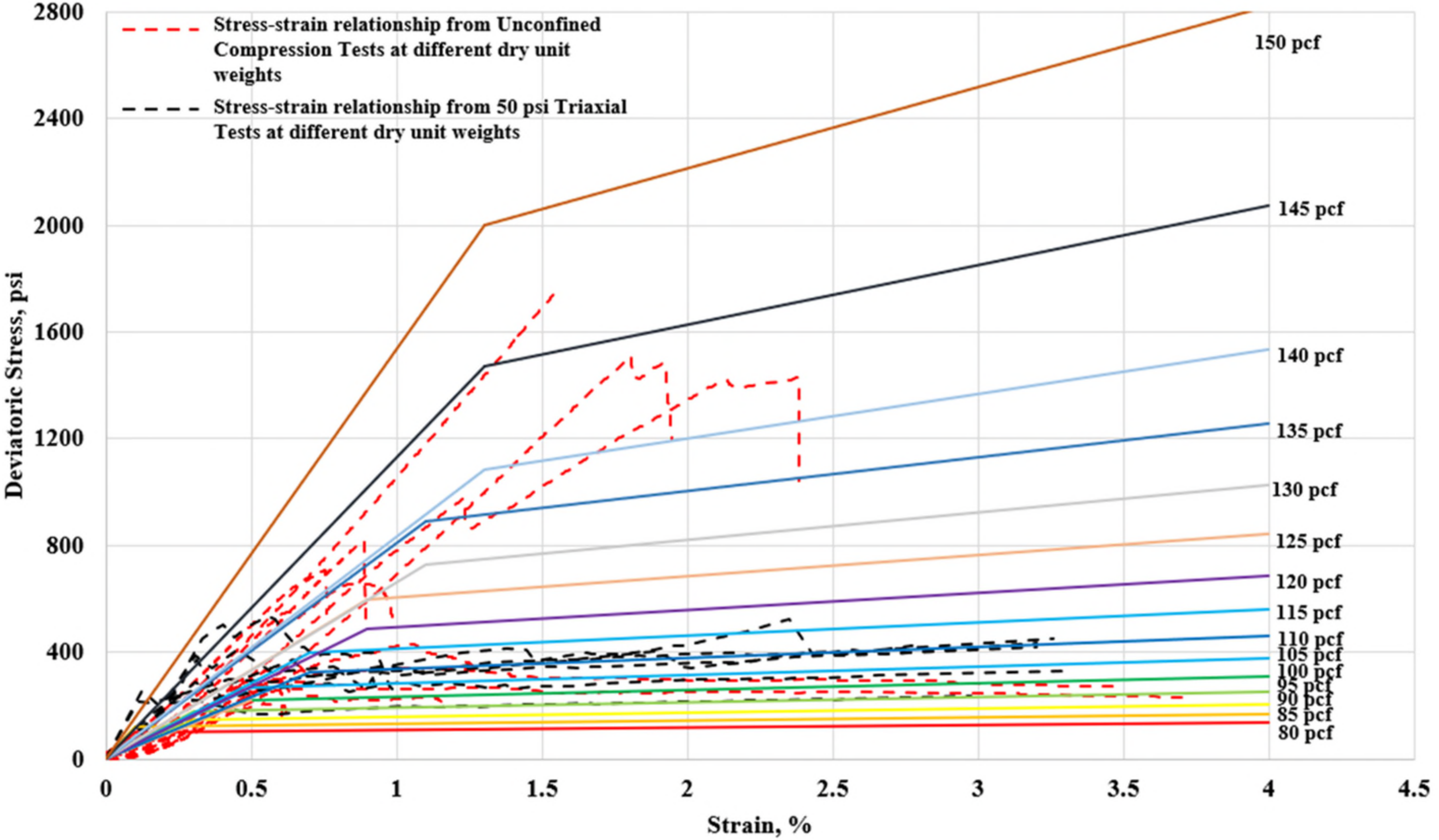
Loading Step	Load, tons	Layer 2							
		Left: 90 pcf				Right: 97 pcf			
		σ1', psi	σ3', psi	Elastic Modulus, psi	Poisson's Ratio	σ1', psi	σ3', psi	Elastic Modulus, psi	Poisson's Ratio
1	0	2.90	2.47	1000	0.01	2.56	2.17	1500	0.05
2	50	19.71	-0.31	1000	0.01	24.74	-0.40	1500	0.05
3	150	71.89	7.26	6500	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



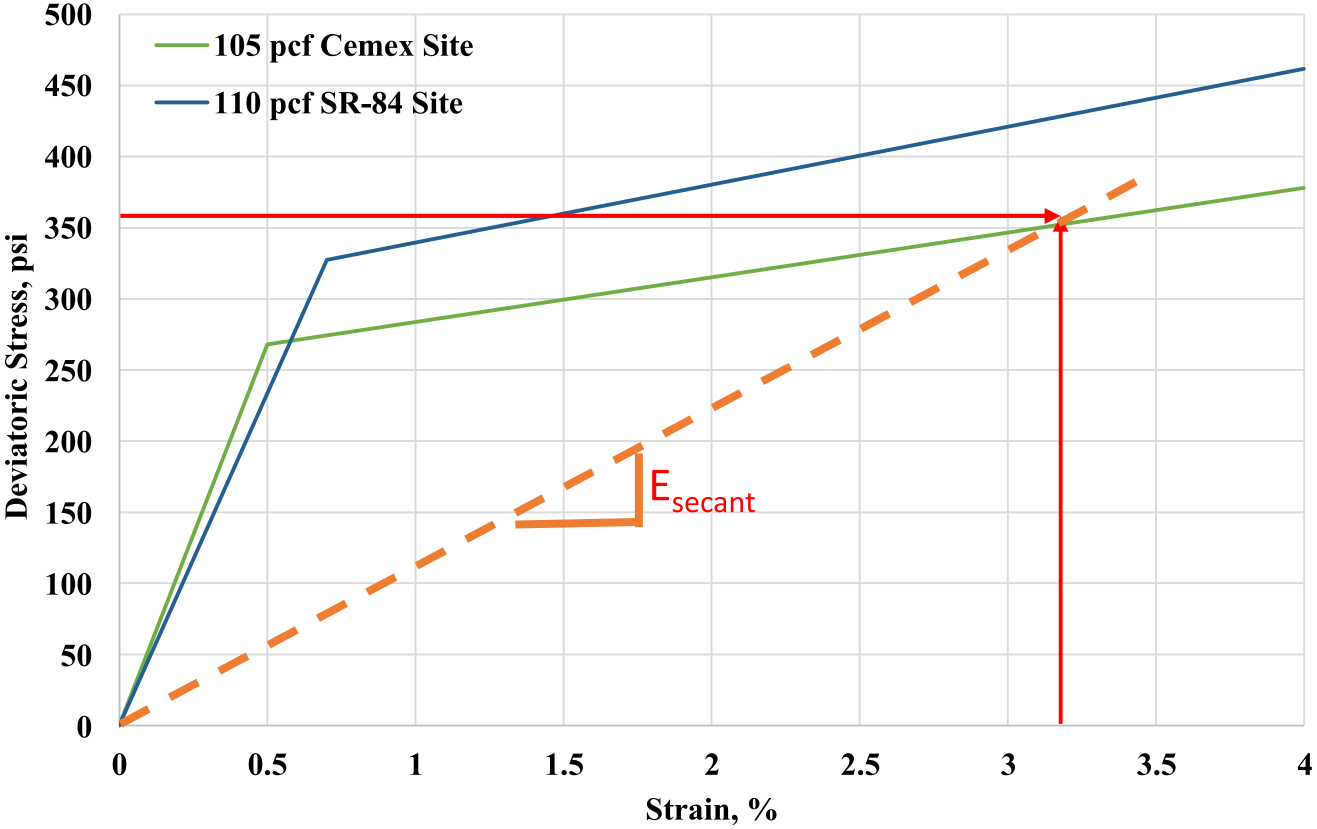
Measured and FEM: Load-Settlement Response

Estimated Settlement from Hand Solution – Assume Bilinear Stress vs. Strain

It's recommended that the stress-strain relationship of unconfined compression test and lower confining triaxial test (i.e., 50 psi) be used to characterize the stress-strain relationship for rock mass (function of dry unit weight) and characterized as Bilinear.



Estimation of Secant Young's Modulus from Stress-Strain Relationship



For Cemex Site (105 pcf, Homogeneous Miami Limestone):
Compute Bearing Capacity = 352 psi
Using dry unit weight Bilinear Stress vs Strain, and BC: finding the vertical strain = 3.17%
Secant Young's Modulus, E_{secant} = 11104 psi

Influence of Heterogeneity of Modulus on Settlement - Fenton & Griffiths (2002)

For Florida Intermediate Geomaterials:

$$q_u = 1.7265 \times q_{dt}^{1.2167}$$

Coefficient of Variation:

$$CV = \frac{\sigma}{\mu}$$

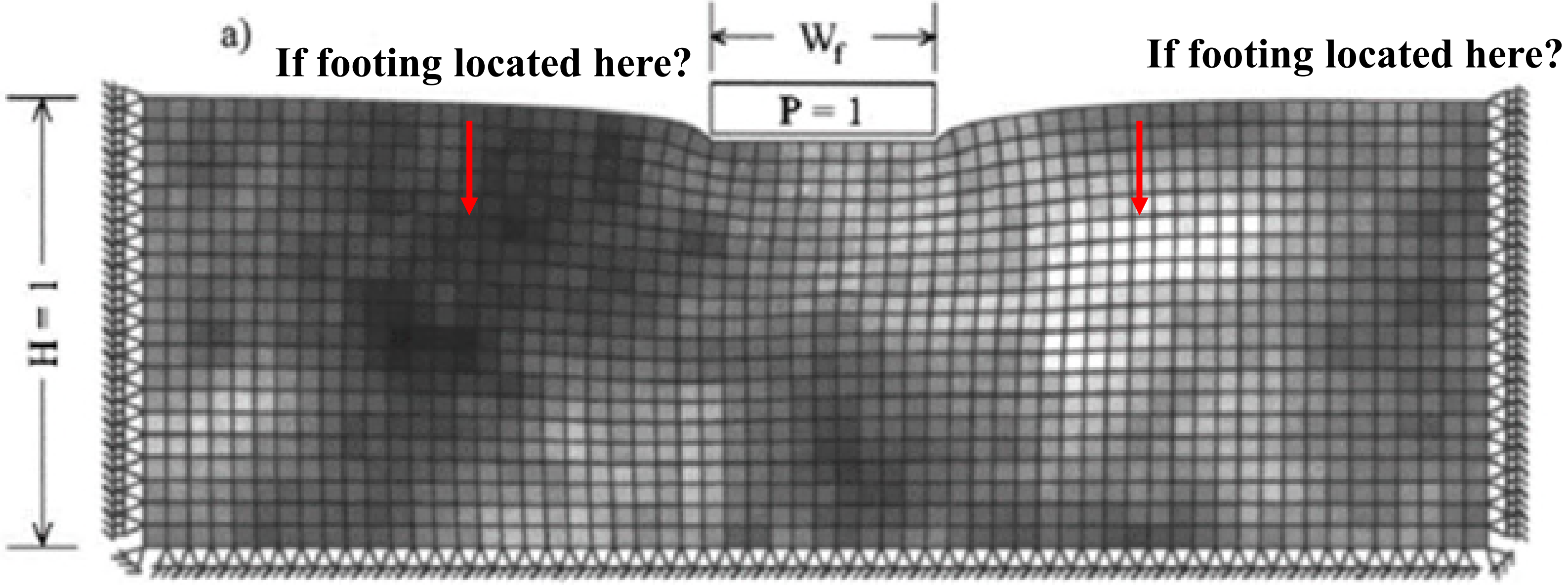
Where,

σ = standard deviation

μ = mean

For Cemex Site (Load Test 1), CV = 1.05

For SR 84 Site (Load Test 2), CV = 1.06



Random Field/FEM Representation of a Single Footing

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

Deterministic Settlement:

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

Geometric Mean Modulus

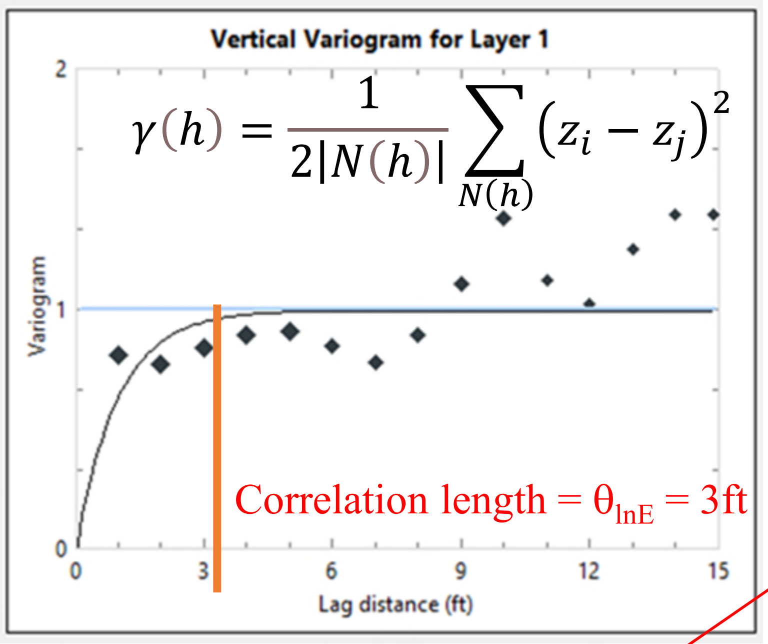
Where,

μ = Poisson's Ratio

Δq_s = Bearing Pressure

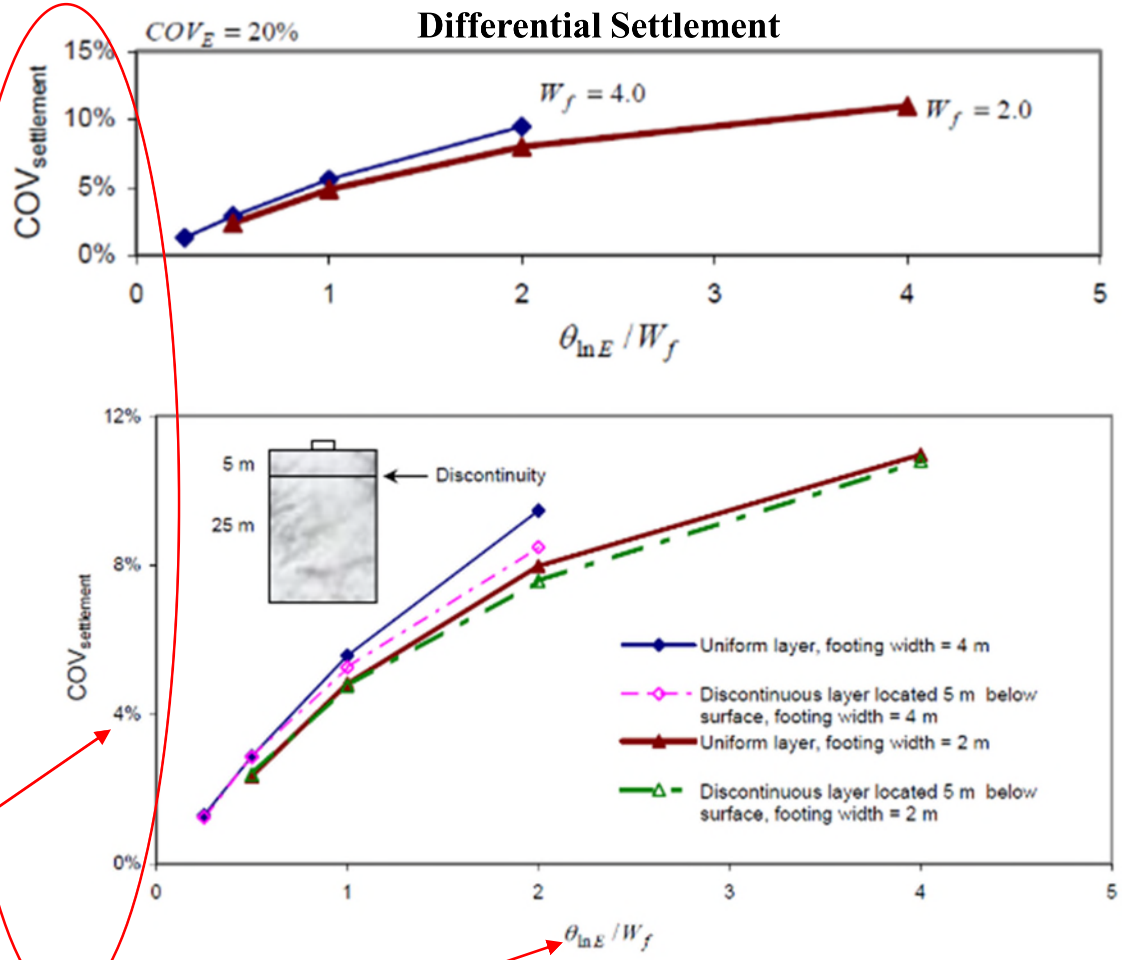
Influence of Heterogeneity on Differential Settlement

Number of data points: 68



$$\gamma(h) = \frac{1}{2|N(h)|}\sum_{N(h)}(z_i - z_j)^2$$

Variogram for Miami Limestone, Geostat, BSI



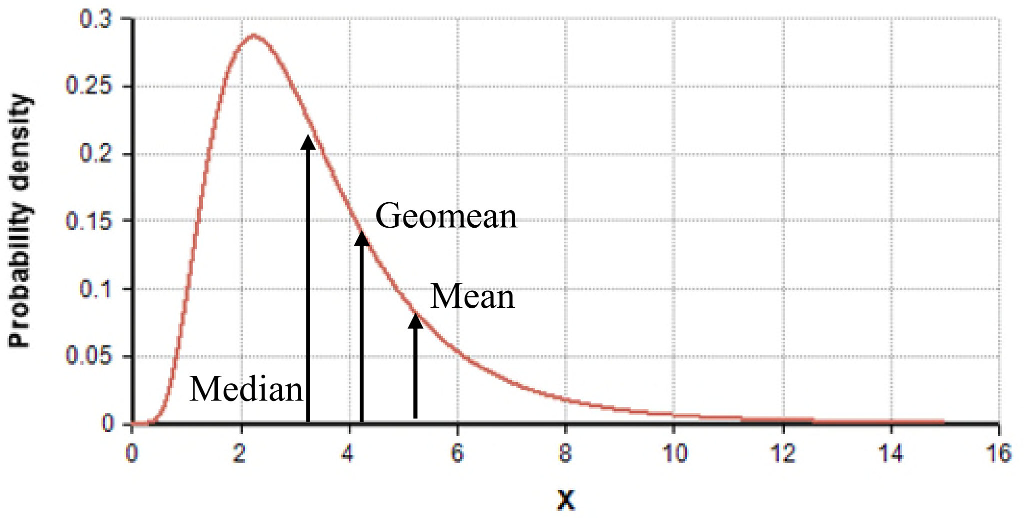
θ_{lnE}: Scale of Fluctuation; W_f: Footing Width.

Coefficient of Variation of Settlement with Varying θ_{lnE}/W_f: (a) for a Single Soil Layer Profile; (b) for a Two Soil Layers Profile. (Y.L. Kuo, etc., 2004)

Note as Width of Footing Increases compared to Correlation Length, Differential Decreases.

Mass Modulus of Heterogenous Rock

Variability of Florida Limestone



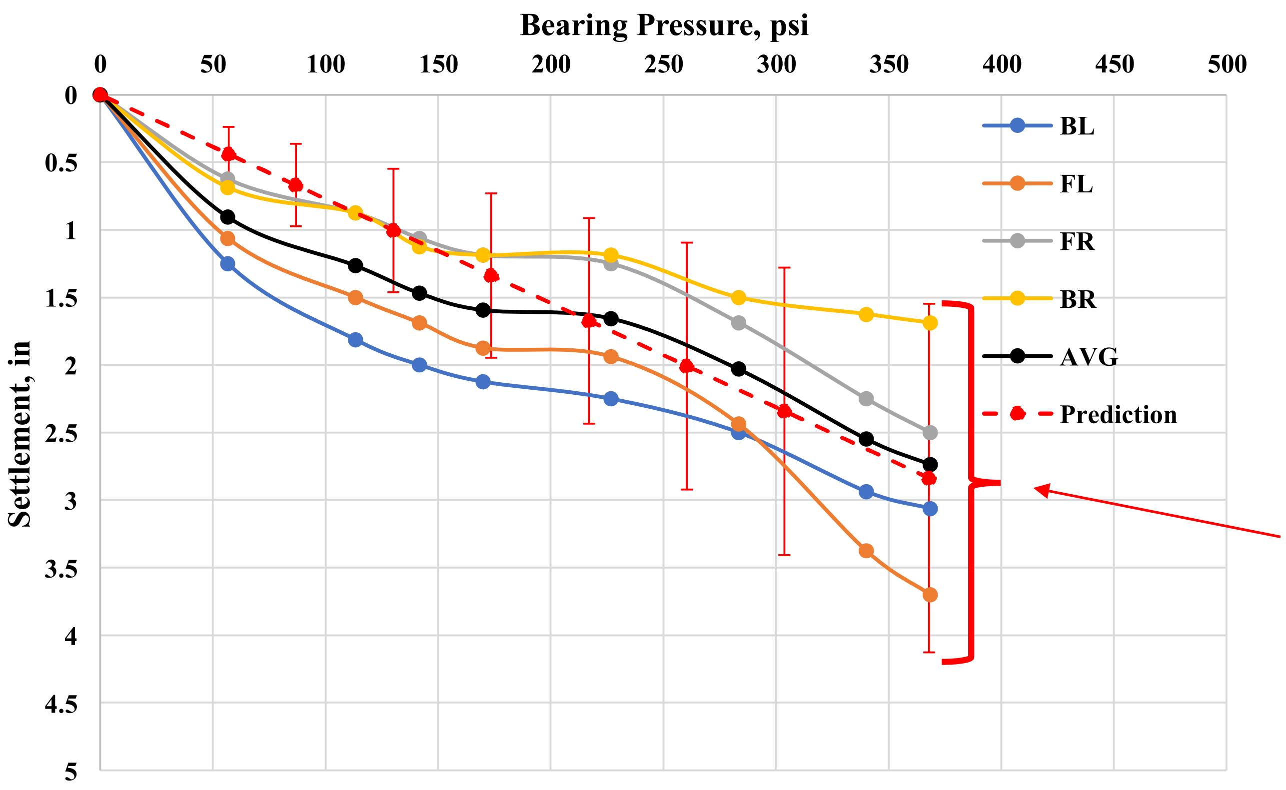
- Strength, Dry Density and Modulii of Florida Limestone is lognormally distributed which may be characterized with Mean, Median and Geometric mean;

- 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) – Hand & Linear Solution



Measured and Fenton & Griffiths method: Load-Settlement Response

Settlement at BC (q_s):

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

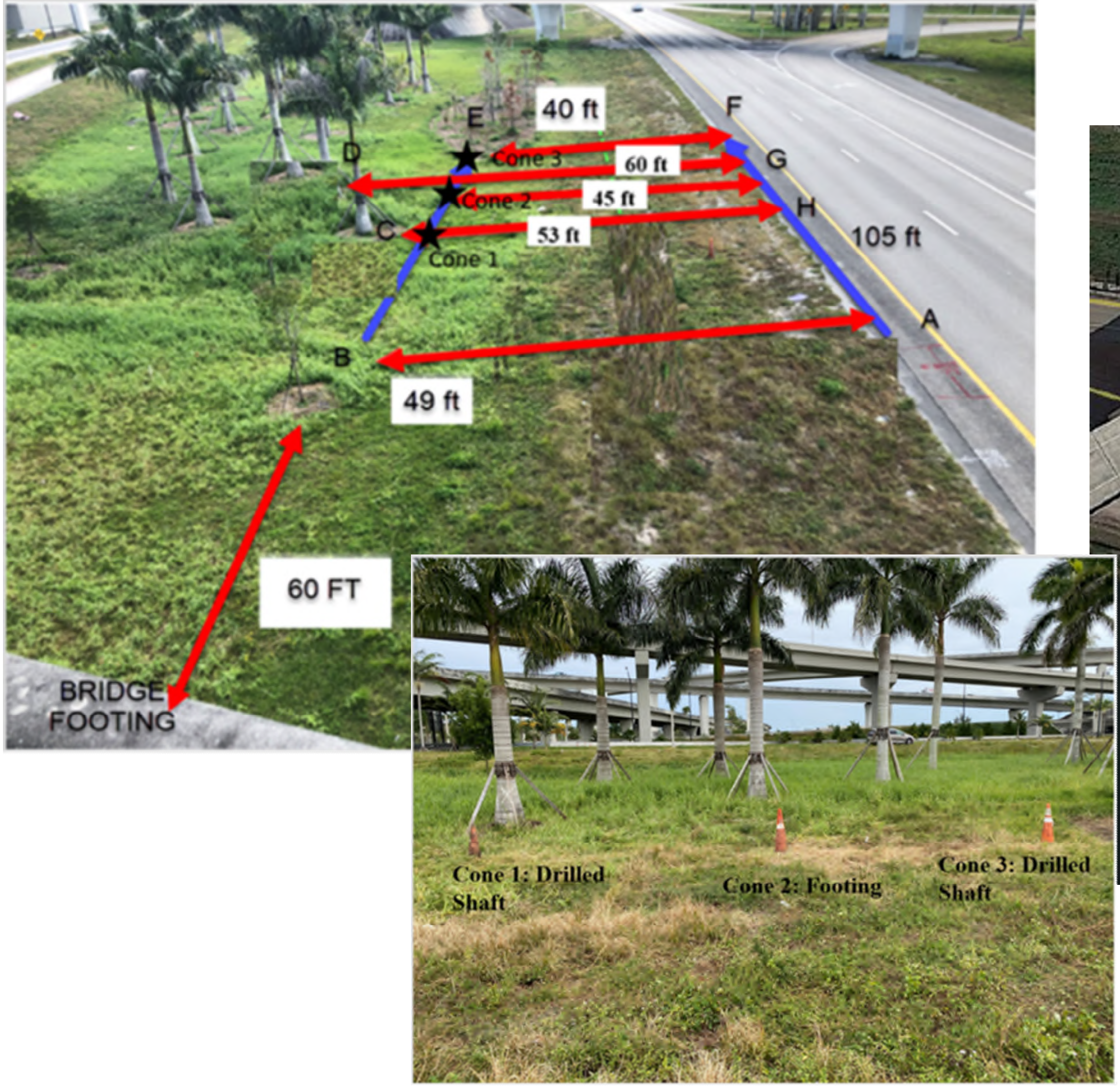
Where,

μ = Poisson's Rati

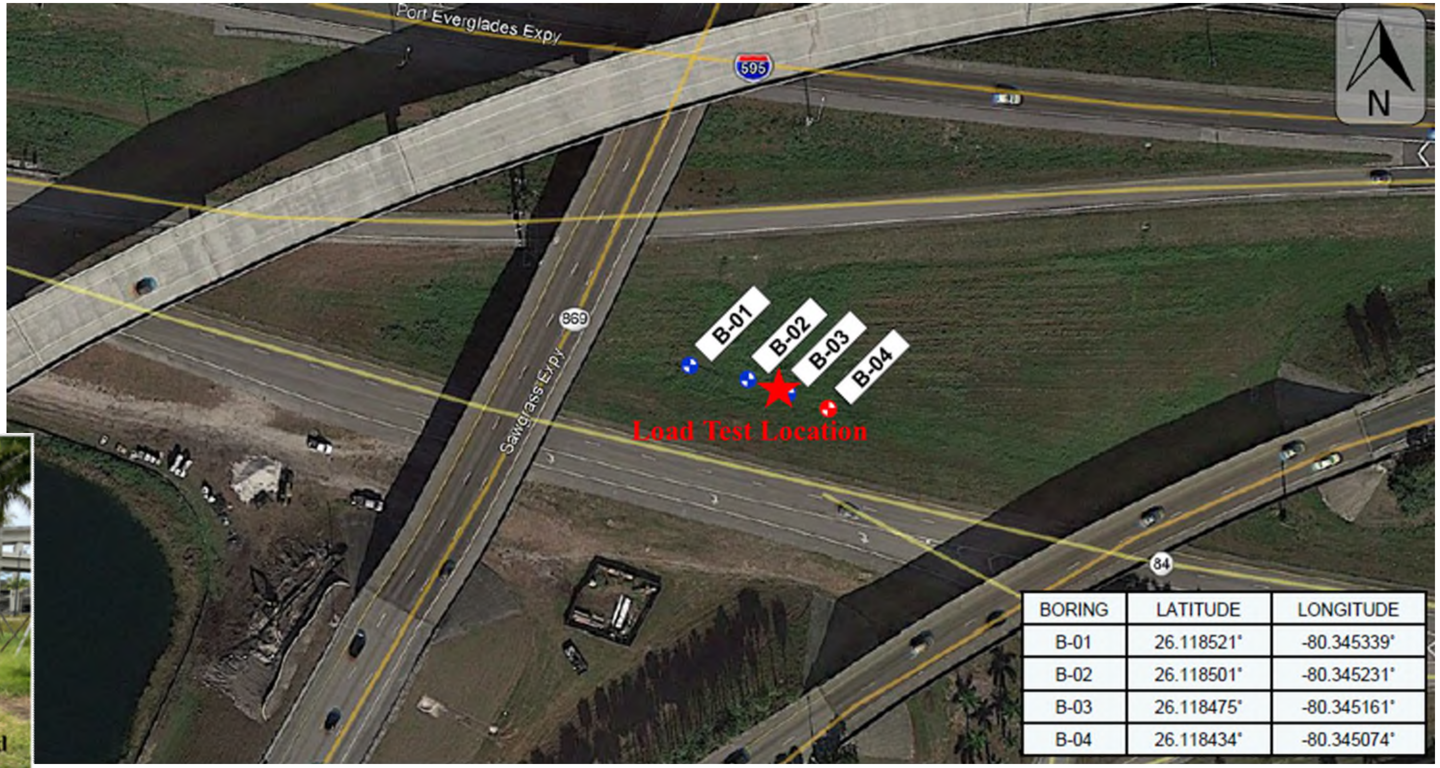
Δq_s = Bearing Pressure

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
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)

B1 3' to 13', Miami Limestone

B1 33' to 43', Fort Thompson Limestone

B2 3' to 13', Miami Limestone

B2 33' to 43', Fort Thompson Limestone

B3 3' to 13', Miami Limestone

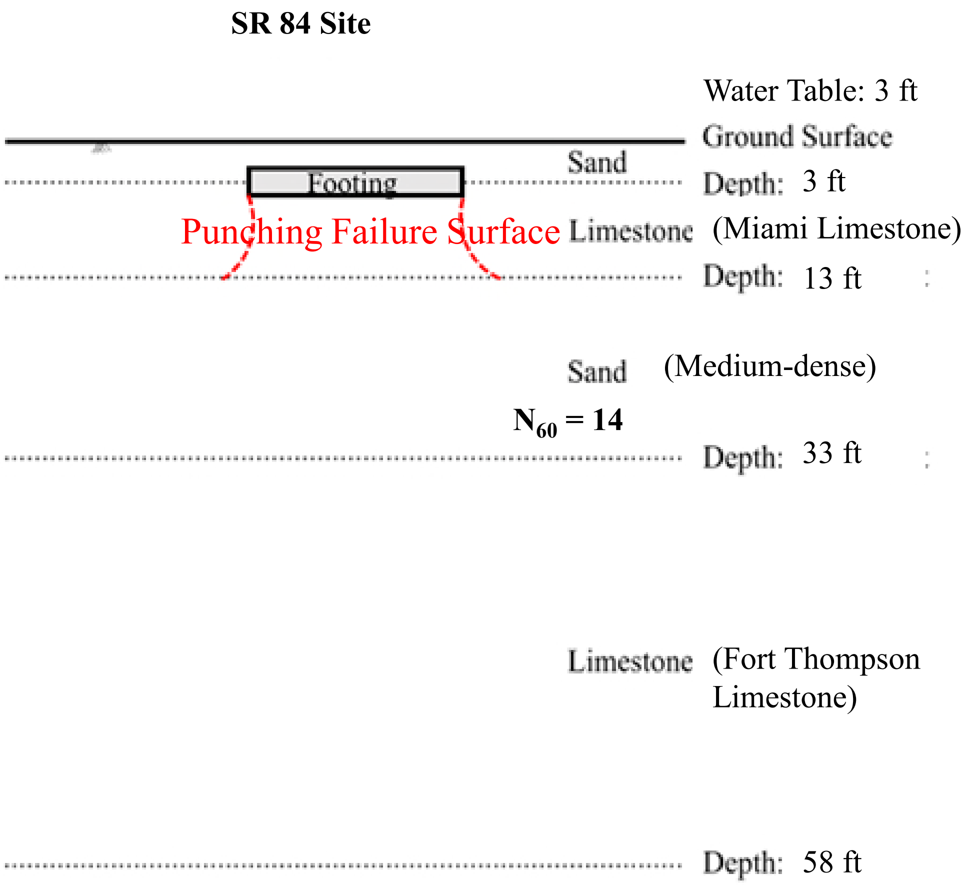
B3 33' to 43', Fort Thompson Limestone

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

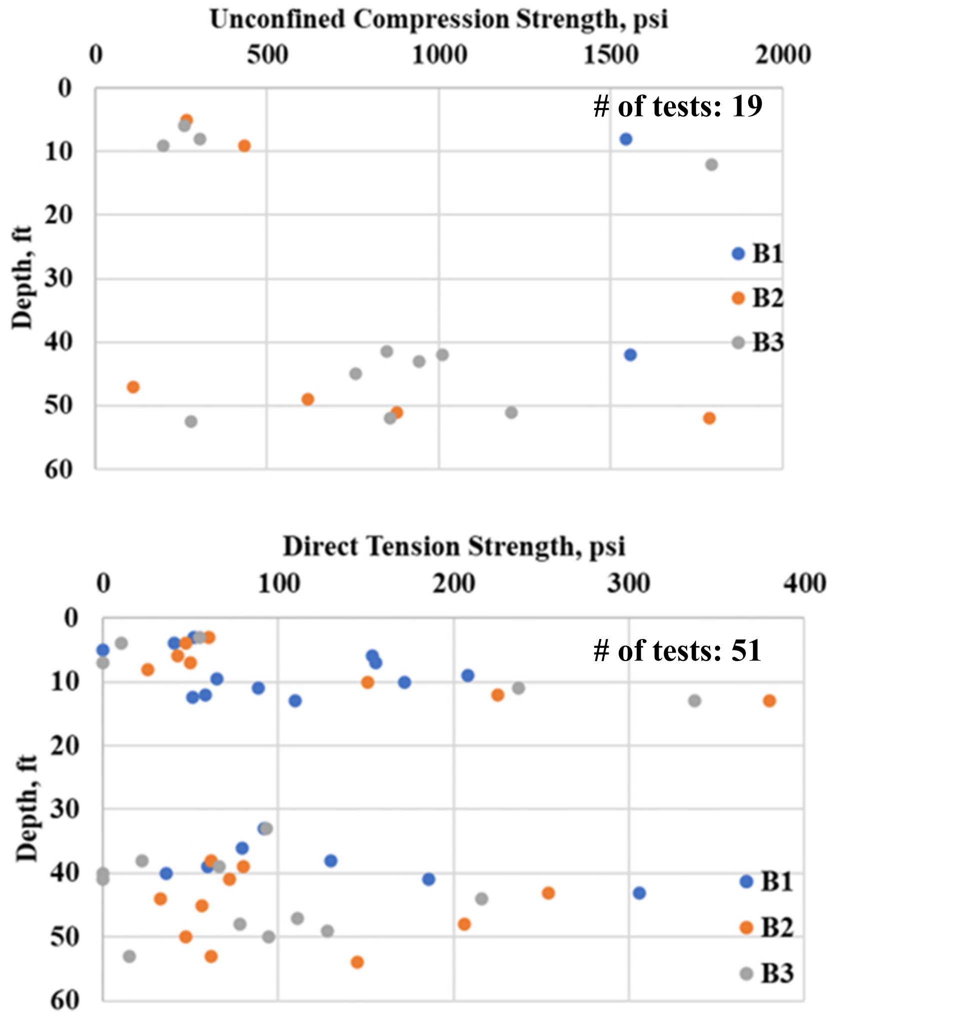
Load Test 2: SR 84 Site, Site Investigation

Strength Assessment and Spatial Variability Evaluation

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



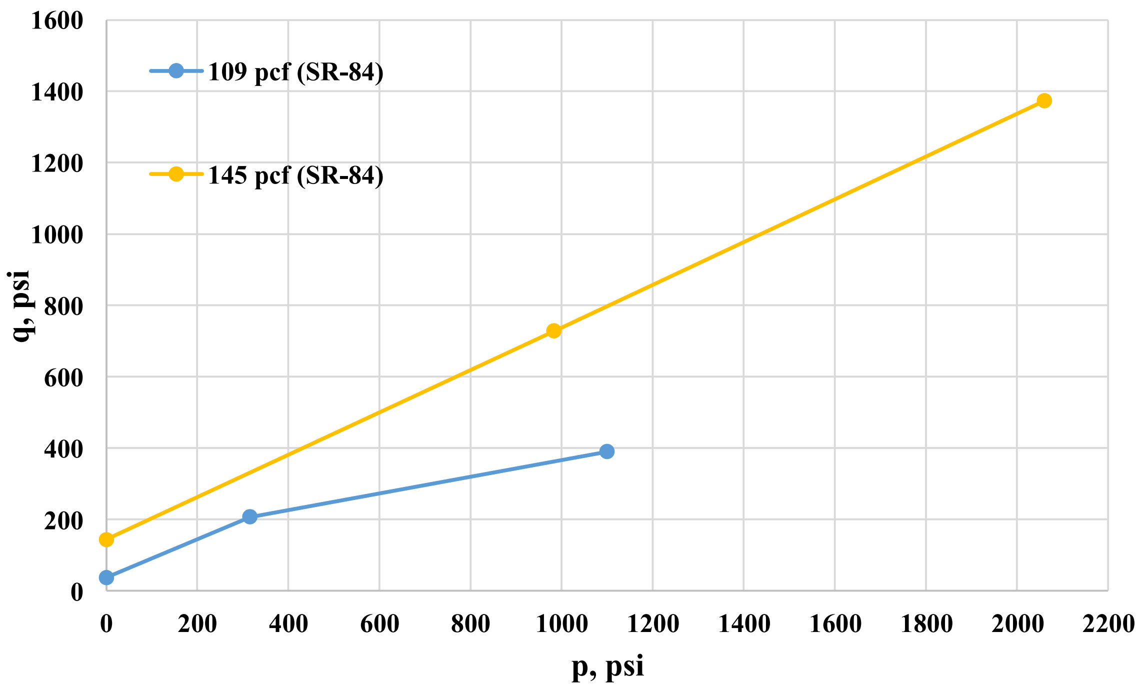
Subsurface layering based on rock coring and SPT



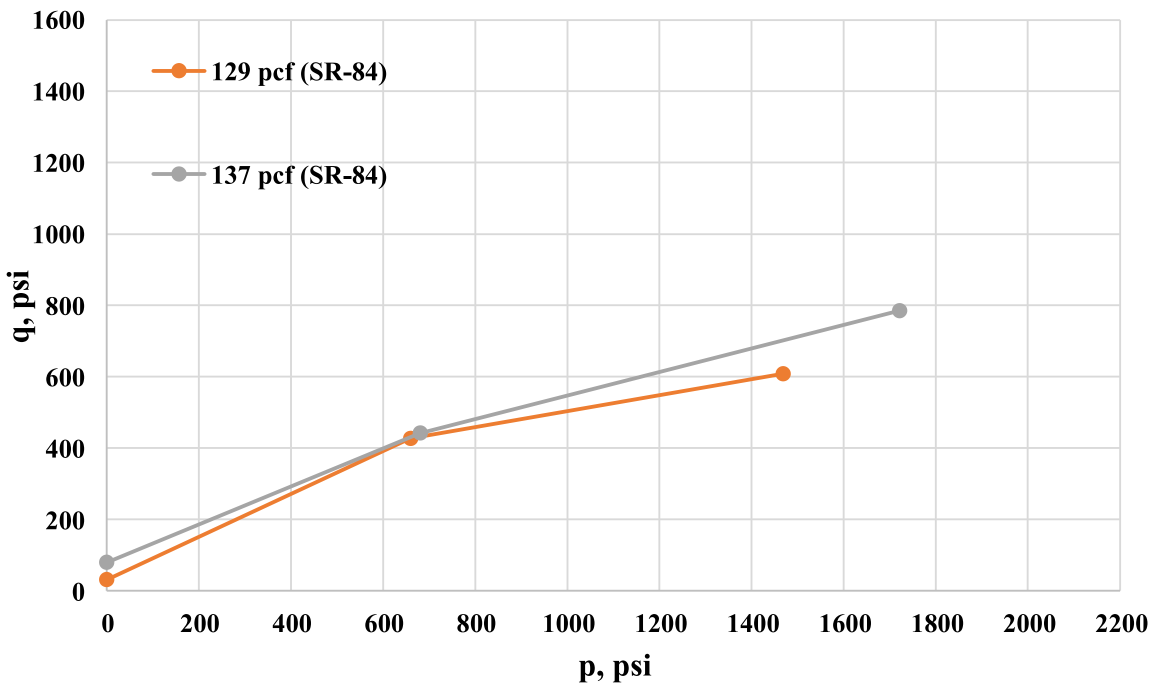
Strength (q_u & q_{dt}) versus Depth for SR 84 Site (q_{dt} = 0.7 × qt, Perras, M. A., etc., 2014)

Load Test 2: SR 84 Site, Site Investigation

Strength Envelope



Strength Envelope for Miami Limestone at SR 84 Site



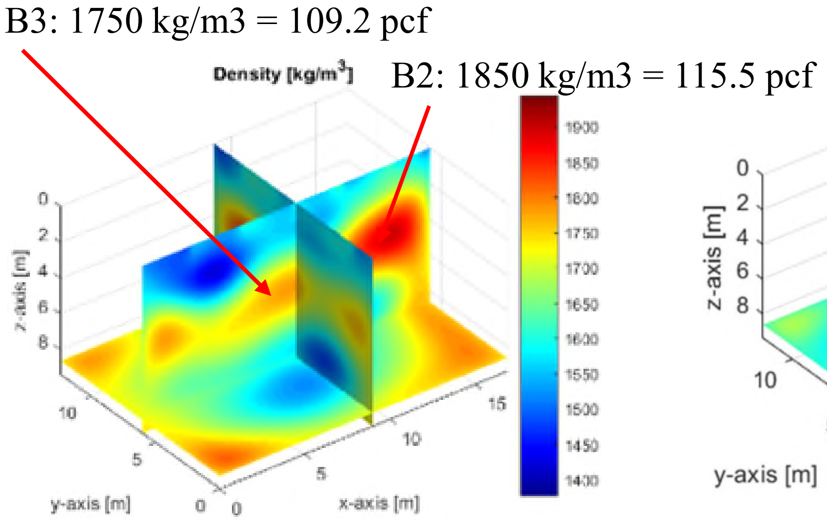
Strength Envelope for Fort Thompson Limestone at SR 84 Site

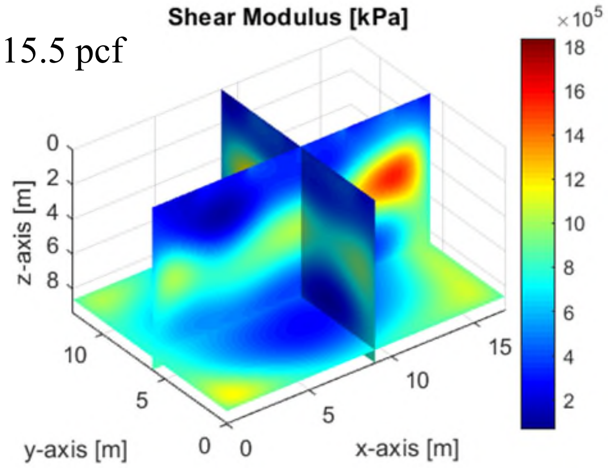
Load Test 2: SR 84 Site Properties & Seismic Shear

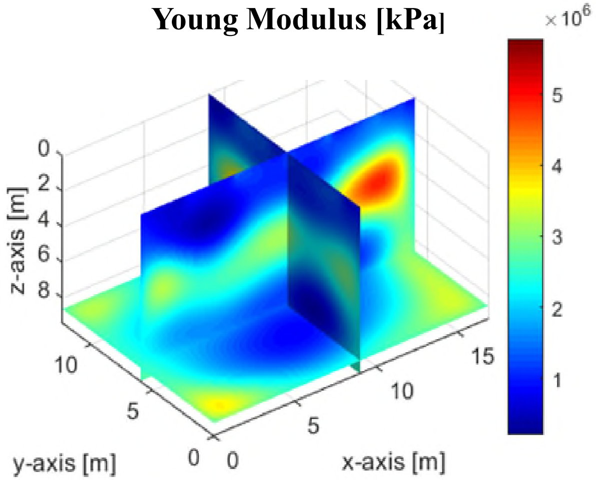
Footing Location: B-3

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





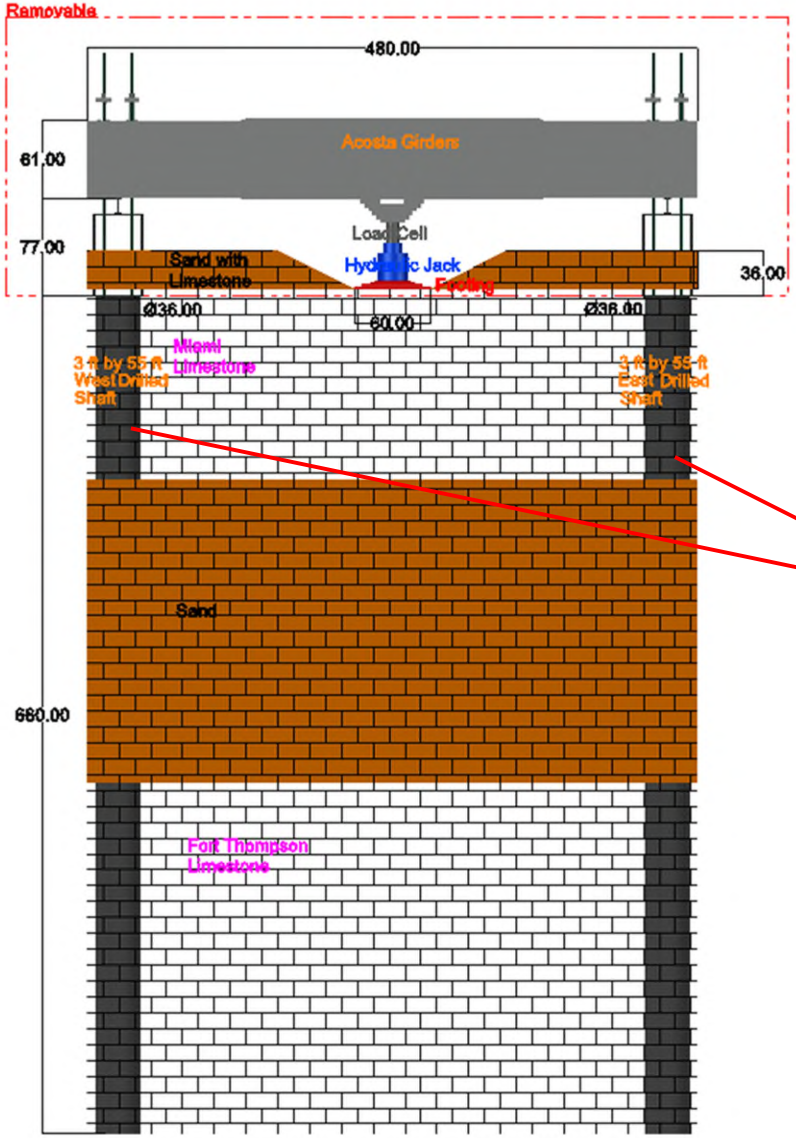


Distribution of Density, Shear Modulus, and the Young's Modulus for the SR-84 Site (Deliverable 5)

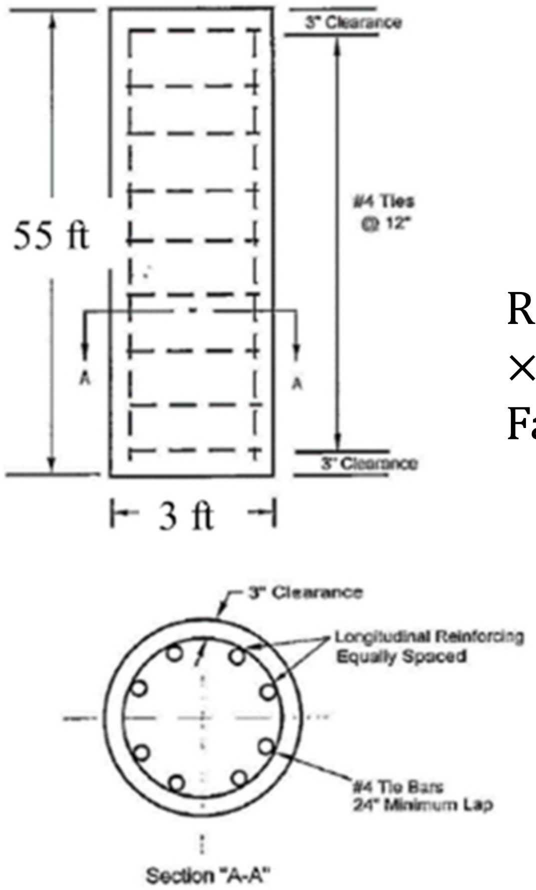
Using the Bearing Capacity equations with strength from B-3 (unit weight 110 pcf) Q_u = 335 psi , a 5' x 6' rectangular footing was selected

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

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

$\text{Reaction} = 98 \times 25 \times 12 \times 36 \times 3.14 \times 2\ /\ 2000 = 3323\ \text{tons}$

$\text{Factor of safety} = 3323/900 = 3.7$

Load Test 2: SR 84 Site, West Shaft (R.W. Harris)

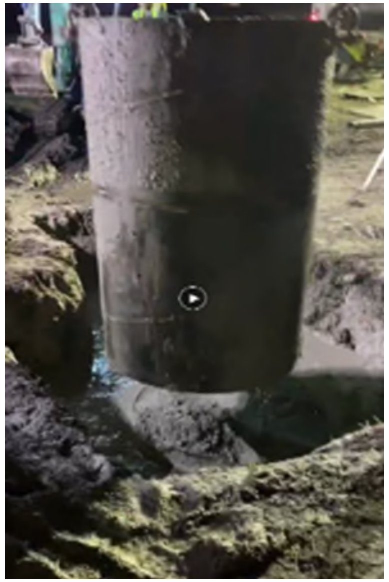
Steel Rebar Cage

Drilling Setup


Drilling

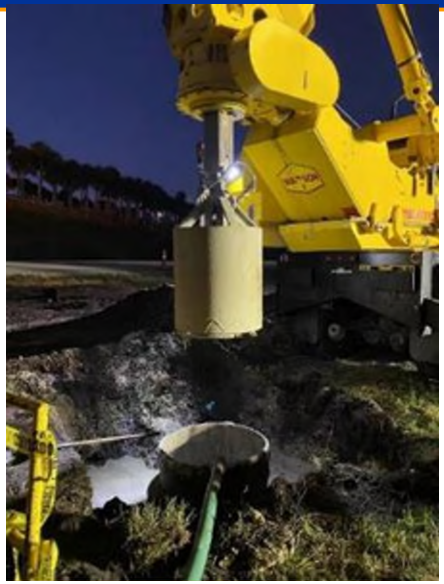

Placement for Rebar Cage


Concrete Pumping


Overflow

Load Test 2: SR 84 Site, East Shaft (R.W. Harris)

Drilling



Bailing Bucket to stable the hole

Alignment with West Shaft

Picture of East Shaft after two days

Measured Properties of the Bentonite Slurry for the East and West Shaft

Properties	East Shaft	West Shaft	Range Specified in FDOT Specification: 455-15.8.1 (65°F)
Density, pcf	66 ~ 67	66 ~ 67	64 ~ 73
Viscosity, Seconds	34 ~ 36	32 ~ 40	30 ~ 40
pH	9	9	8 ~ 11
Sand Content	2%	1%	≤4%

Measured Compressive Strength of Concrete Specimens for East and West Shaft

Days	Compressive Strength (East), psi	Compressive Strength (West), psi
14	9107.85	7821.9
21	8973.31	9061.28
28	9926.22	9625.19

Measured Slump of Concrete for East and West Shaft (same)

Properties	Measured	Range specified in 346
Slump, in	9 ~ 10	7 ~ 10

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

Fasting Setting Concrete for footing placement

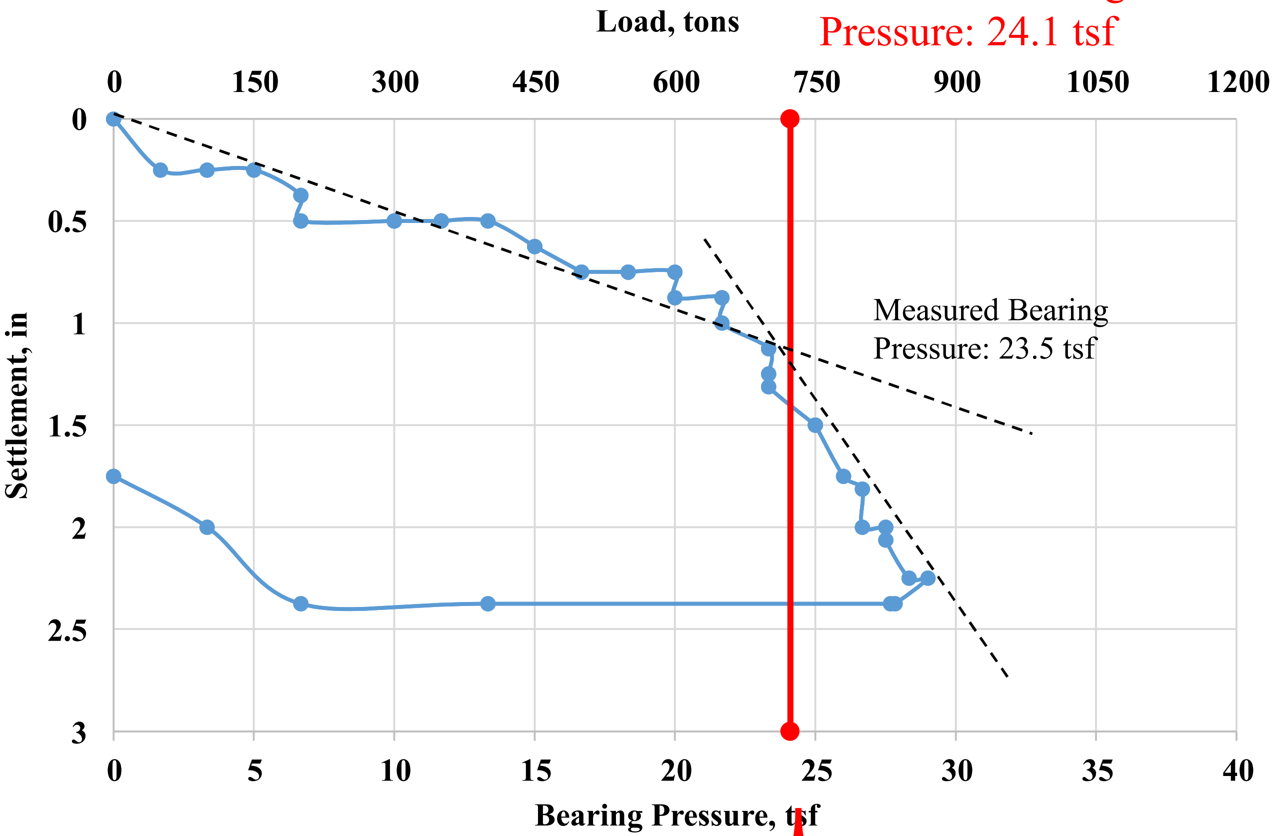
Installation of Girders

Measuring System

Hydraulic Jack and Load Cell Setup

Load Test 2: SR 84 Site, Load Test Results 5’ x 6’ 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, $\mathbf{N_R}$ = **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 $R \geq 0.3$

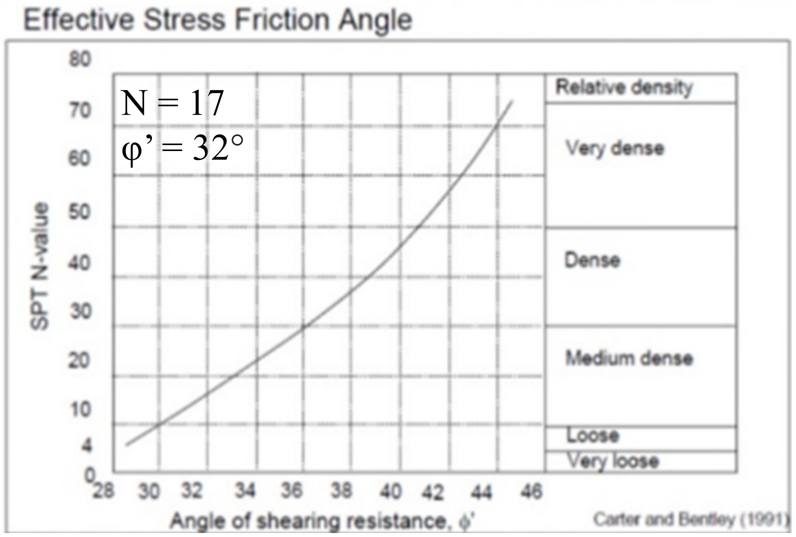
$R = 0.093T^2 (E_{soil} / E_{rock})$, limit R to 2.0

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

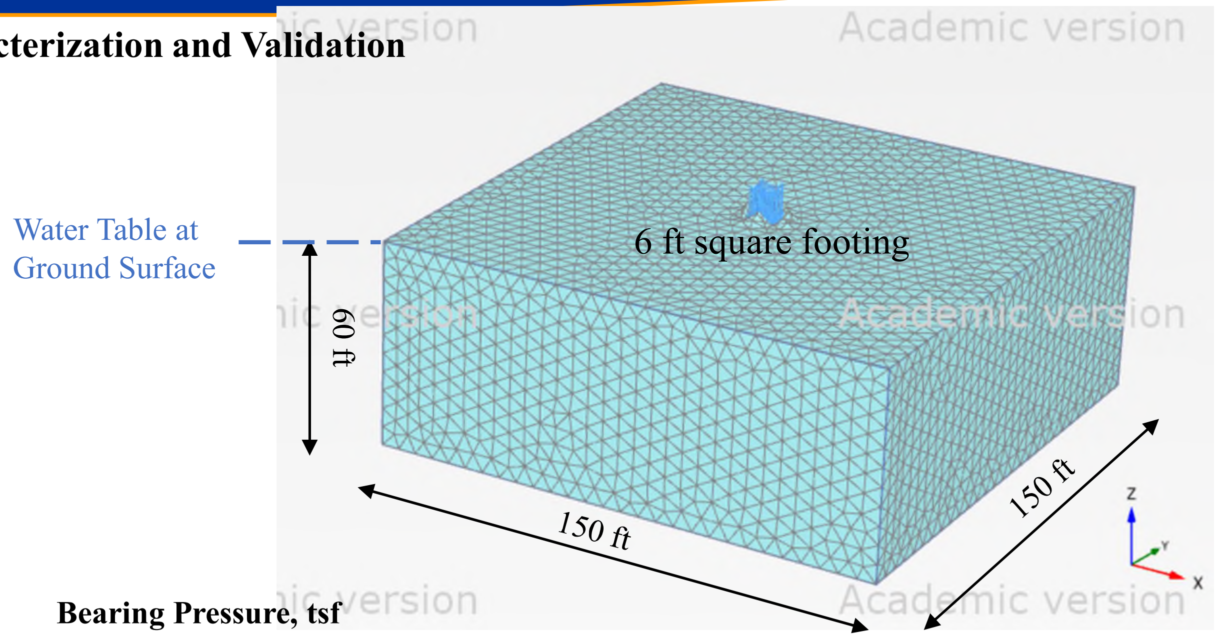
E_{soil} / E_{rock} (1,087/38,000) = Modulus ratio of soil and rock layers

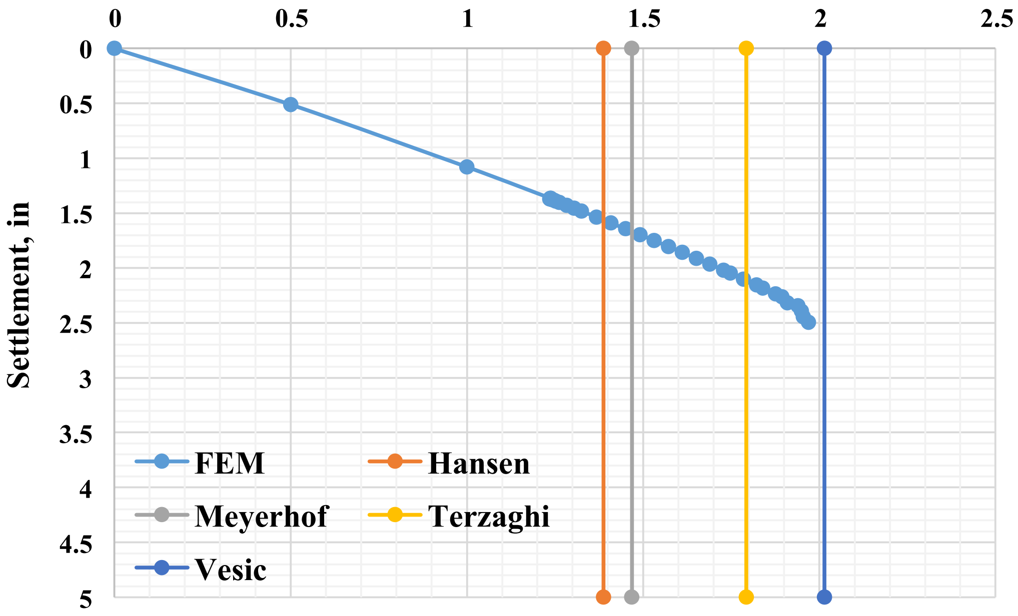
Load Test 2: SR84 – Rock over Sand - Estimating Sand Secant Modulus

FEM-Sand Layer Characterization and Validation



Effective Friction Angle and SPT N-Value (Peck, et al., 1974)





For submerged sand:

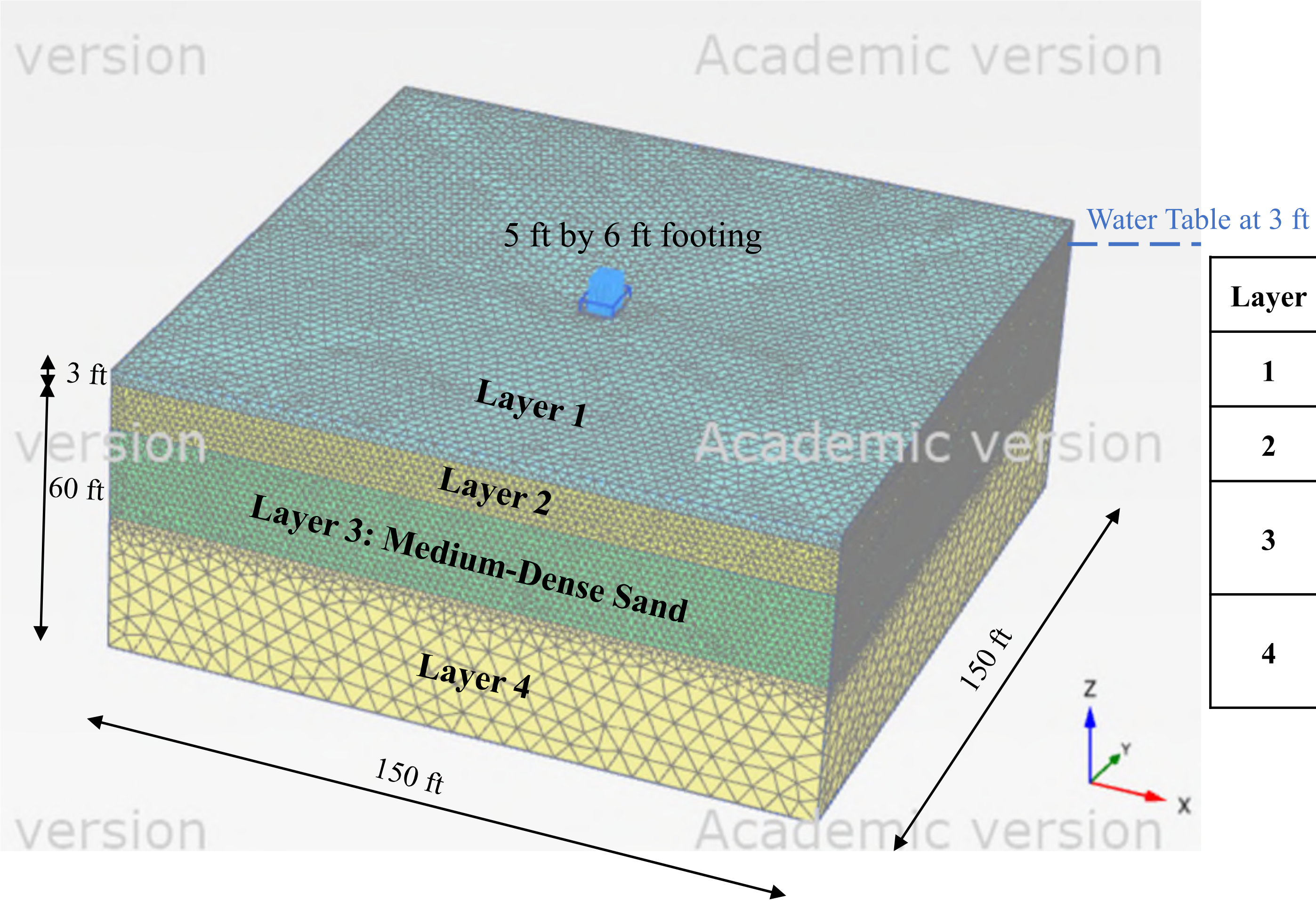
$E_s = 250(N_{55}+15)$, kPa (Bowles, 1996)

$N_{55} = 15$

$E_s = 1087$ psi

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

FEM – Plaxis 3D - Nonlinear Solution



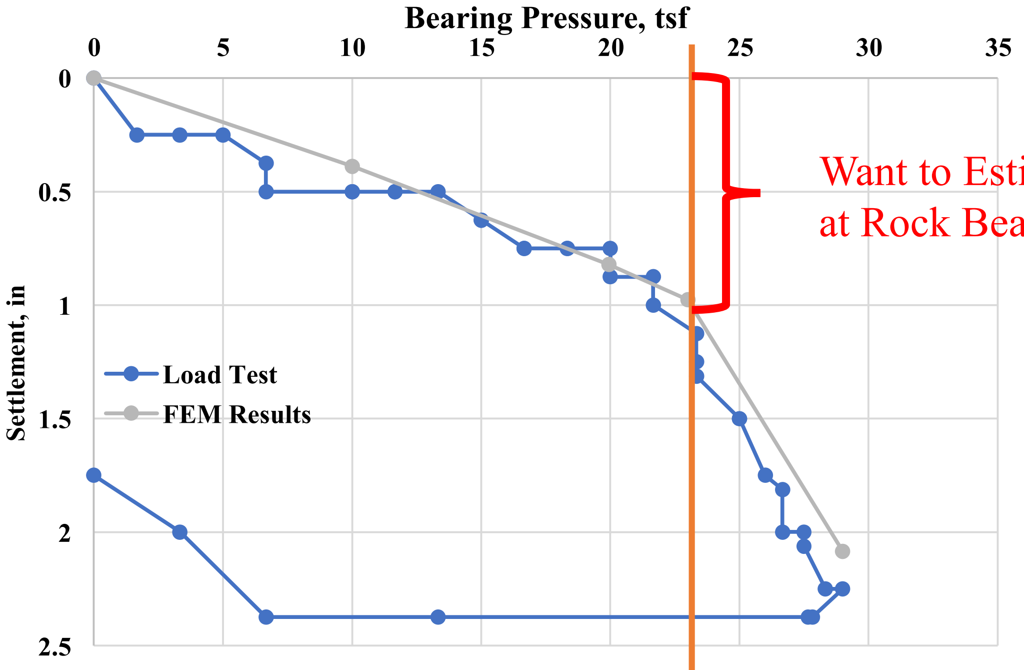
Material Model, Strength and Stiffness Parameters of Different Layer

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	0.10	50,000
2	110	Miami Limestone	Mohr-Coulomb	44	35.4	0	0.10	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	0.3	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

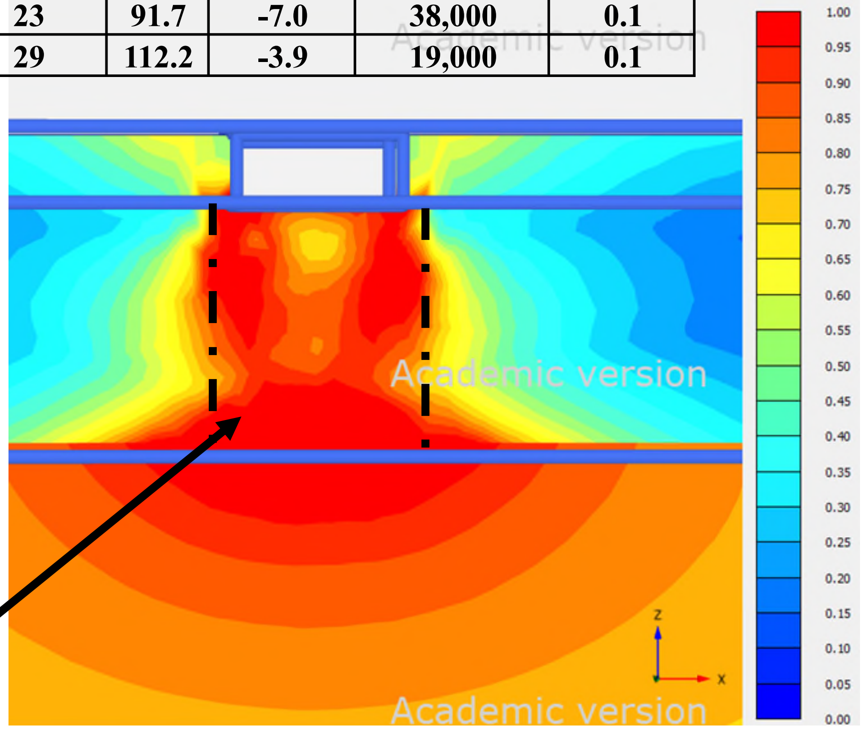
Loading Step	Bearing Pressure, tsf	Layer 2 110 pcf			
		σ1', psi	σ3', psi	Elastic Modulus, psi	Poisson's Ratio
1	0	3.7	1.5	38,000	0.1
2	10	2.7	1.6	38,000	0.1
3	19.95	40.5	-3.5	38,000	0.1
4	23	91.7	-7.0	38,000	0.1
5	29	112.2	-3.9	19,000	0.1



Measured versus FEM: load-settlement response

Want to Estimate Settlement at Rock Bearing failure

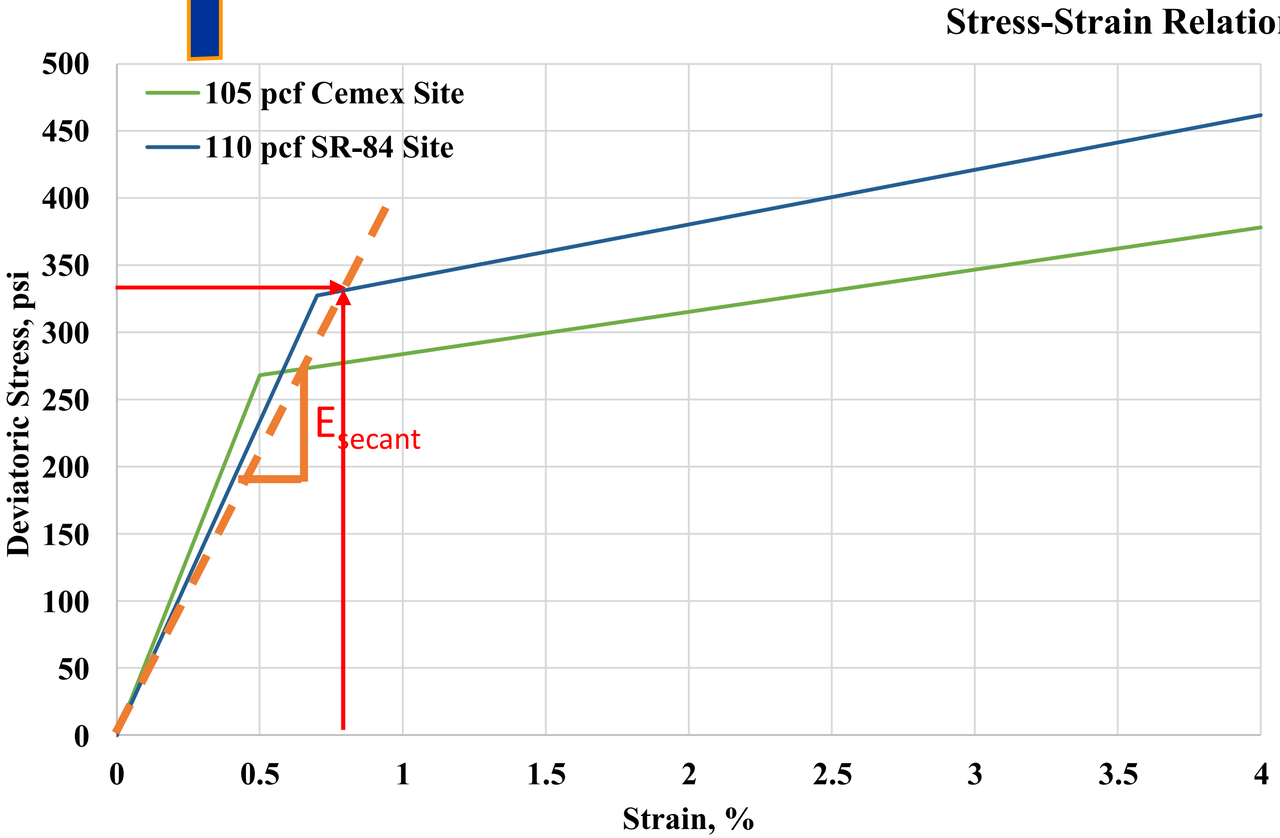
Bearing Failure of Rock – Punching Shear



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

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

Estimated Settlement -Hand Solution at Bearing Capacity



Modified (vertical failure) - Meyerhof Method (1968) estimate of bearing capacity (independent of sand modulus) of rock layer for layered system

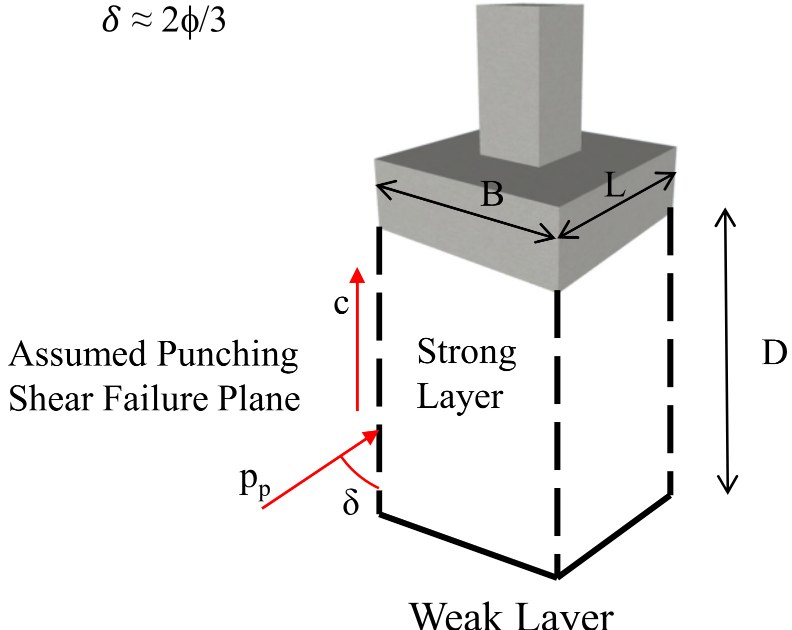
Note, there is **no N_R** - considers the ratio of secant modulus of rock to soil
Error may exceed 40% versus Florida Bearing Capacity equations

$$Qu = 2c(B+L)D + (2sB + L - B)p_p \sin\delta = 335 \text{ psi}$$

Where,

s = 1.3, shape factor governing the passive earth pressure

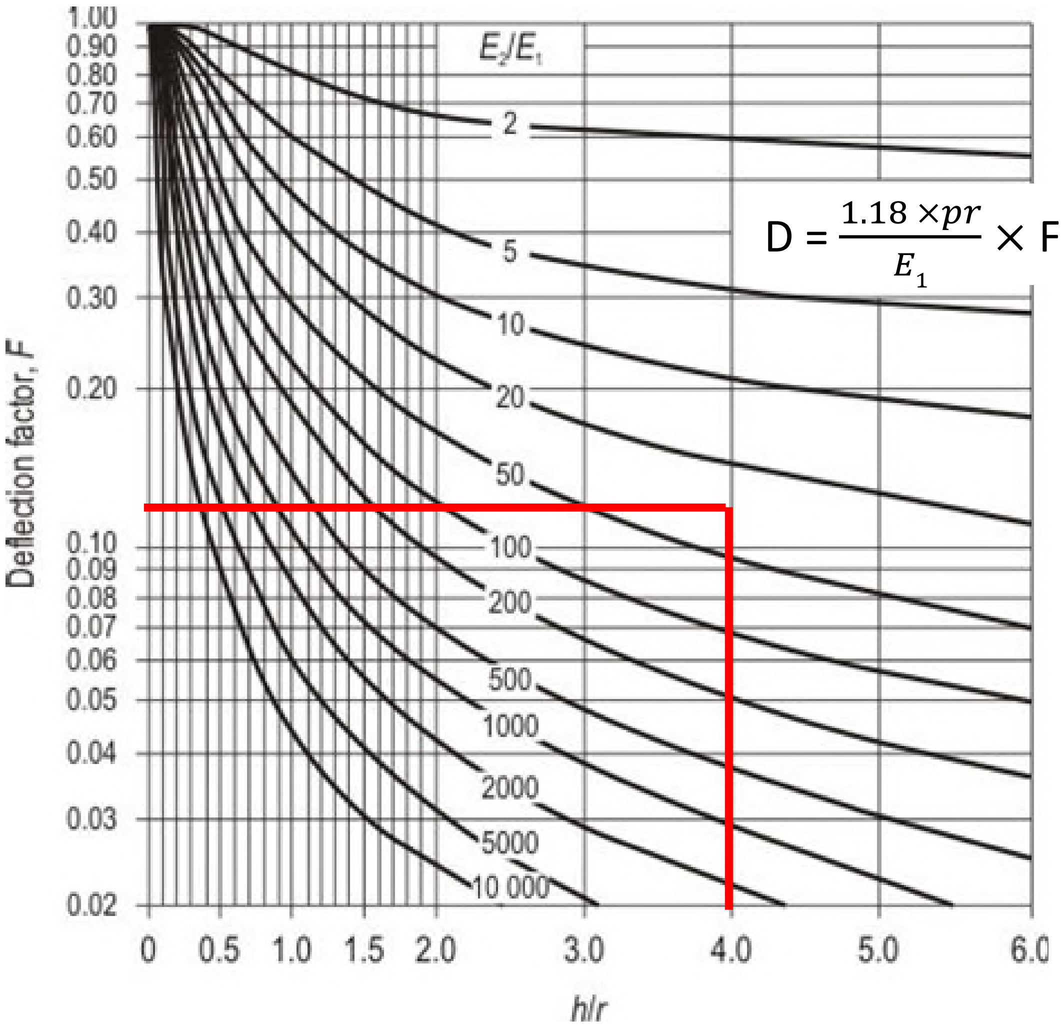
$\delta \approx 2\phi/3$



For SR 84 load test: Florida Bearing Capacity equations - Q_u = 335 psi,
From Stress vs. Strain curve, Strain = 0.89%, Secant Modulus, E_{secant} = 37,807 psi

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

Burmister Solution (1958) – settlement for layered solution – Hand & Linear Solution



E_2 = Upper layer elastic modulus = 38,000 psi
E_1 = Lower layer elastic modulus = 1,087 psi
r = radius 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

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

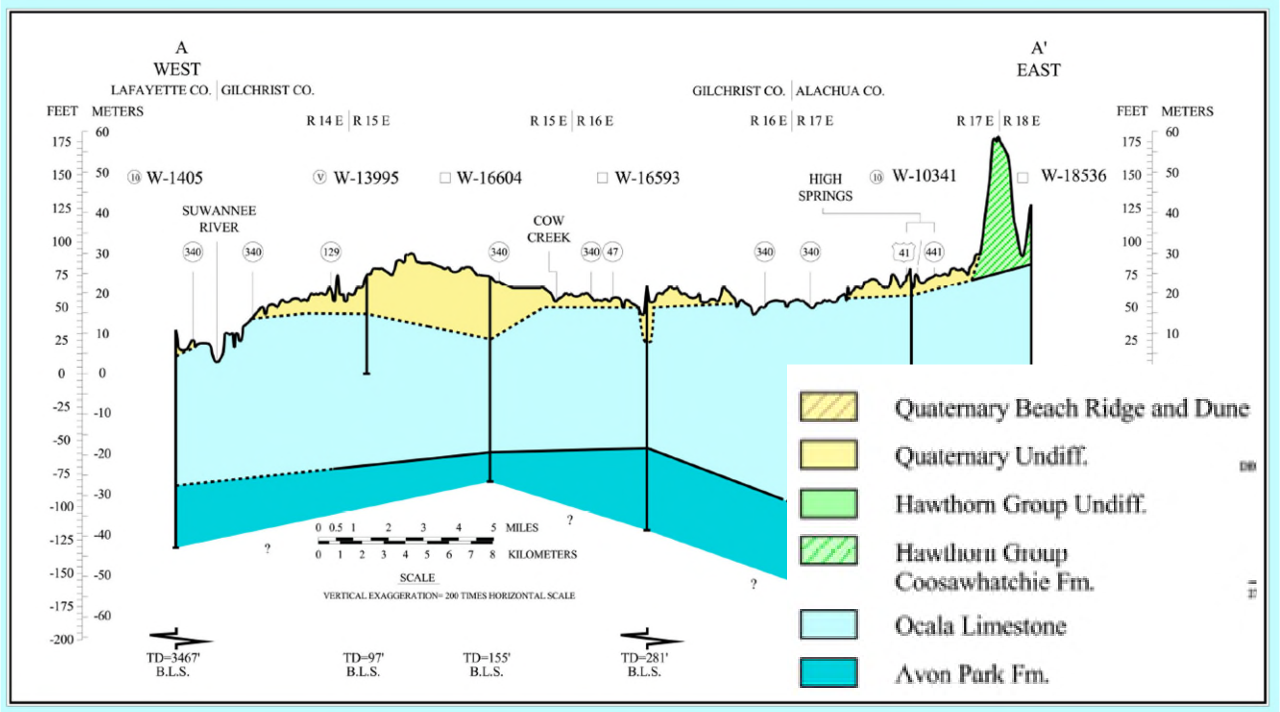
Using Secant Modulus – At Punching Failure
FEM: 0.98 in vs. Burmister's Method: 1.14 in

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**

Planned Load Test 3: Bell, Gilchrist County, Deliverable 4



Geologic Map of Bell, USGS

Load Testing of Shallow Foundation on Ocala Formation (limited prior triaxial strength data):

- Rock coring and strength test will be performed to size the footing and drilled shaft
- MWD and Seismic Shear Test will be conducted to aid the site characterization (function of dry unit weight)
- Measure and predict the bearing capacity and load-settlement response

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, the **geomean or median** are recommended to characterize the rock dry unit weight. Only use the modified **Meyerhof Solution (1968)** punching shear capacity of rock for two-layered as a rough estimation when no modulii of rock and sand.

- For footing settlement, the **bi-linear 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)** with **Bowles (1996)** estimate of sand modulus provides a good estimate of two-layer settlement.

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

- Further 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).

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	10/2021
5.) Seismic Field Testing to develop Mass Properties of rock	10/2021
6a.) Draft final (Task 6)	11/2021
6b.) Closeout teleconference (Task 6)	12/2021
7.) Final report (Tasks 7)	12/2021

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- H2R Drilling Crew for micro-piles installation
- R.W. Harris Drilling Crew for drilled shaft installation
- Terracon and PSI Rock Coring Crew
- Powell Family Structures & Materials Laboratory: Scott Powell, Caitanya Jivan (CJ) Bhakti
- Weil Hall Structure Lab: Dr. Taylor Rawlinson

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

Thank You!

Q & A