Strength Envelopes for Florida Rock and Intermediate Geomaterials

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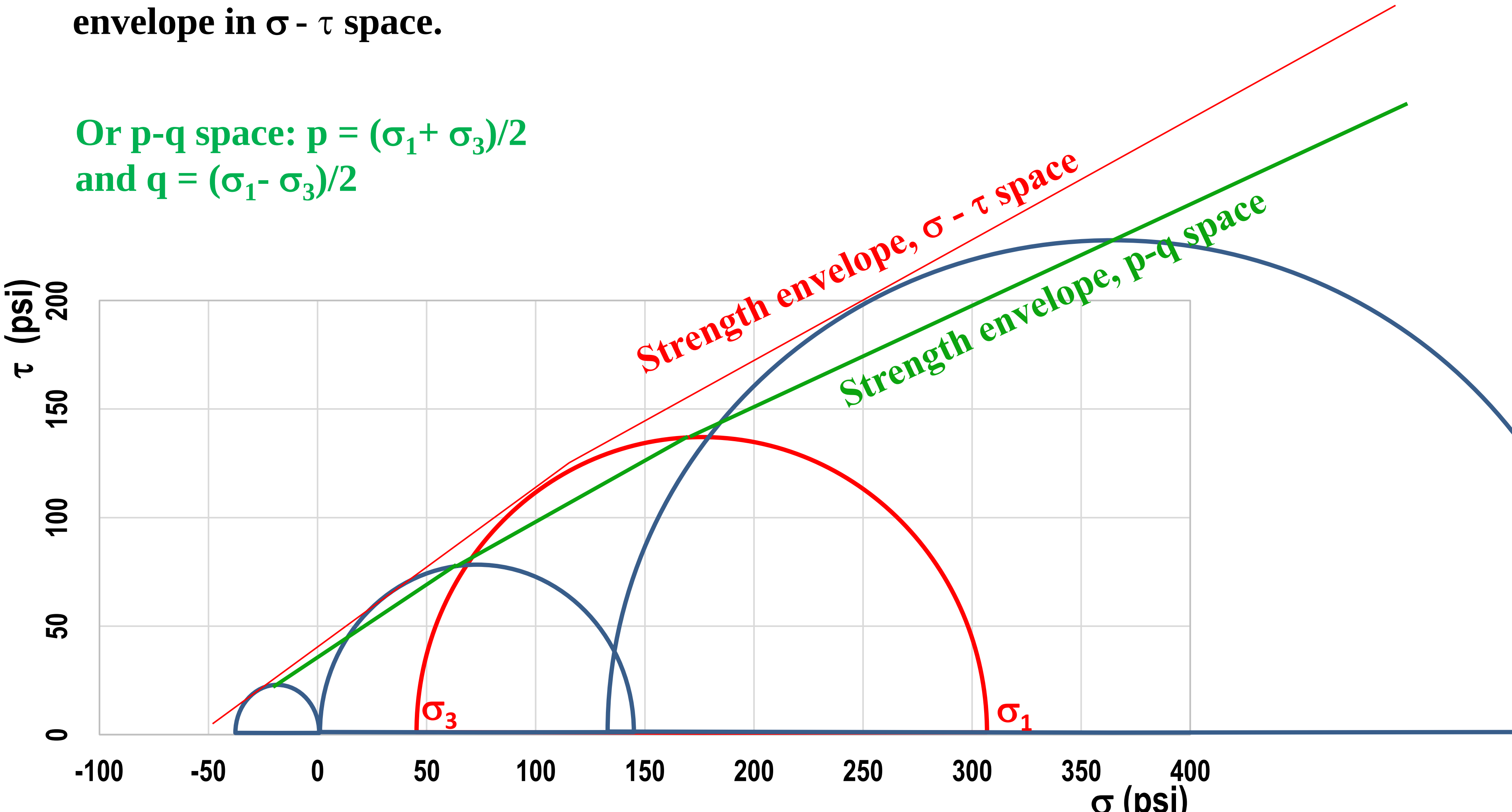
Background

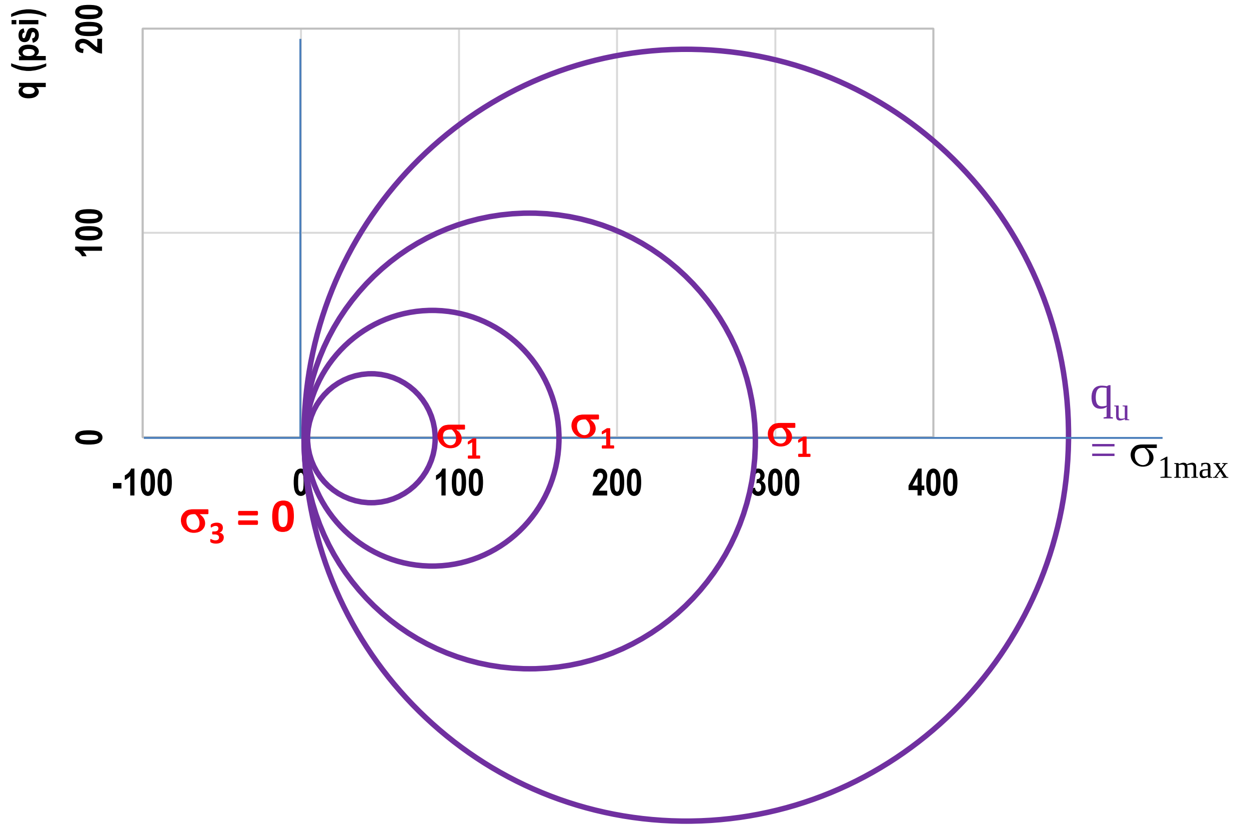
Tangent line with all Mohr circles (σ_{1i} and σ_{3i}) at Failure Stress State is strength envelope in σ - τ space.

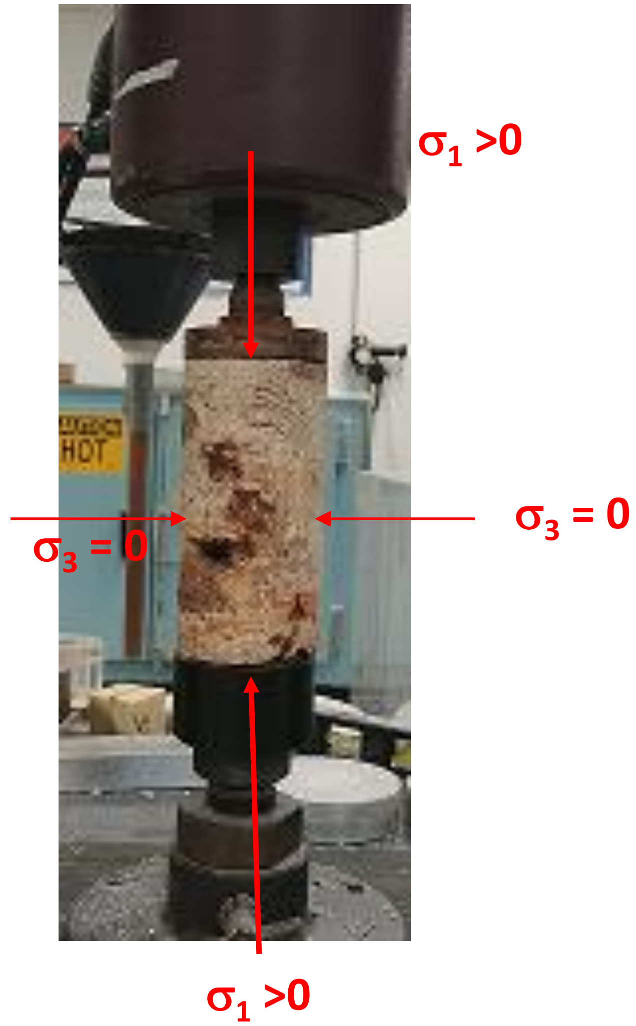
Or p-q space: $p = (\sigma_1 + \sigma_3)/2$ and $q = (\sigma_1 - \sigma_3)/2$



Background – Rock Strengths

Unconfined compression q_u





Unconfined compression Mohr circle at failure

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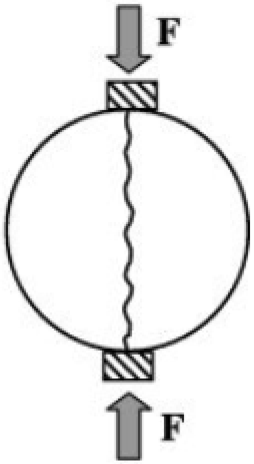
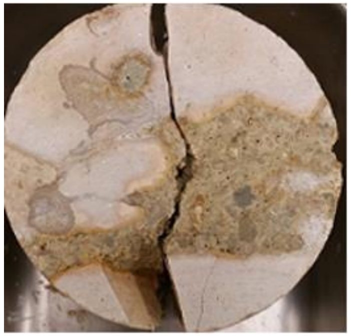
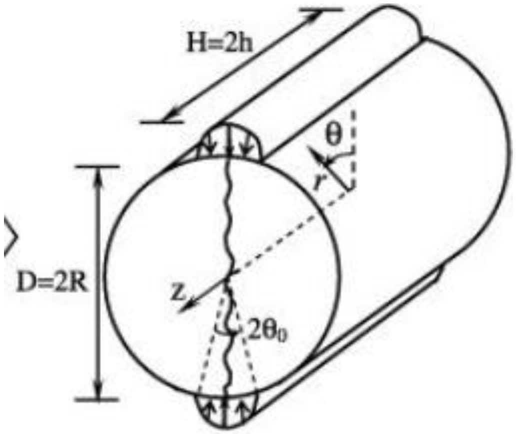
Background – Rock Strengths

Brazilian Splitting Tension Strength BST

For rocks, it is impractical to fabricate a bone-shape specimen for Direct Tension.
⇒ Tension strength is indirectly evaluated by BST method:





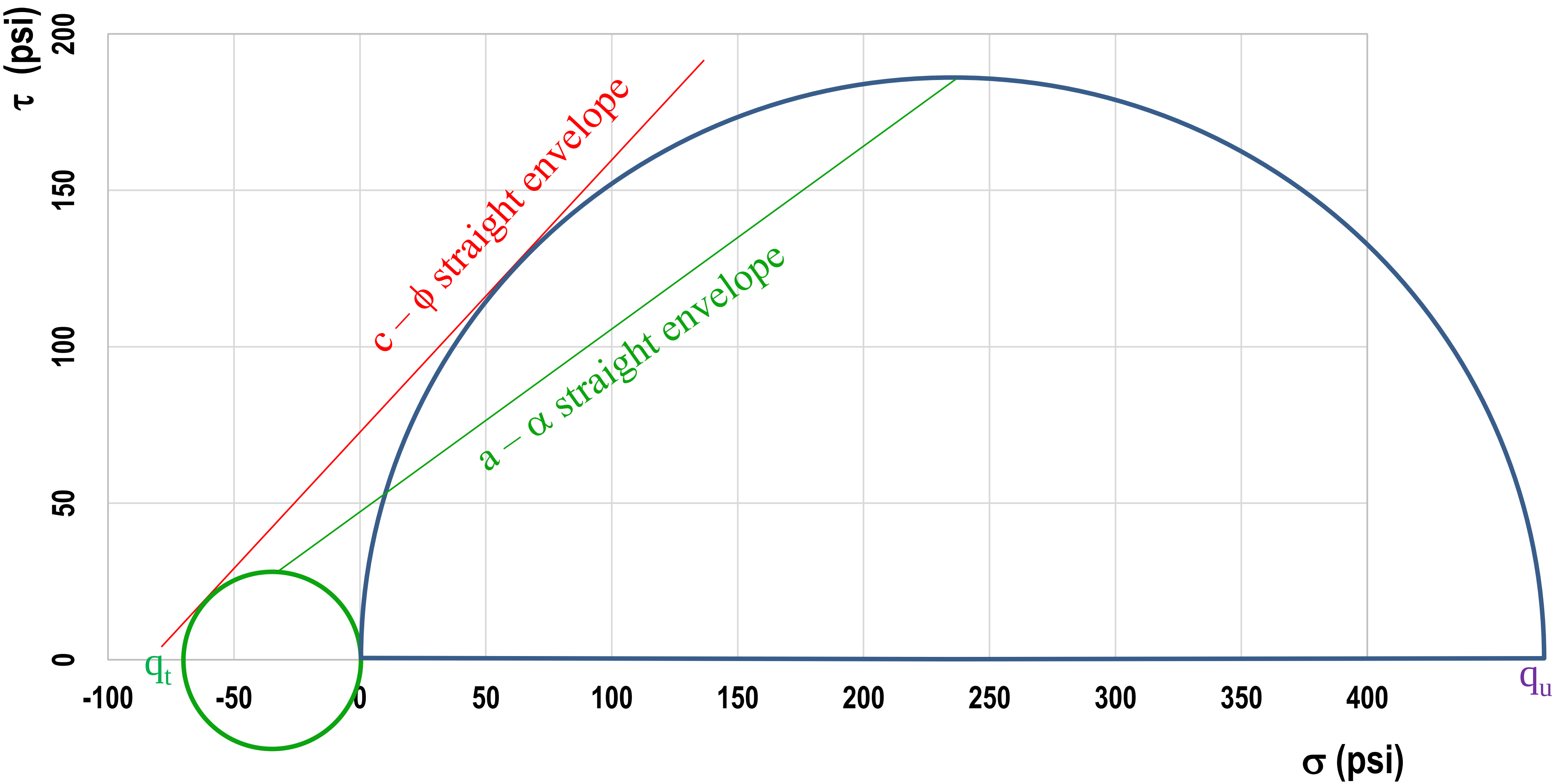


Perras and Diederichs (2014): Statistically, q_t = 0.7 * BST

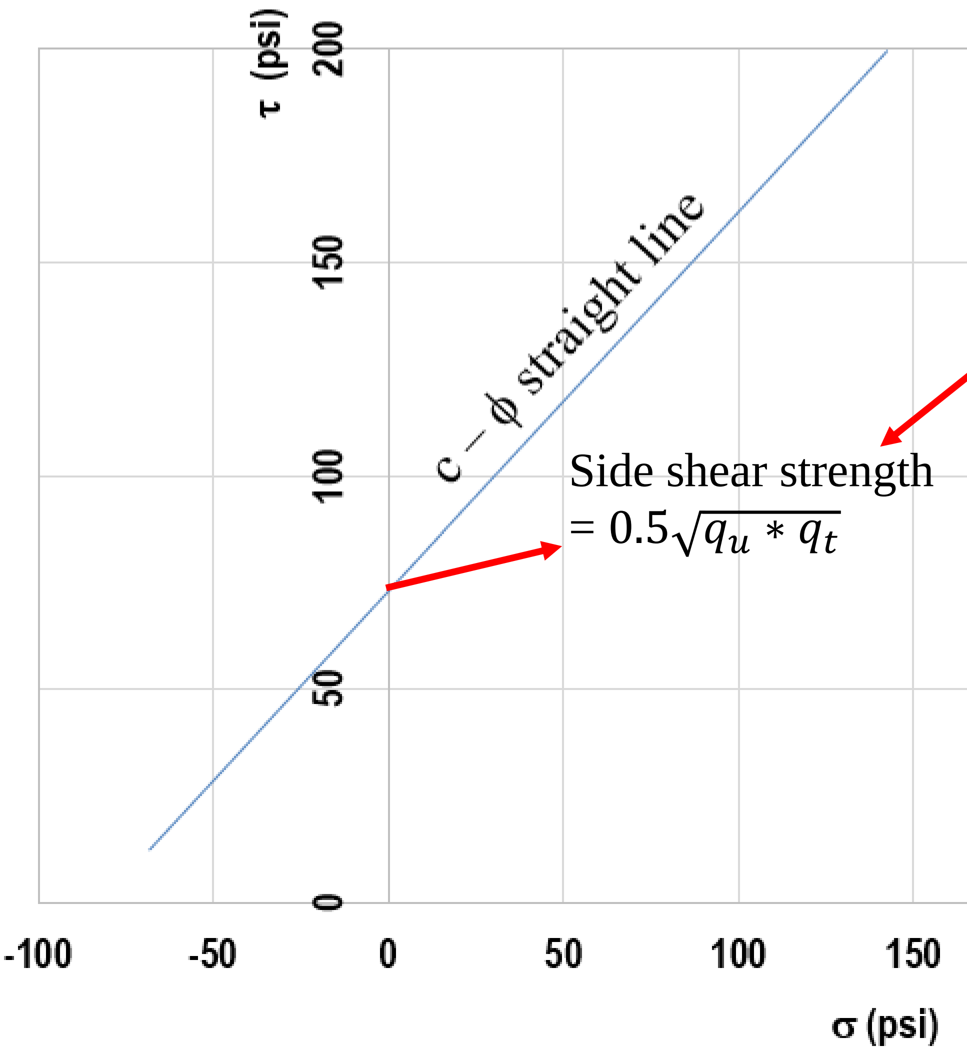
Background – from Rock Strength to Strength Envelope

Mohr-Coulomb Strength Envelope

q_t (=70% BST) and q_u are the 2 points on the strength envelope



Background – from Rock Strength to Strength Envelope

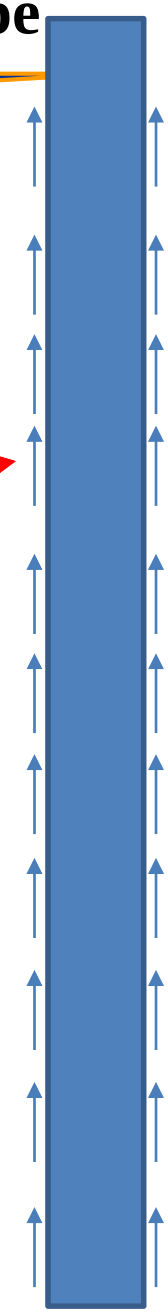


Mohr-Coulomb c - φ Strength Envelope

This straight c - φ line is sufficient enough for side shear strength (adhesion/friction between concrete surface and rock) calculations for deep foundation design

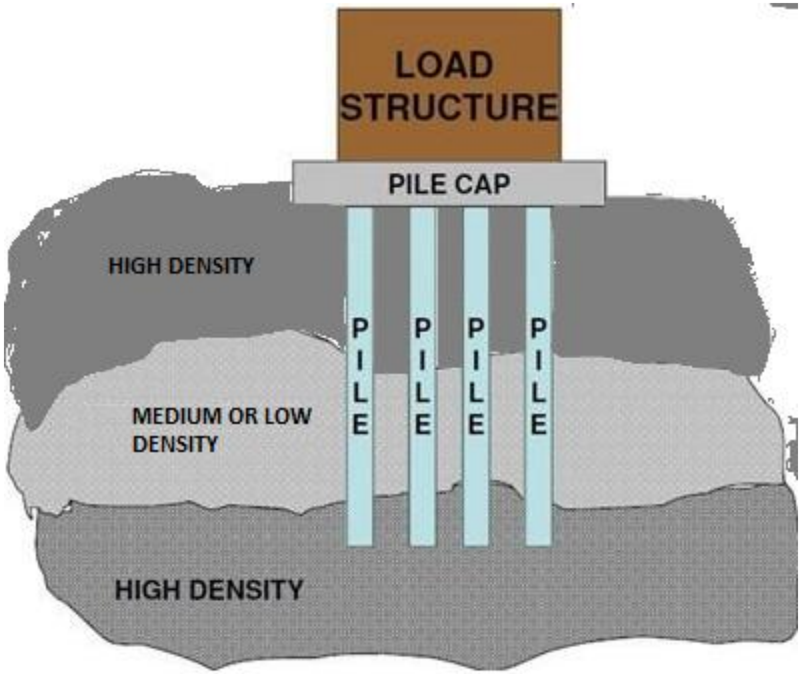
Thus historically, only BST and q_u strength tests have been conducted.

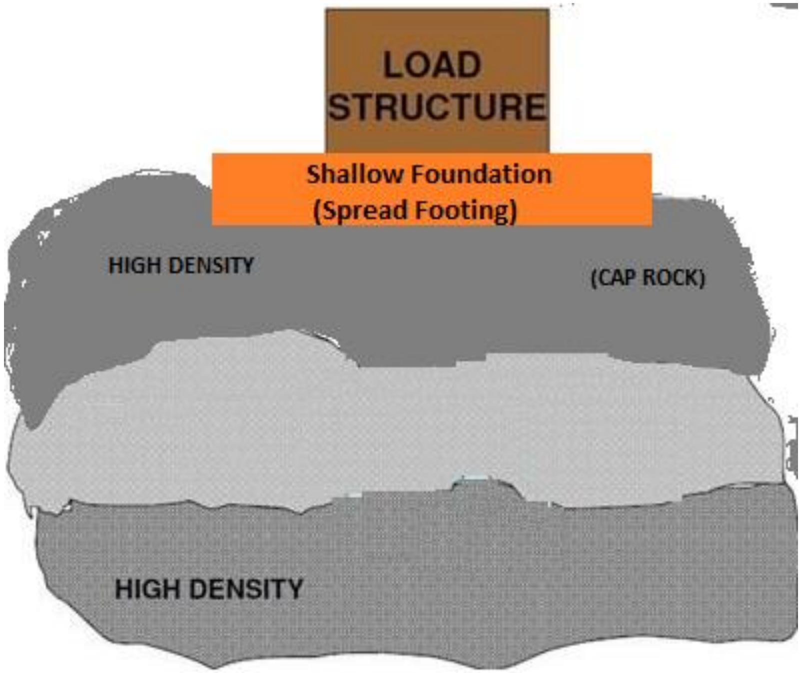
Rock triaxial strength tests have never been performed for low strength/ porous rocks, such as Florida rocks.



Background - Foundation

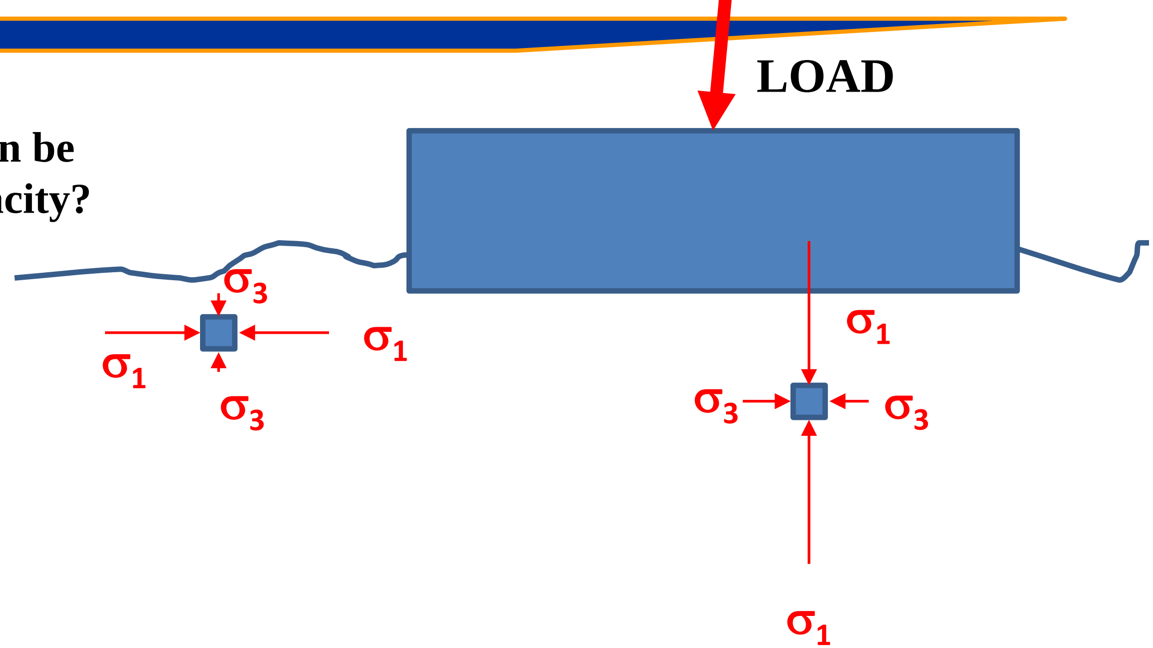
Footing size would be larger than a typical pile cap, but still adequately sized so that there is room to construct: e.g. 4x12 ft^2 to 15x15ft^2



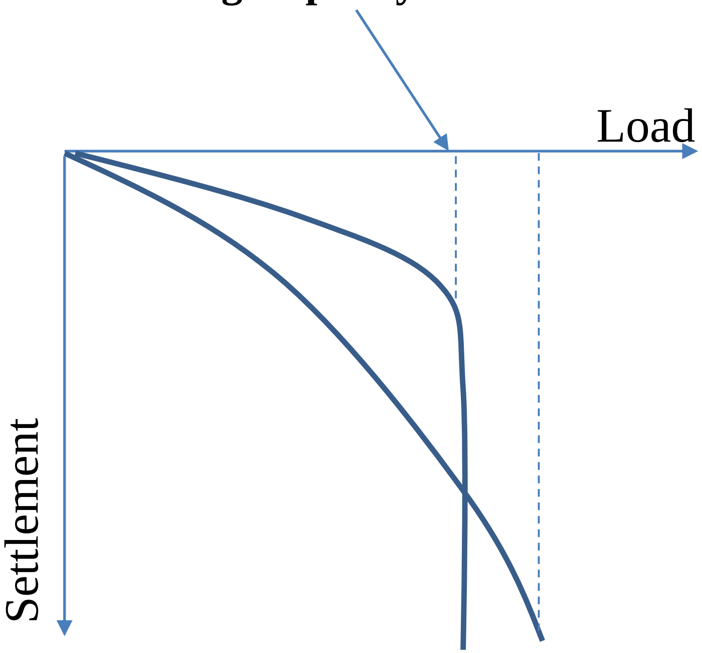


Problem Statement

Question: Can a shallow foundation be adequate in terms of Bearing Capacity?



When elements underneath footing reach **strength envelopes** (failure), the applied load reaches its **Ultimate Bearing Capacity**



<u>Bearing Capacity Evaluation Methods:</u>

1) Finite element method: Directly needs Strength Envelopes

2) Bearing equation: Indirectly needs Strength Envelopes
   e.g.:

- Terzaghi or Meyerhof (for Soils): use Mohr-Coulomb strength parameters (i.e., envelopes)
- Carter and Kulhawy (for Rocks), currently used by FHWA and TRB, equation derived from Hoek-Brown envelopes:

$$p_u = \left[\sqrt{s} + \sqrt{m\sqrt{s} + s}\right] q_u$$

s and m: Rock parameters using Hoek-Brown

Problem Statement

Can we use the straight line Mohr-Coulomb c-φ envelopes (using BST, q_u) ?

Or should we use Hoek-Brown strength envelope using our q_u ?:

$\sigma_1 = \sigma_3 + q_u\,(m\frac{\sigma_3}{q_u} + s)^a$

$s = e^{(GSI-100)/(9-3D)}$

$m = m_i\ e^{(GSI-100)/(28-14D)}$ $m_i \approx 10$ for carbonate rocks

$a = 0.5 + (e^{-GSI/15} - e^{-20/3})\ /6 \approx 0.5$

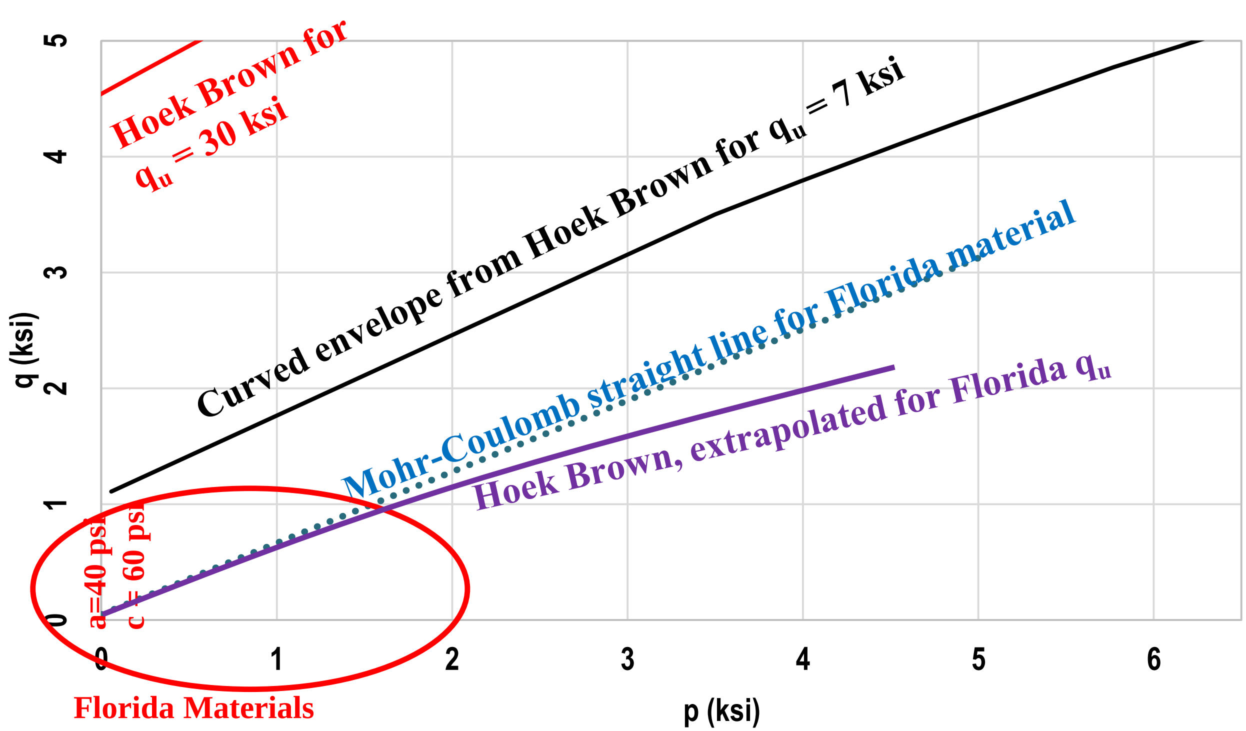
GSI: Geological Strength Index, evaluated based on discontinuities of jointed rock

Problem Statement

Hoek-Brown and Mohr-Coulomb

TABLE 1.—Values of Strength Parameter *m* for Intact Rock Materials (s = 1.0)

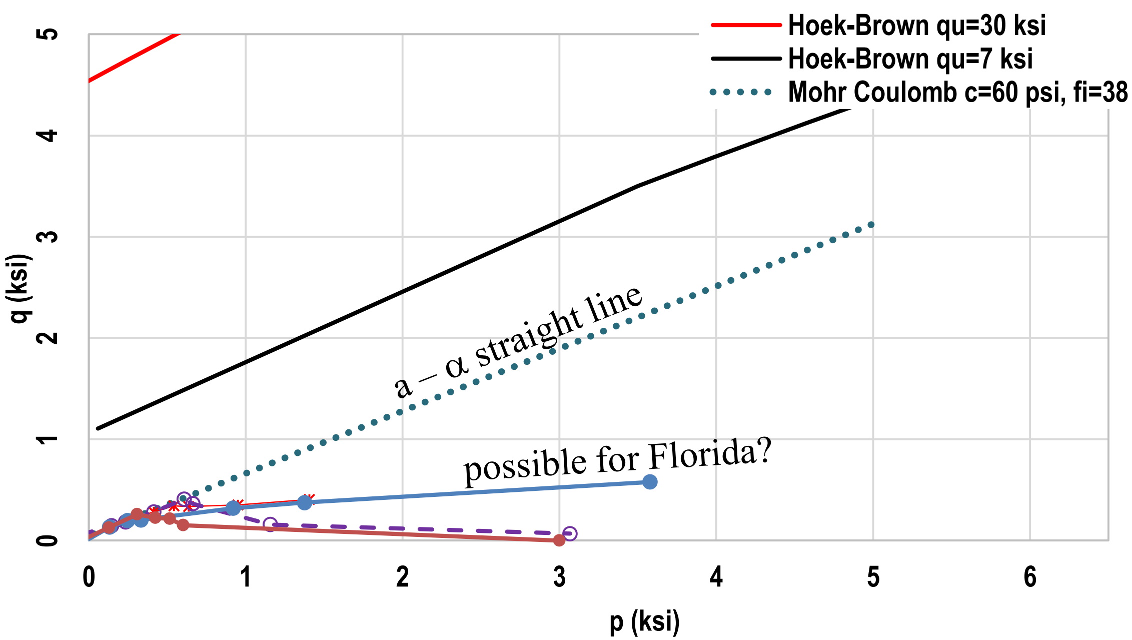
Rock type (1)	Data from reference numbers (2)	Number of data points (3)	Range of q_u in pounds per square inch (4)	m (5)	Coefficient of determination, r^2 (6)
Limestone	21, 41, 51	84	6,380–29,200	5.4	0.68
Dolomite	9, 43	25	21,500–73,400	6.8	0.90



Problem Statement

Hoek-Brown Limitations:

- Not a direct function of porosity (or bulk dry unit weight)
- Developed for underground excavation (tunnel) in hard rock with **brittle fracture**
- Rock strength (q_u) of Florida carbonate rocks are typically much lower than other rock
- They **do not** always behave as **brittle**
- Florida carbonate rocks are soft in general, but appear as jointless ⇒ Not suitable to evaluate Florida materials using GSI



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Objectives

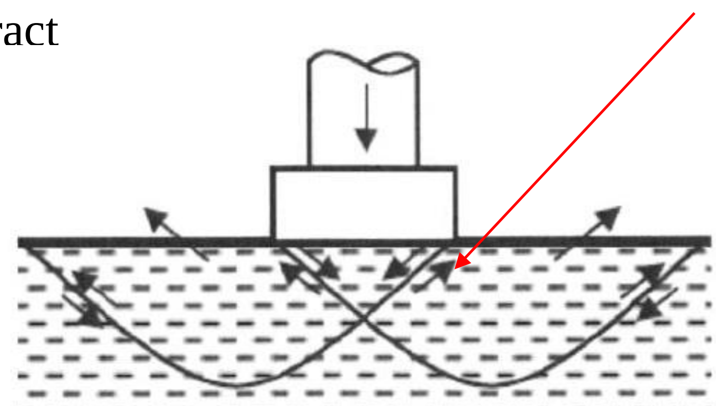
MAIN OBJECTIVE:

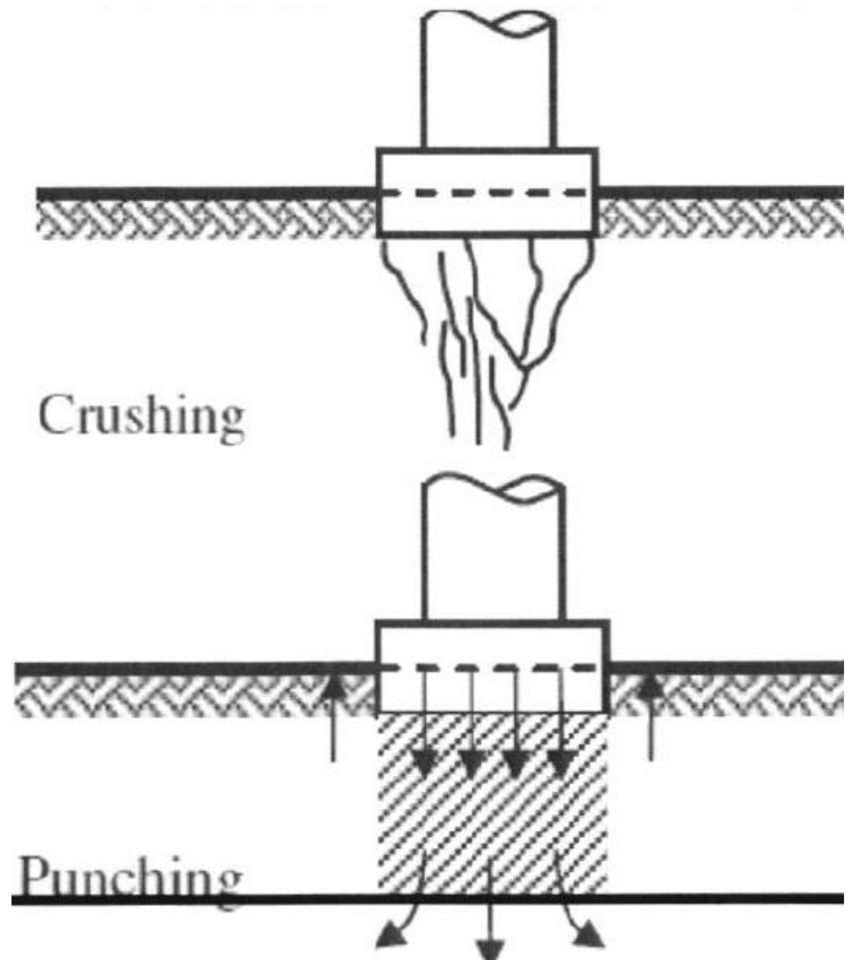
Develop strength envelopes for both compression (underneath footing) and extension (outside footing) loading for Florida carbonate-rocks.

This is unique, as none is available. Using conventional envelope can lead to unsafe foundation design.

SIDE OBJECTIVES:

1) Develop volumetric strain models for Florida carbonate-rocks: (i) when the material tends to dilate; or (ii) contract





Investigation topics

- **Identify key index parameters:**
  - **Formation/ Mineral type**
  - **Porosity (Dry unit weight),**
  - **Carbonate content,**
- **Identify key strength test and test ranges:**
  - **Traditional BST test**
  - **Traditional q_u test**
  - **Triaxial test**
- **Identify method to measure volumetric responses thru triaxial tests**

Florida Rock Index Parameters

<u>Mineral Components:</u>

- Calcium carbonate $CaCO_3$:
  - Calcite: depending on grain size:

Microcrystalline

Calcarenite

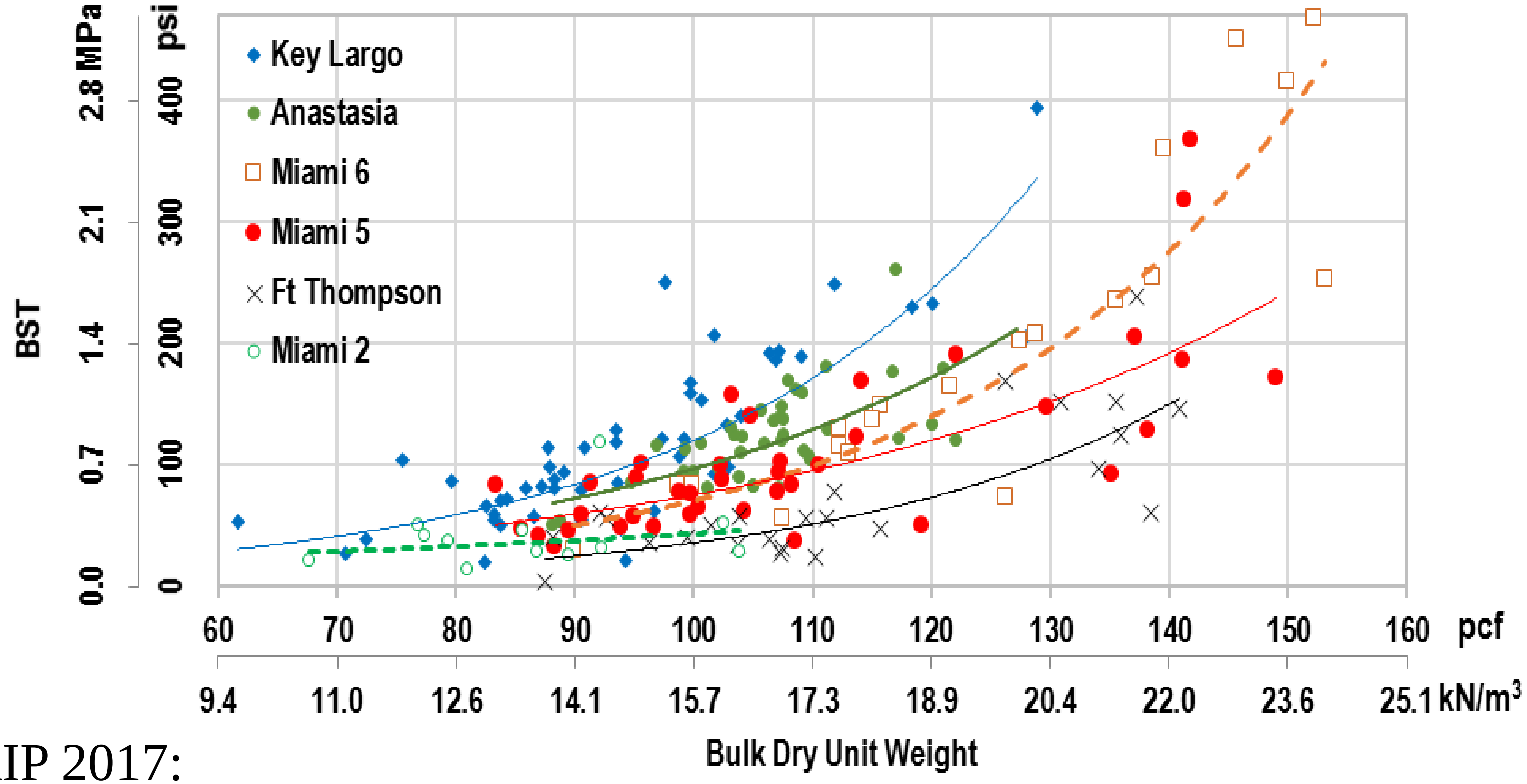
Coquina

  - Aragonite: Typically microcrystalline
- Dolomite $MgCa(CO_3)_2$: Typically microcrystalline
- Quartz SiO_2
- Other trace elements: Al, K, Mn, Na, Fe, and Zn

Florida Rocks are of very low strengths. Approximately 65% of 3,300 data tested resulted in q_u less than 700 psi (threshold that FHWA defines as Intermediate GeoMaterial – IGM)
So, Florida carbonate materials are rocks – geologically, but most of them are classified as IGM – geotechnically.

Correlations BST, q_u versus Index Parameters

Each formation has a different trendline, due to the properties of each formation, and most dominantly the carbonate content seen earlier



GRIP 2017:

$$BST\ (psi) = 2.468\ F_t\ e^{0.5C}\ e^{0.03\ \gamma_{dt}\ B}$$

$$q_u\ (psi) = 3.24\ F_u\ e^{2C/3}\ e^{0.04\ \gamma_{dt}\ B}$$

$$B = 1 \text{ if } \gamma_{dt} < \gamma_{dt0}$$

$$B = \sqrt{\gamma_{dt}/\gamma_{dt0}} \text{ if } \gamma_{dt} \geq \gamma_{dt0} \qquad \gamma_{dt0} = 140 \text{ pcf}$$

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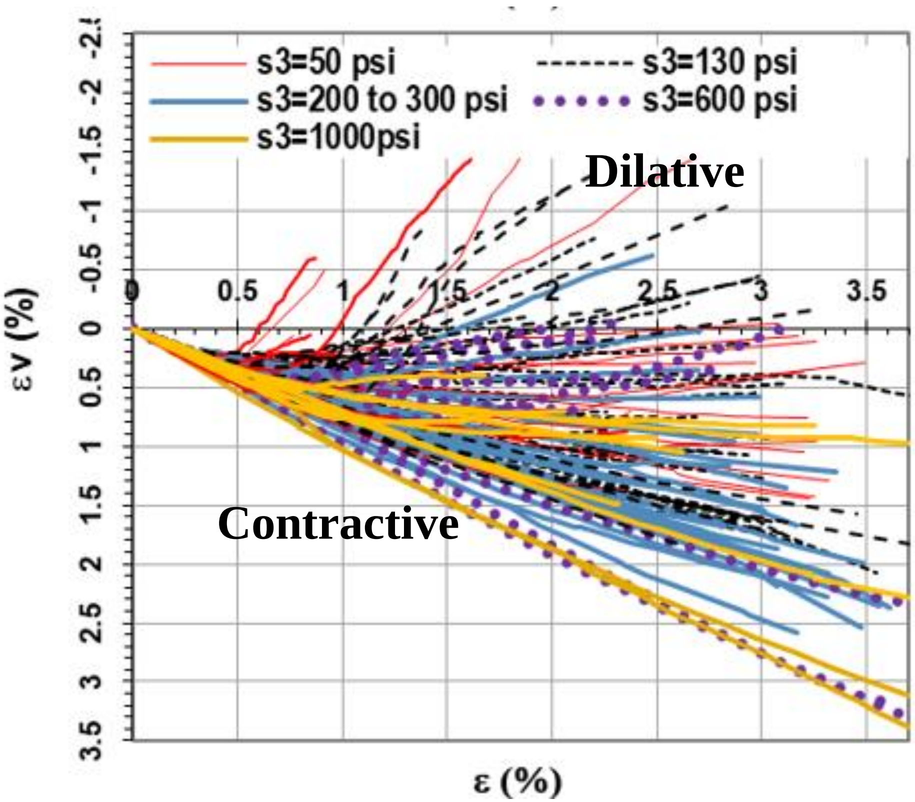
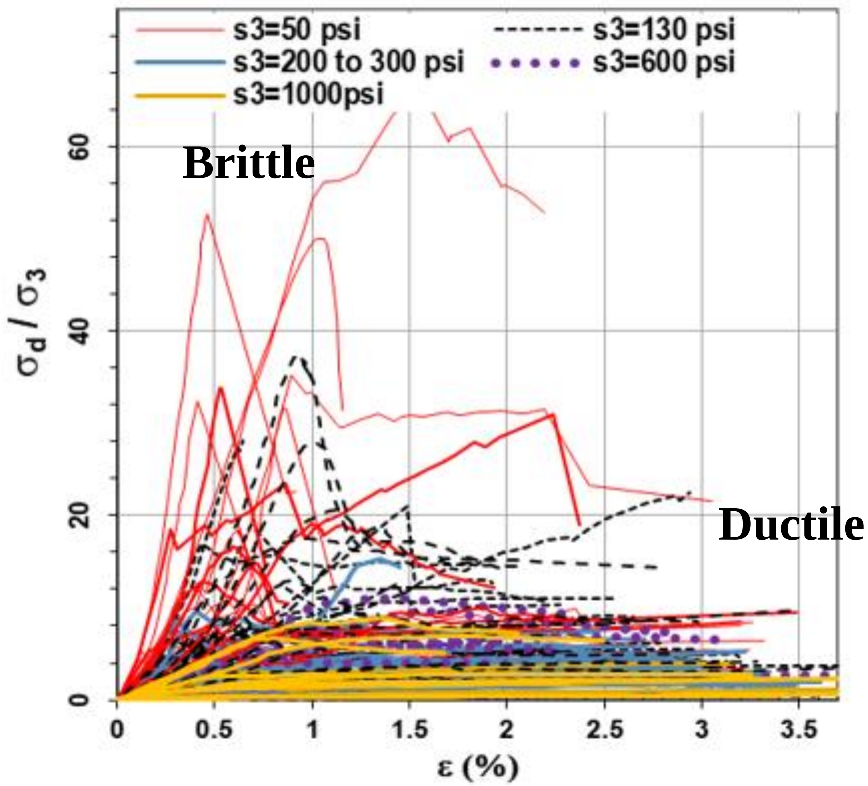
Triaxial tests

Hoek Cell

Oil volume change measurement is measured (GRIP 2017)
so that volumetric responses of triaxial tests can be measured

Approximate range of triaxial behavior





- Brittle stress-strain (rupture) behavior typically associates with dilative volumetric responses.
- Ductile ……associates with ……contractive.

Approximate range of triaxial behavior

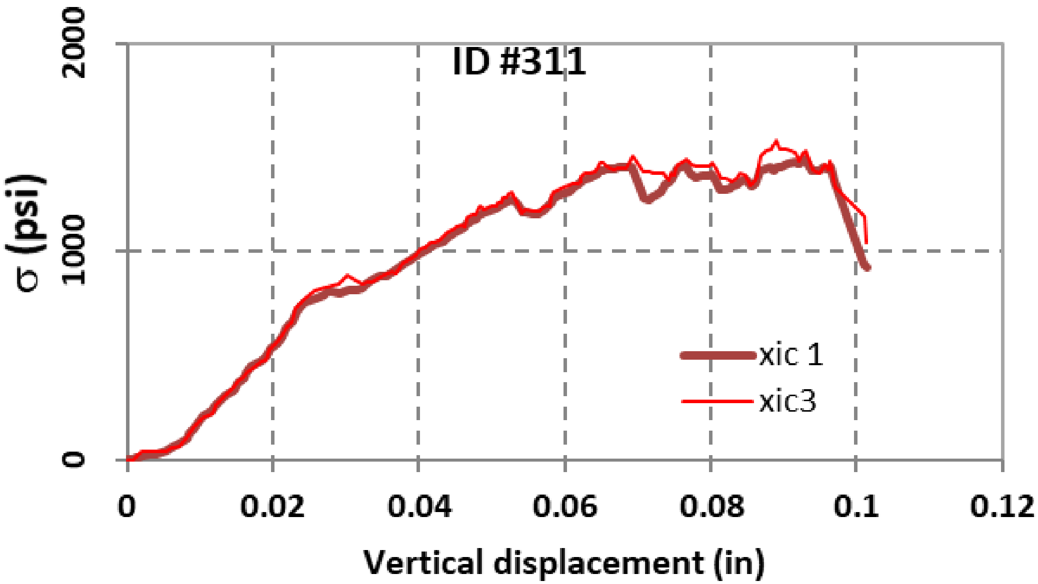
Not typically encountered for shallow formations

σ3		Bulk Dry Unit Weight Range						
		60 – 65	66-85	86-110	111-120	121-130	>130	pcf
(MPa)	(psi)	9.4-10.2	10.3-13.4	13.5-17.3	17.4-18.9	19.0-20.4	>20.4	kN/m^3
0 – 0.1	0 – 10	Transition	Transition	Brittle	Brittle	Brittle	Brittle	
0.2-0.3	25 – 50	Ductile	Transition	Transition	Brittle	Brittle	Brittle	
0.9-1	130-150	Ductile	Ductile	Transition	Transition	Brittle	Brittle	
1.4	200	Ductile	Ductile	Ductile	Transition	Transition	Brittle	
>2.1	>300	Ductile	Ductile	Ductile	Ductile	Transition	Transition	
>7	>1000	Ductile	Ductile	Ductile	Ductile	Ductile	Ductile	

Typically encountered for shallow formations

Some formations (such as Anastasia) would be more predominantly ductile, even in the 121-130 pcf zone.

Crushing behavior for Porous Rock under High Pressure

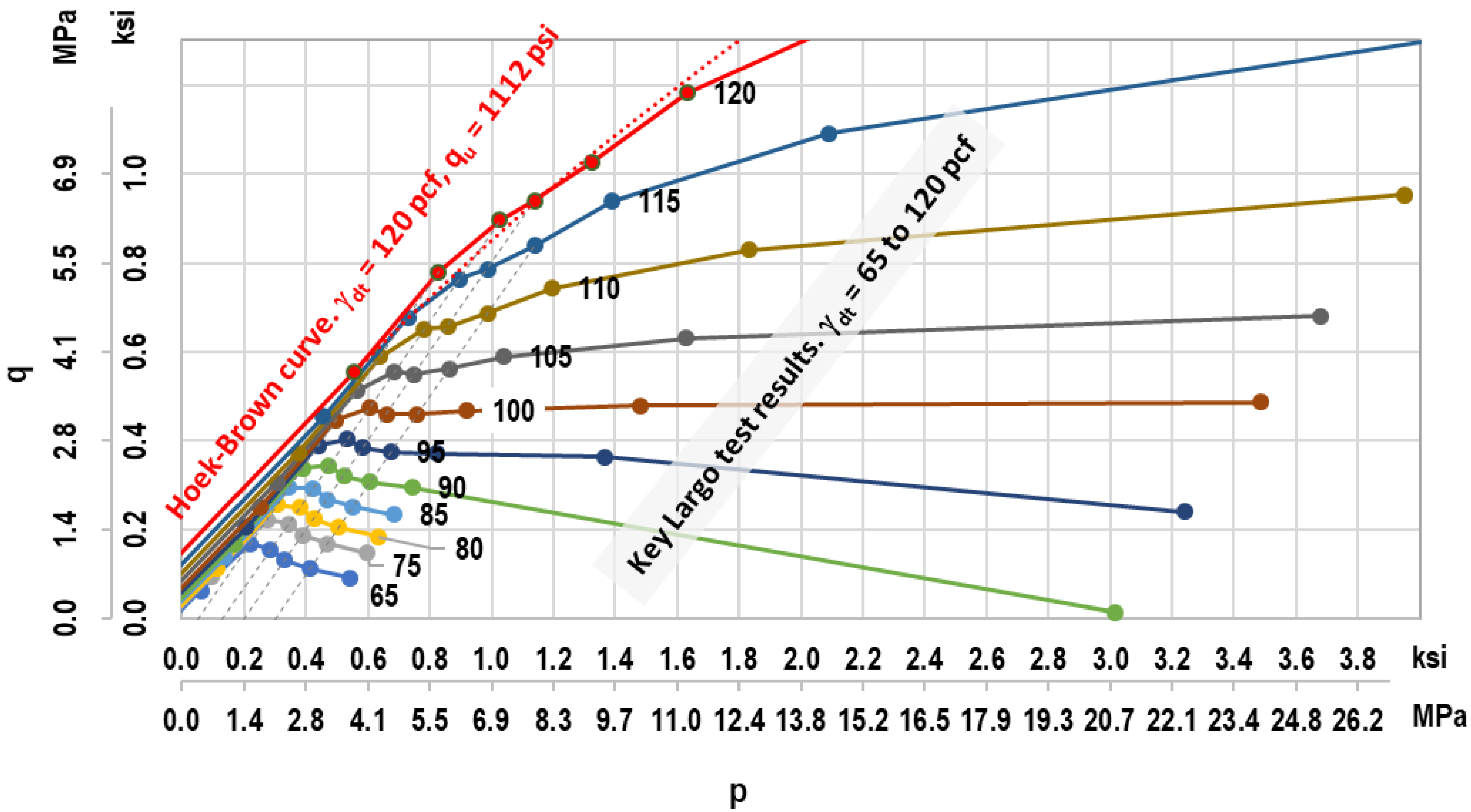


In this case, the deviatoric stress σ_d =0

It does not mean the rock has zero strength. The porous rock was simply "pulverized" or "crushed" under high isotropic pressure. The test stopped due to excessive displacement in combination with drop in pressure.

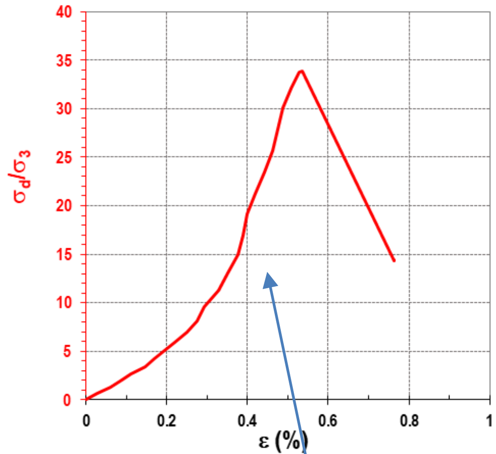
Mean Strength Envelopes – Key Largo

The strength envelopes have steeper downward curve slopes than the Hoek-Brown envelopes.
Florida rocks have high porosities and typically behave ductile, especially under high pressure
Hoek-Brown equation was developed for brittle rupture.
Florida envelopes would ONLY match w/ Hoek-brown in the brittle rupture behavior

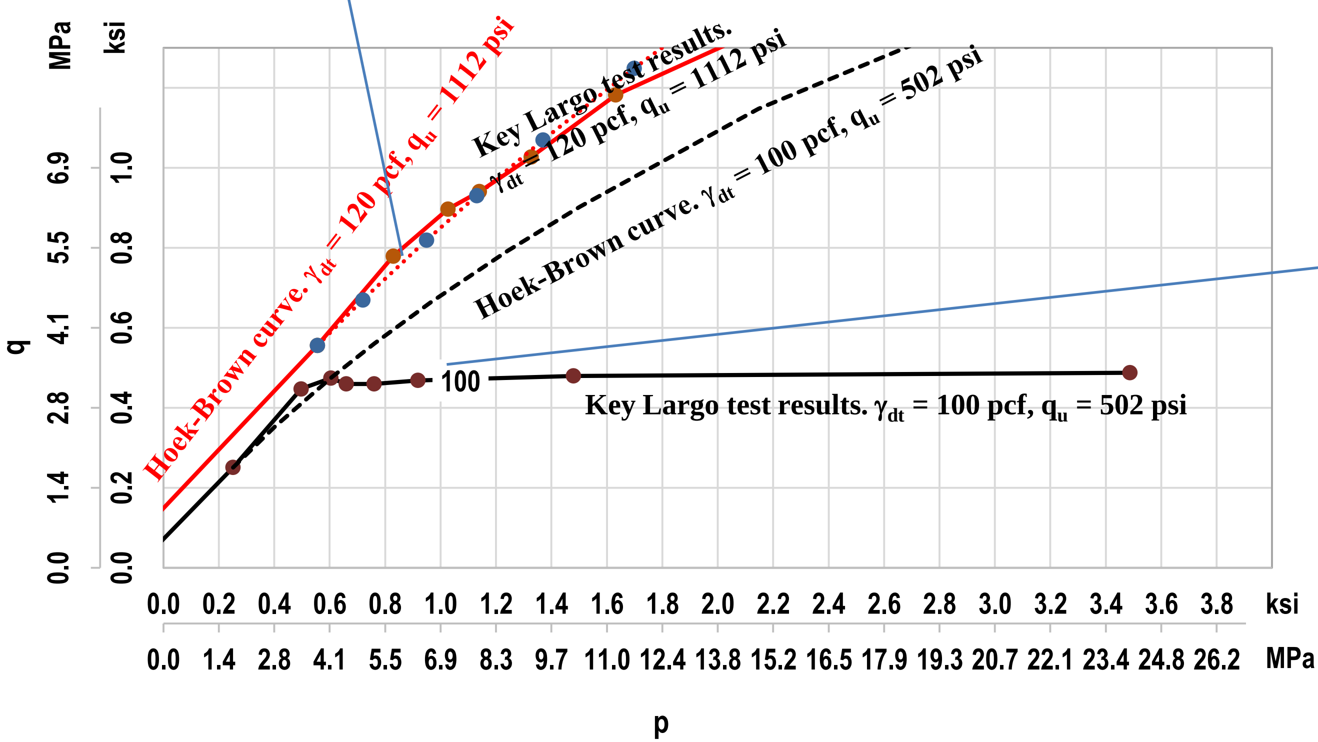


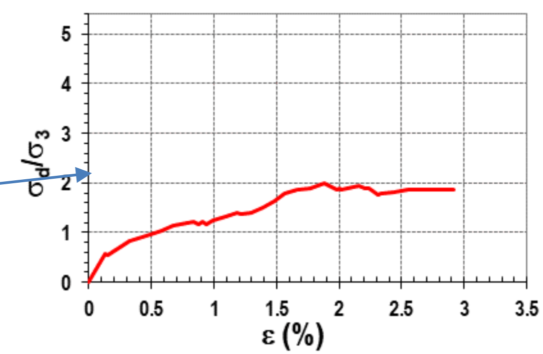
Strength envelopes – Key Largo formation

Mean Strength Envelopes - Key Largo



Florida envelopes would ONLY match w/ Hoek-brown in the brittle rupture behavior, not in the ductile behavior

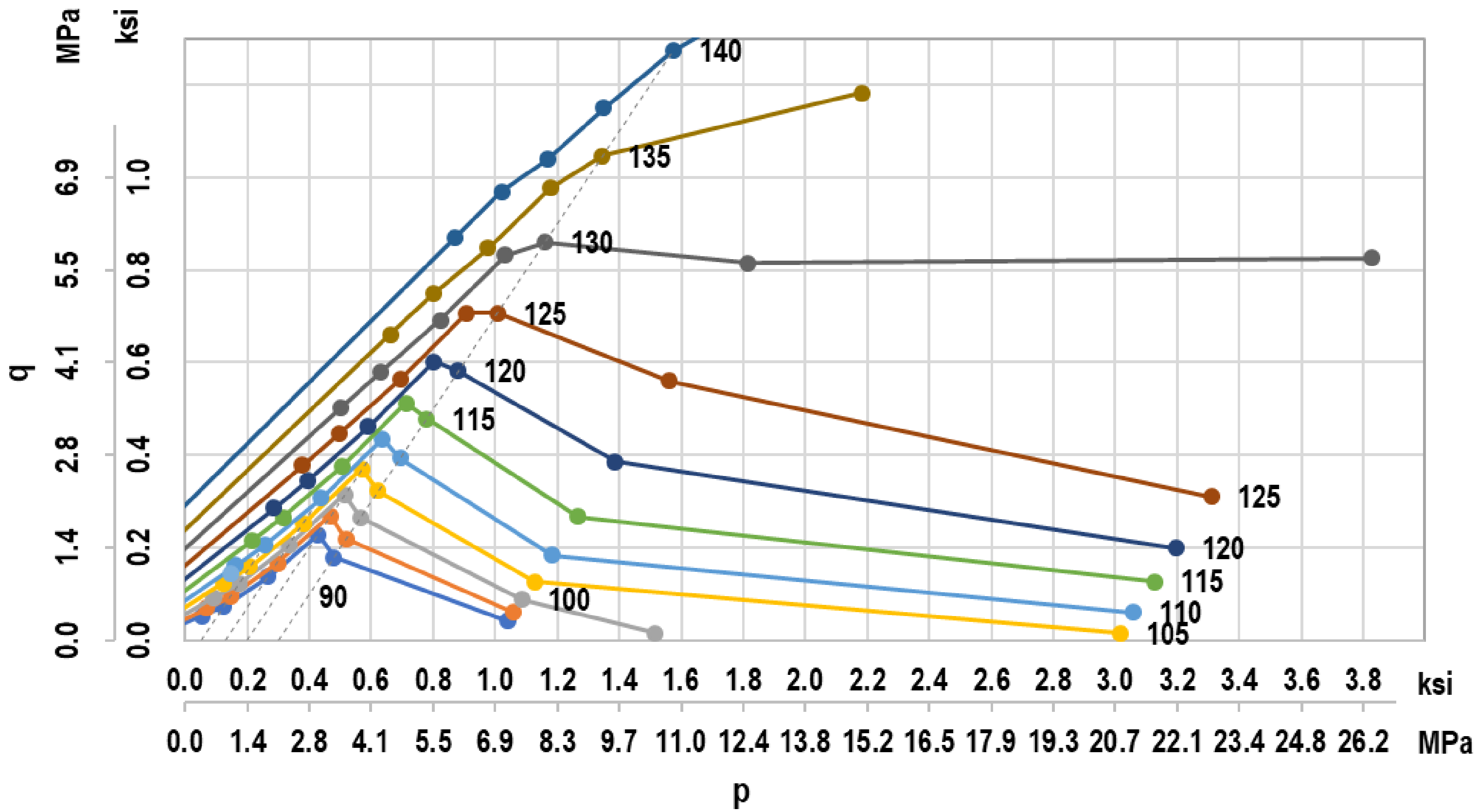




Strength envelopes – Key Largo formation

Mean Strength Envelopes - Anastasia

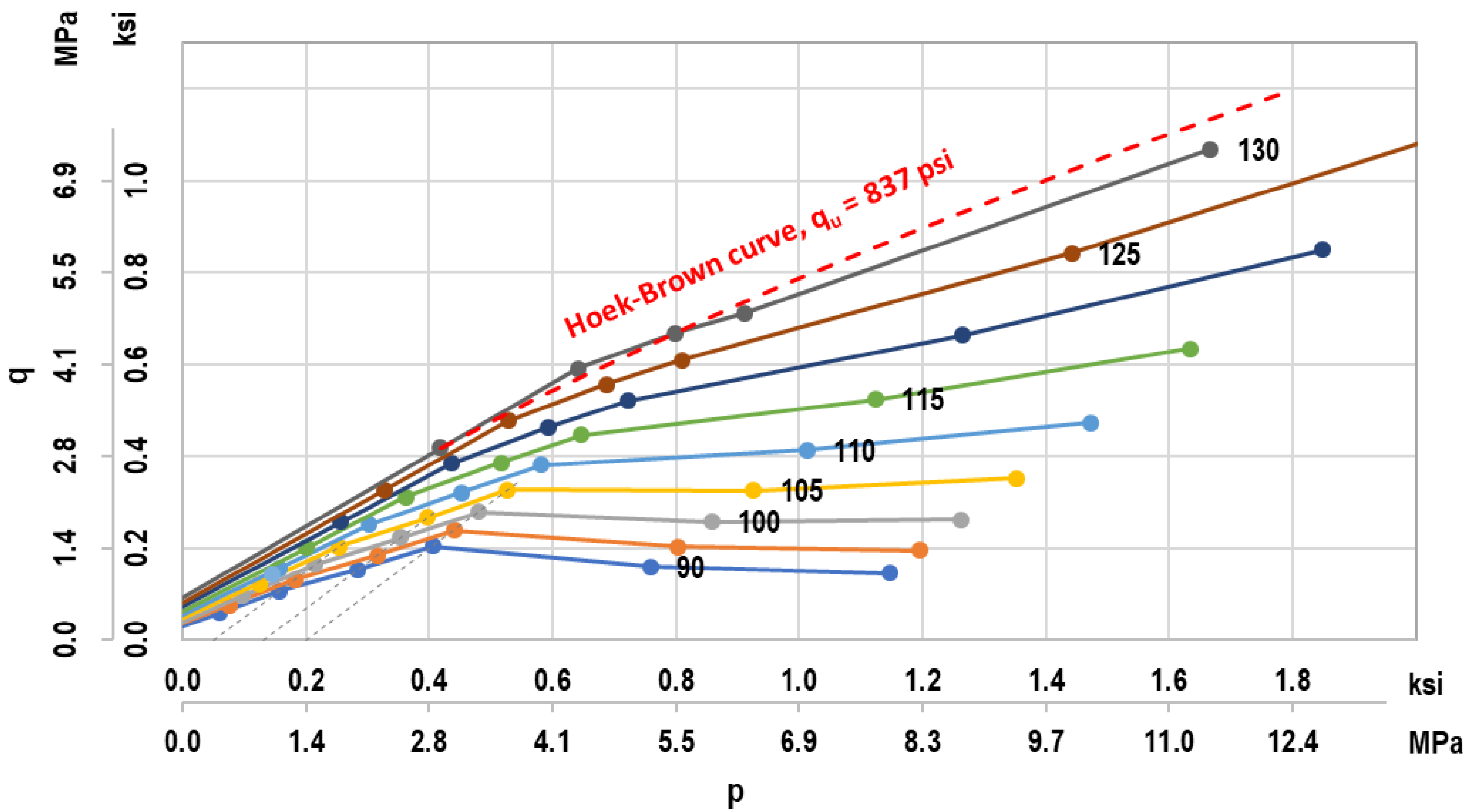
Again, Florida envelopes would match w/ Hoek-brown in the brittle rupture behavior zone. But in ductile zone, Florida envelopes have steeper downward curve slopes



Strength envelopes – Anastasia formation

Mean Strength Envelopes - Miami

Again, Florida envelopes would match w/ Hoek-brown in the brittle rupture behavior zone.
In ductile zone, Florida envelopes have steeper downward curve slopes



Strength envelopes – Miami formation

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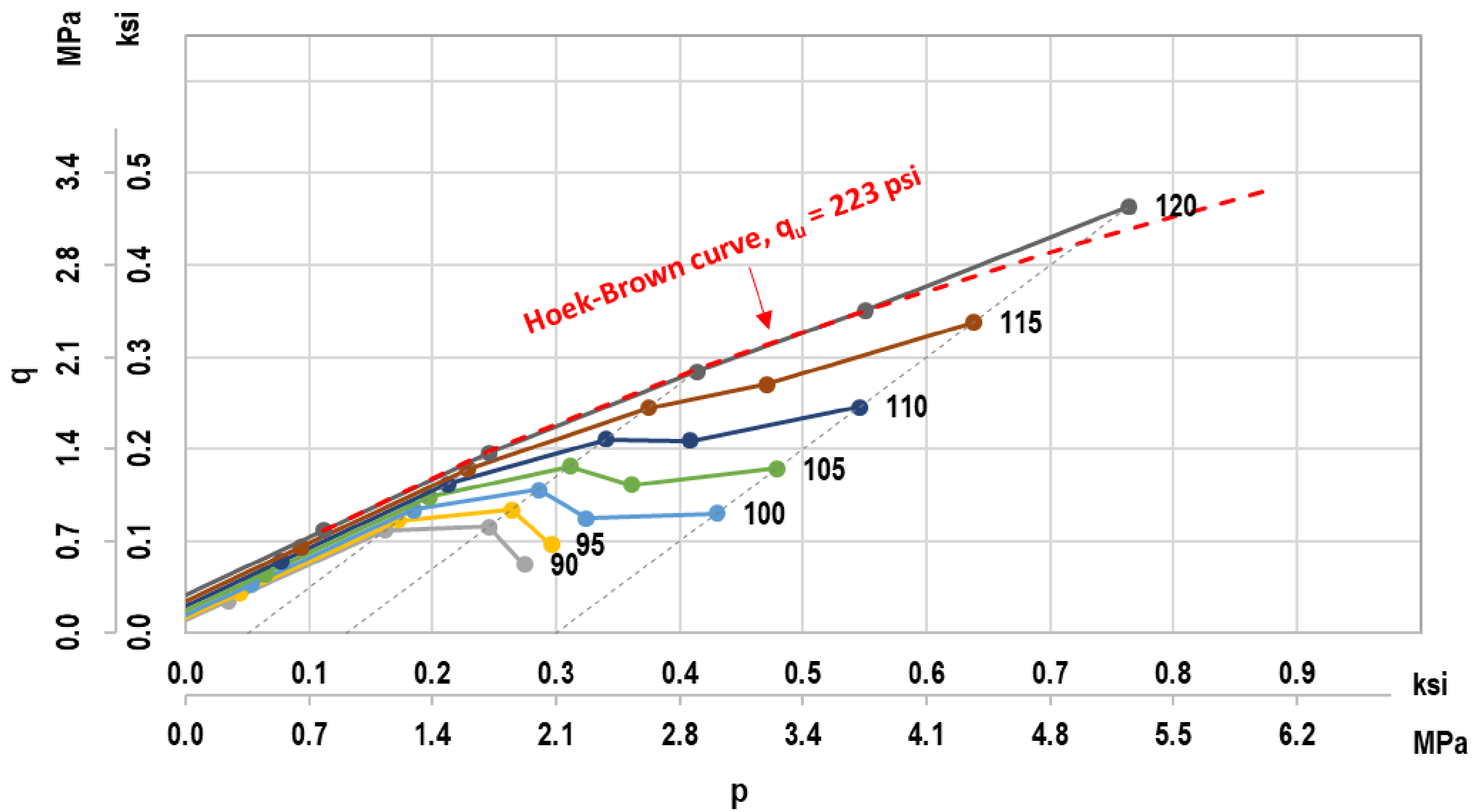
Mean Strength Envelopes – Ft Thompson

Again, Florida envelopes would match w/ Hoek-brown in the brittle rupture behavior zone.
In ductile zone, Florida envelopes have steeper downward curve slopes



Strength envelopes – Shallow Ft Thompson formation

SUMMARY

- **Florida carbonate materials are very porous (up to 55% void).**
- **Unconfined strength (i.e., BST or q_u) as well as confined strength depend on:**
    1. **Cementation is influenced by binder (carbonate content and mineral)**
    2. **Grain size (i.e., calcarenite) versus microcrystalline grain size (calcite or dolomite)**

CONCLUSIONS

1) Florida strength envelope:

   a) is not defined by conventional strength envelopes (i.e., Hoek-Brown using GSI index)

   b) Is a function of:

      a) Porosity (bulk porosity, vug porosity, permeable porosity)

      b) Carbonate formation :

         - Carbonate contents (which controls the cementation binder)
         - Mineral structure: Mineral type and grain size: i.e., sand grain size (calcarenite) versus microcrystalline grain size (calcite or dolomite)

2) Florida strength envelopes have steep curves, especially in the ductile behavior zone. Design using conventional envelopes would be unsafe

In the brittle rupture zone, the envelopes are similar to Hoek-Brown.

3) Stress – Strain behaviors (ductile or brittle) depend on:

- Porosity (dry unit weight)
- Confining pressure

Most Florida carbonate rocks, would behave as ductile (i.e., can be modelled as elastic – perfectly plastic)

4) Volumetric response:

- Brittle stress-strain behavior tends to have dilative volumetric response
- Ductile stress-strain behavior tends to have contractive volumetric response

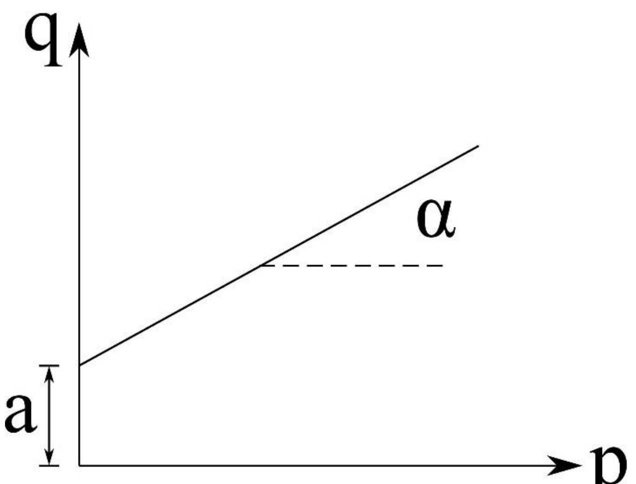
RECOMMENDATION

RECOMMENDATIONS FOR DESIGN

- **Engineer to perform index testing: Porosity, and/or specific gravity/ carbonate contents**
- **Conventional BST and q_u strength tests. These are still the 2 points on the envelopes.**
- **Limited triaxial tests to verify a selection of appropriate strength envelope on Intact Rock**
- **Strength envelope for the Rock Mass:**
  - **Evaluate potential void when REC is low**
  - **Reduce the weighted average dry unit weight for the mass in selecting strength envelope**
  - **Reduce the strength envelope based on REC or RQD**

Numerical Modeling

1. Validate the numerical simulation



Vesic's Bearing capacity equations:

$$q_{u,B.C.} = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma,$$

Where

c: cohesion intercept

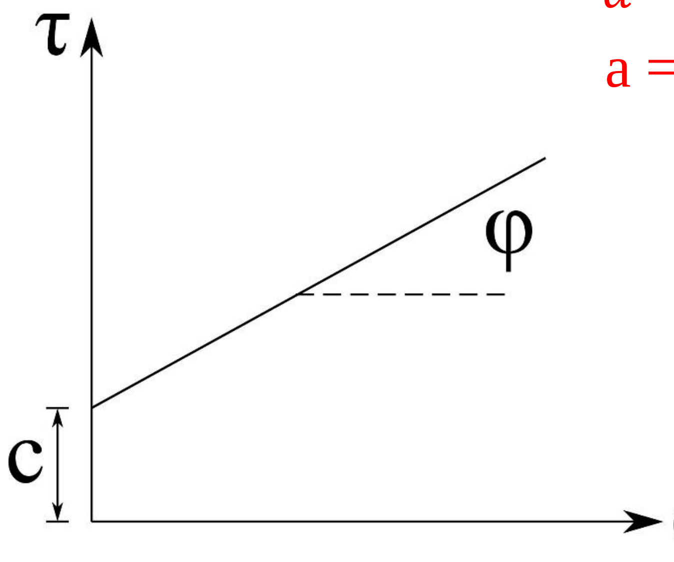
γ: dry unit weight of soil

D: foundation depth

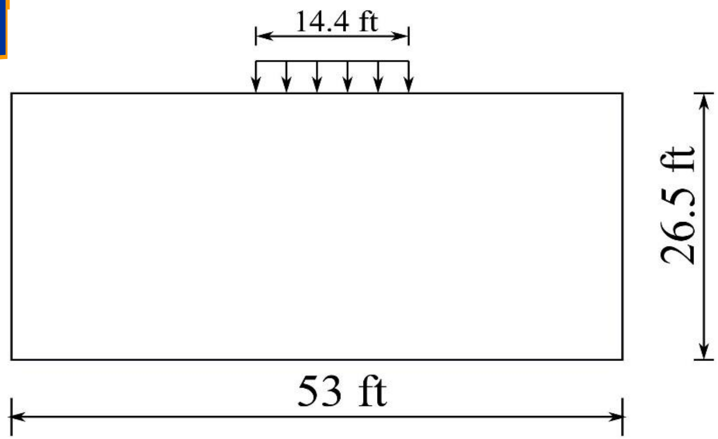
B: width of foundation

$\alpha = tan^{-1}(\sin\varphi)$

$\mathrm{a} = \mathrm{c}\cos\varphi$



Lambe & Whitman (c, phi = 0) Strip Footing



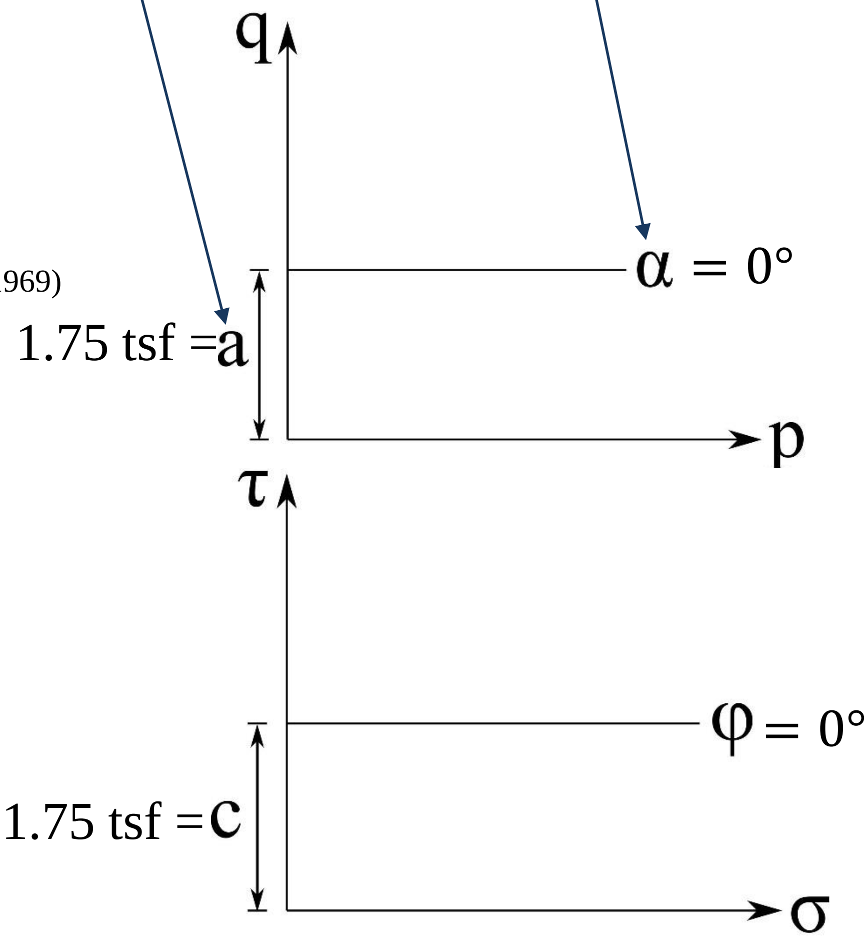
Geometry of simulation model (replot from Lambe and Whitman, 1969)

Table 1 Material parameters used in the simulation

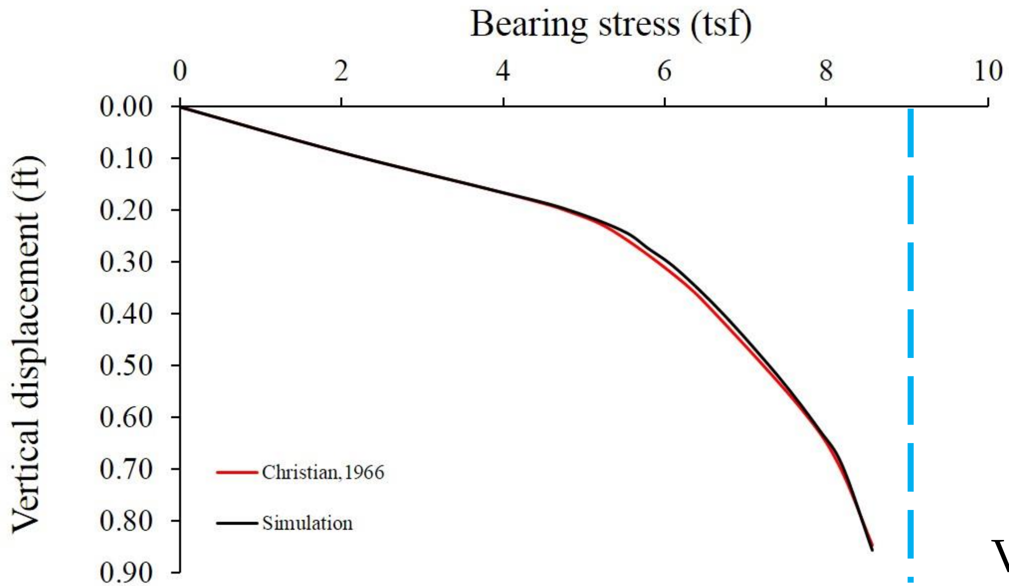
porosity	0.4
limestone density	110 pcf
Young's modulus	5000 psi
Poisson's ratio	0.3
Friction angle	0°
cohesion	1.75 tsf

$a = c \cos\varphi$

$\alpha = tan^{-1}(\sin\varphi)$

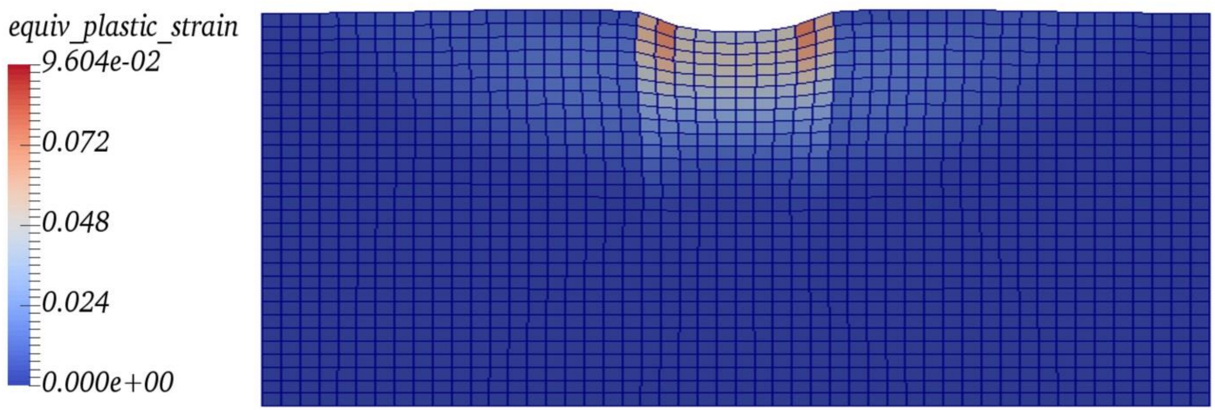


FEM result vs. Lambe & Whitman vs. Vesic B.C.



Vesic:

$$q_{u,B.C.} = cN_c = (1.75\ tsf)(5.1) = 8.9\ tsf$$



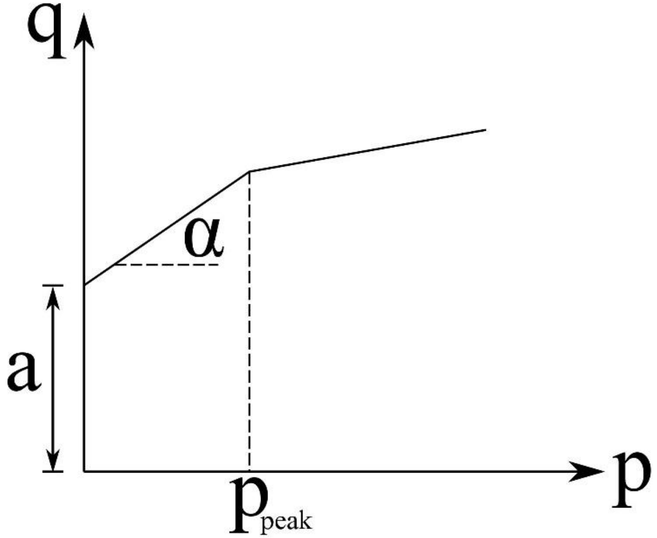
Footing Numerical Simulation

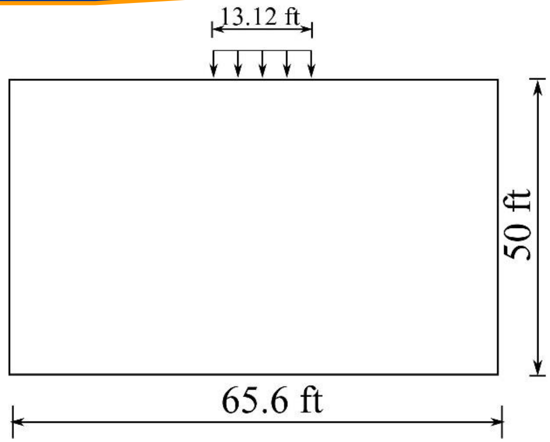
I. Simulations of Strip Footings under same cohesion, but different phi (strength envelope curvatures).

II. Simulations of Strip Footings for Florida carbonate-rocks, for strength envelopes at Different Dry Unit Weights (90, 100, and 110 pcf)

III. Simulations are performed at 3D conditions vs. Strip

Bearing Capacity (Cohesion and phi to P_{peak} then 8^0 slope)





Geometry of simulation model

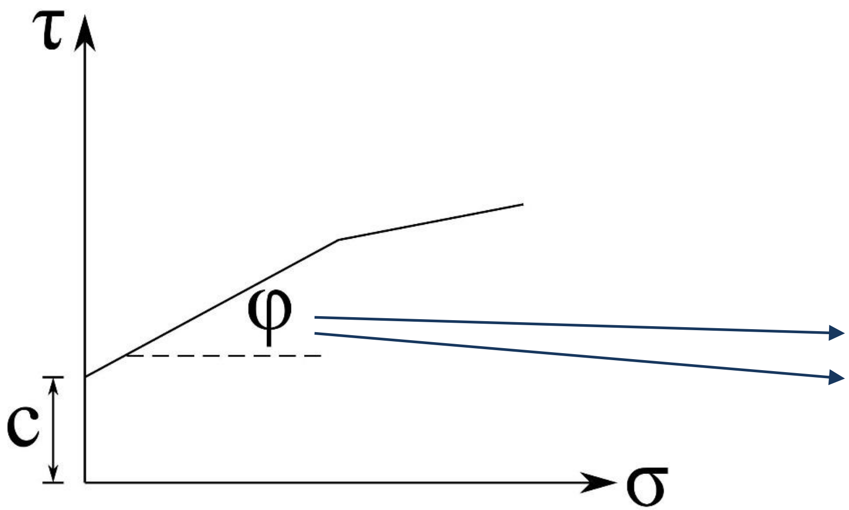
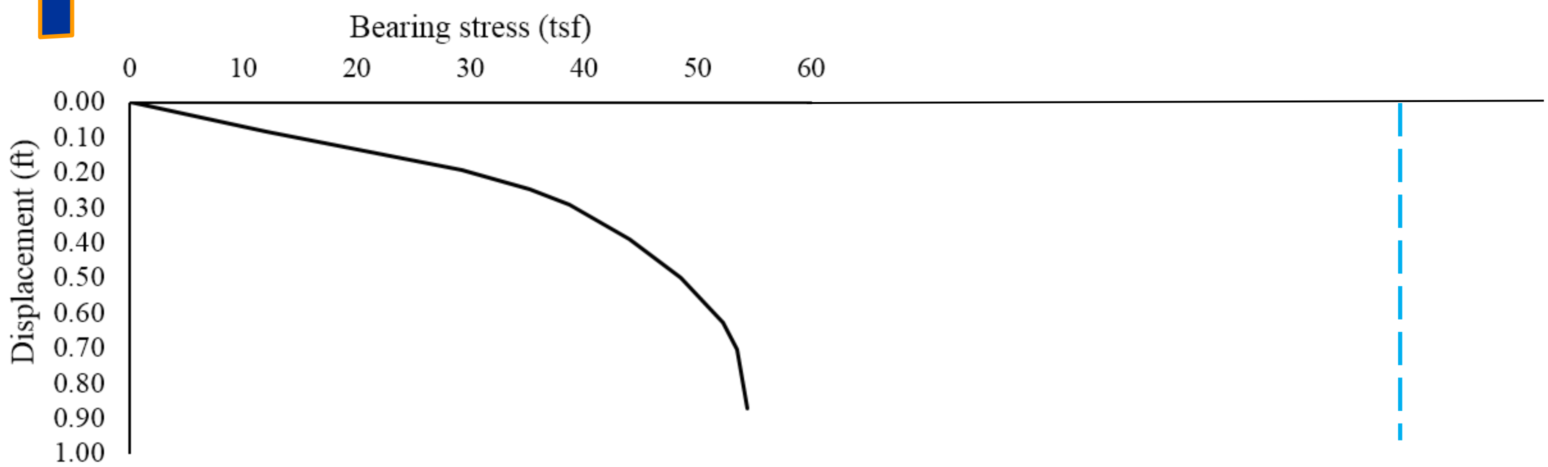


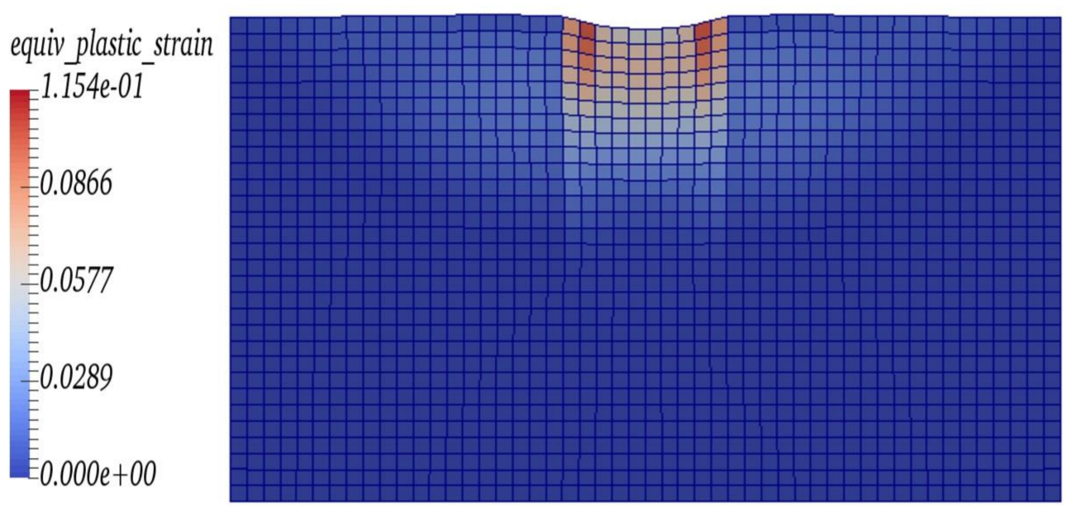
Table 2 Material parameters used in the simulation

porosity	0.41
limestone density	$100 pcf$
Young's modulus	$5266 tsf$
Poisson's ratio	0.3
Slope 1 (in σ–τ space)	45°
Slope 2(in σ–τ space)	8°
Cohesion	$7.56\ tsf$
p_{peak}	$31.2\ tsf$

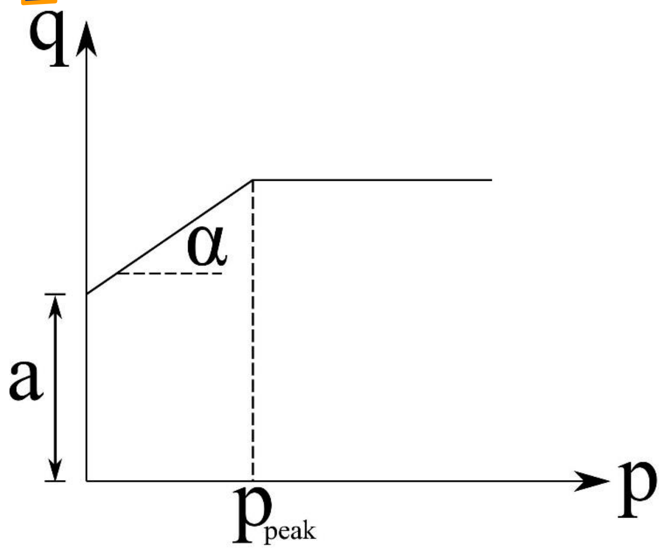
Bearing Capacity (Cohesion and phi to P_{peak} then 8^0 slope)

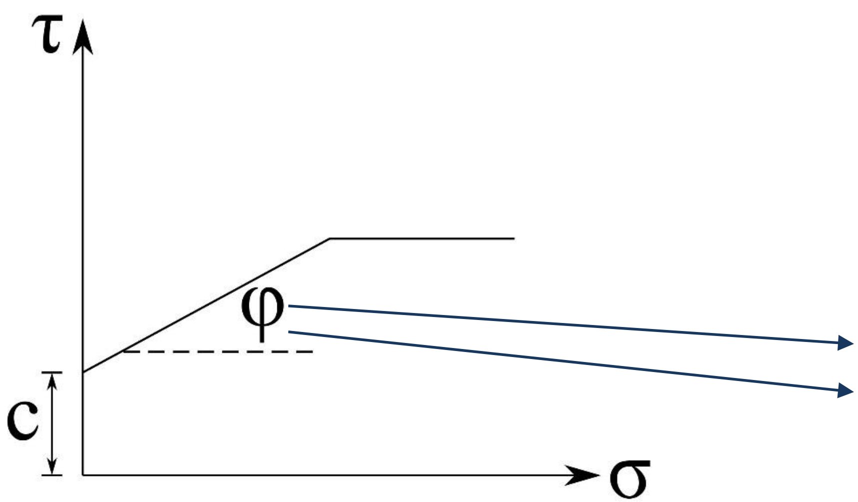


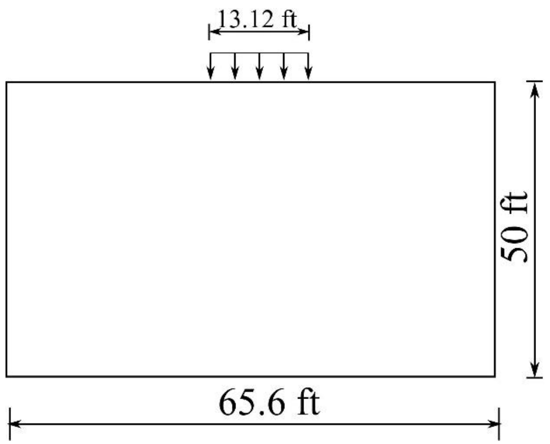
Vesic: $q_{u,B.C.} = cN_c + 0.5\gamma BN_\gamma = (7.56tsf)(5.1)$
$+0.5(100)(13.12)(271)(0.0005) = 127.4\ tsf$



Bearing Capacity (Cohesion and phi to P_{peak} then constant)







Geometry of simulation model

Table 2 Material parameters used in the simulation

porosity	0.41
limestone density	$100pcf$
Young's modulus	5266 tsf
Poisson's ratio	0.3
Slope 1 (in σ−τ space)	45°
Slope 2(in σ−τ space)	0°
Cohesion	7.56 tsf
p_{peak}	31.2 tsf

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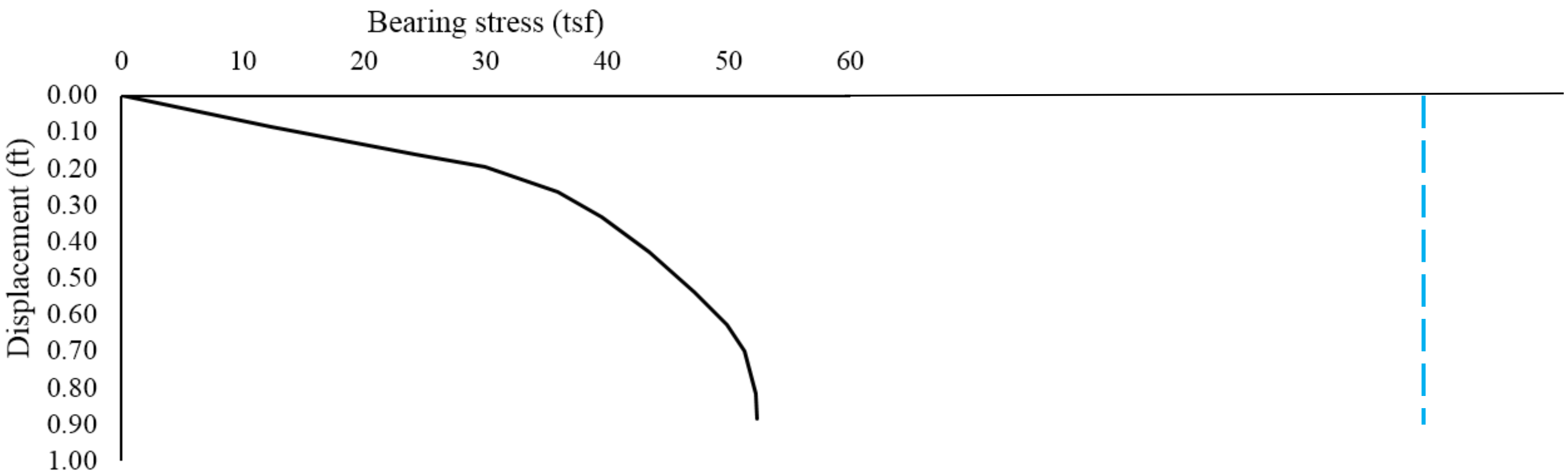
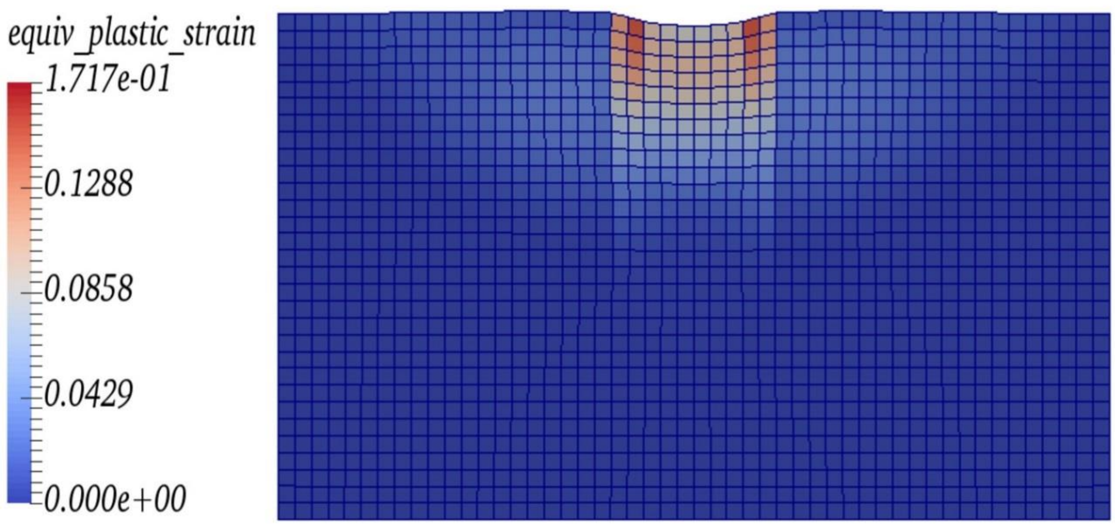
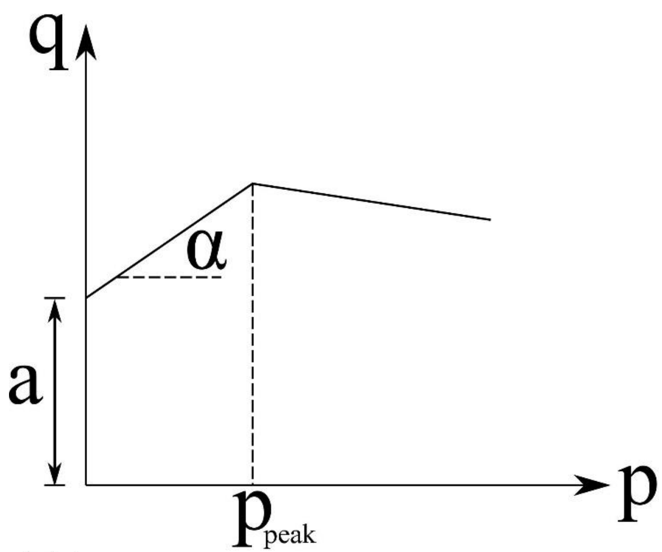
Bearing Capacity (Cohesion and phi to P_{peak} then constant)

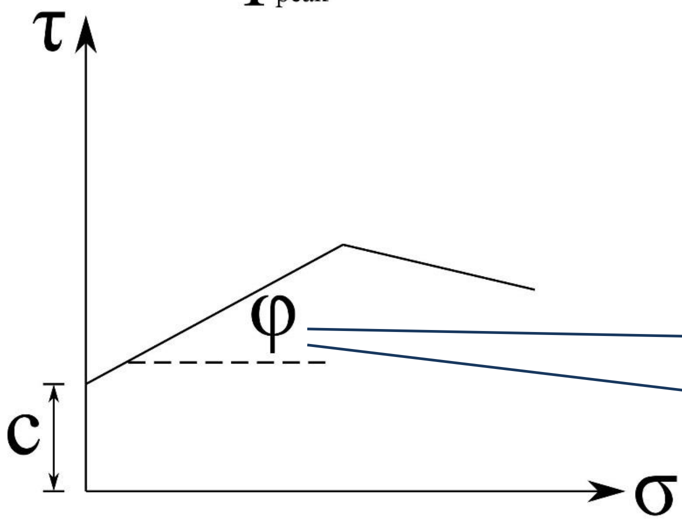


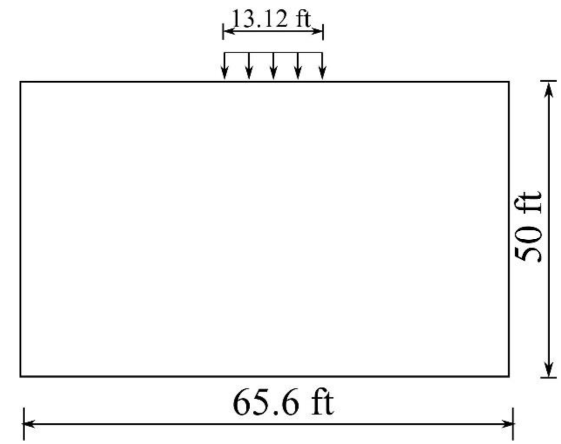
Vesic: $q_{u,B.C.} = cN_c + 0.5\gamma B N_\gamma = (7.56\ tsf)(5.1)$
$+0.5(100)(13.12)(271)(0.0005) = 127.4\ tsf$



Bearing Capacity (Cohesion and phi to P_{peak} then -5.7^0 slope)





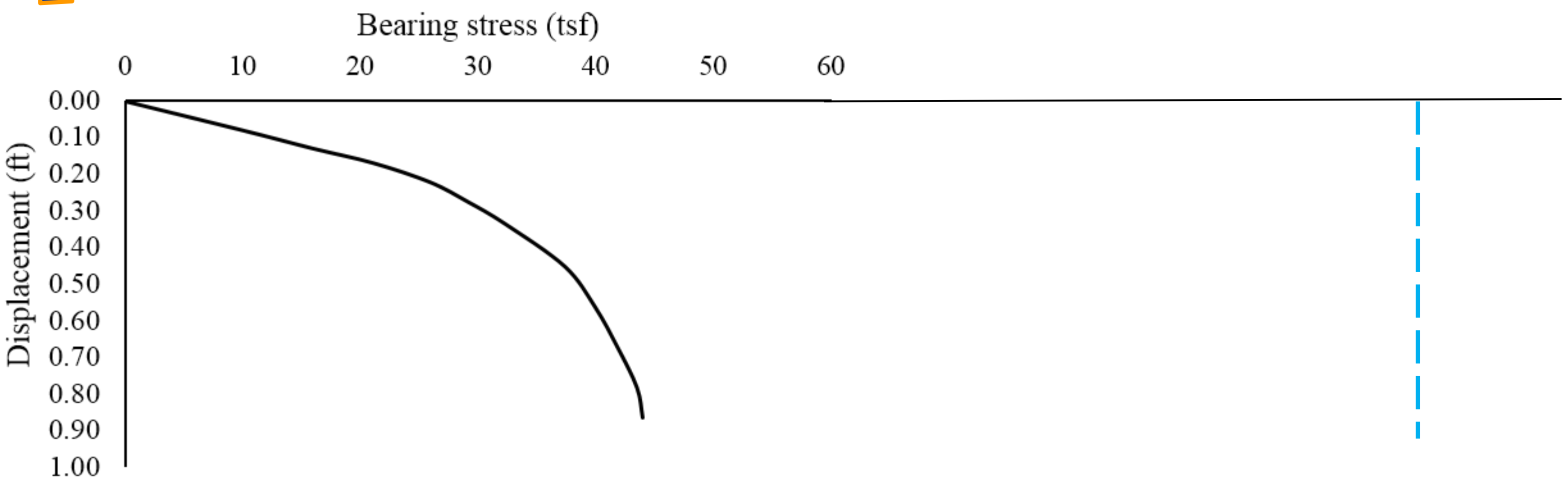


Geometry of simulation model

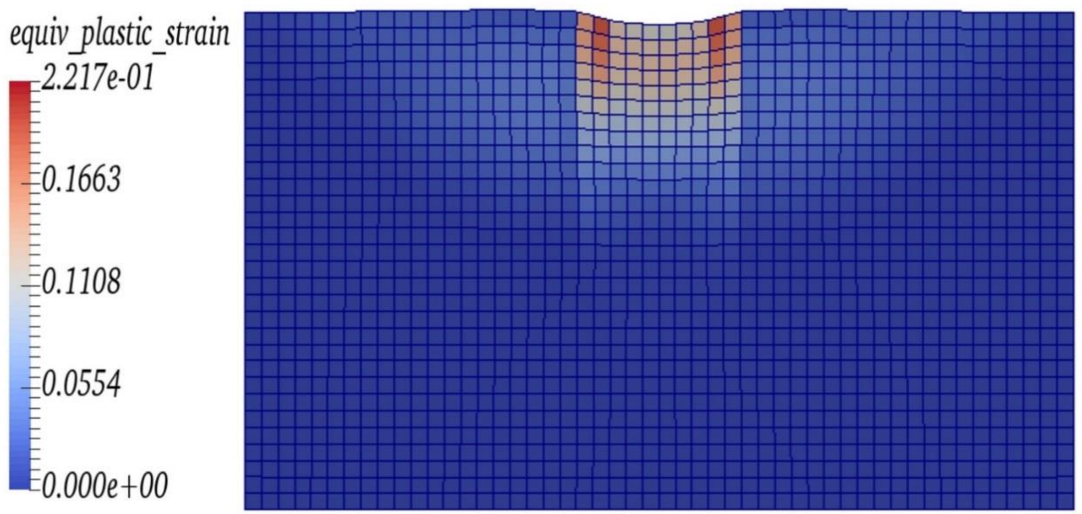
Table 2 Material parameters used in the simulation

porosity	0.41
limestone density	100pcf
Young's modulus	5266 tsf
Poisson's ratio	0.3
Slope 1(in σ–τ space)	45°
Slope 2(in σ–τ space)	−5.7°
Cohesion	7.56 tsf
p_{peak}	31.2 tsf

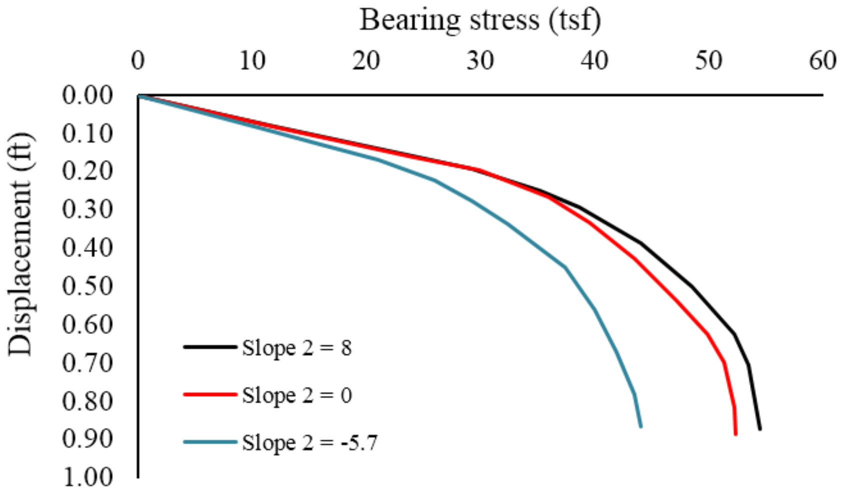
Bearing Capacity (Cohesion and phi to P_{peak} then -5.7^0 slope)

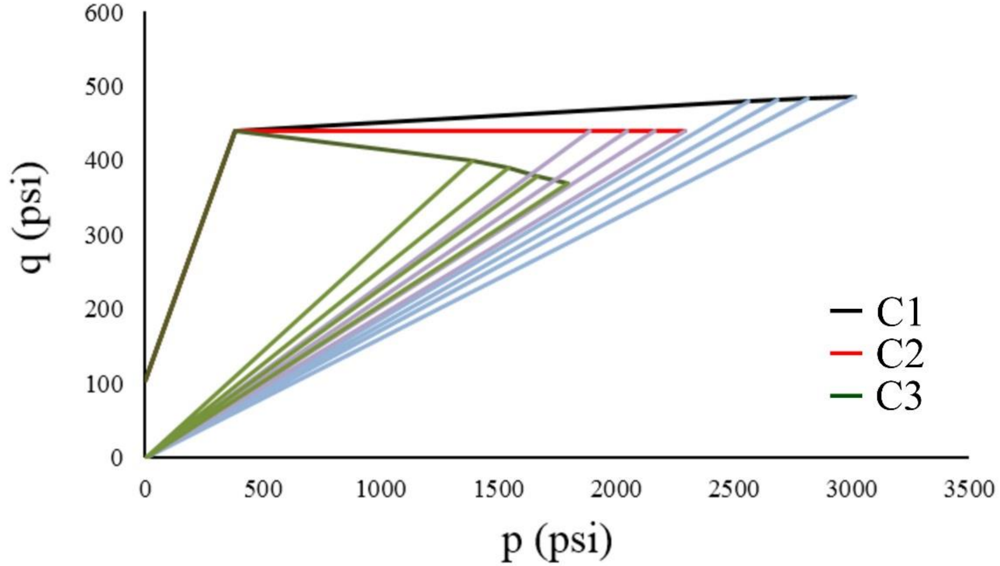


Vesic: $q_{u,B.C.} = cN_c + 0.5\gamma B N_\gamma = (7.56\,tsf)(5.1)$
$+0.5(100)(13.12)(271)(0.0005) = 127.4tsf$

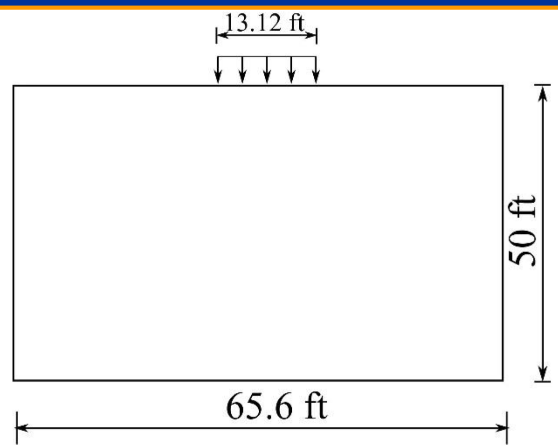


Simulating Florida Limestone



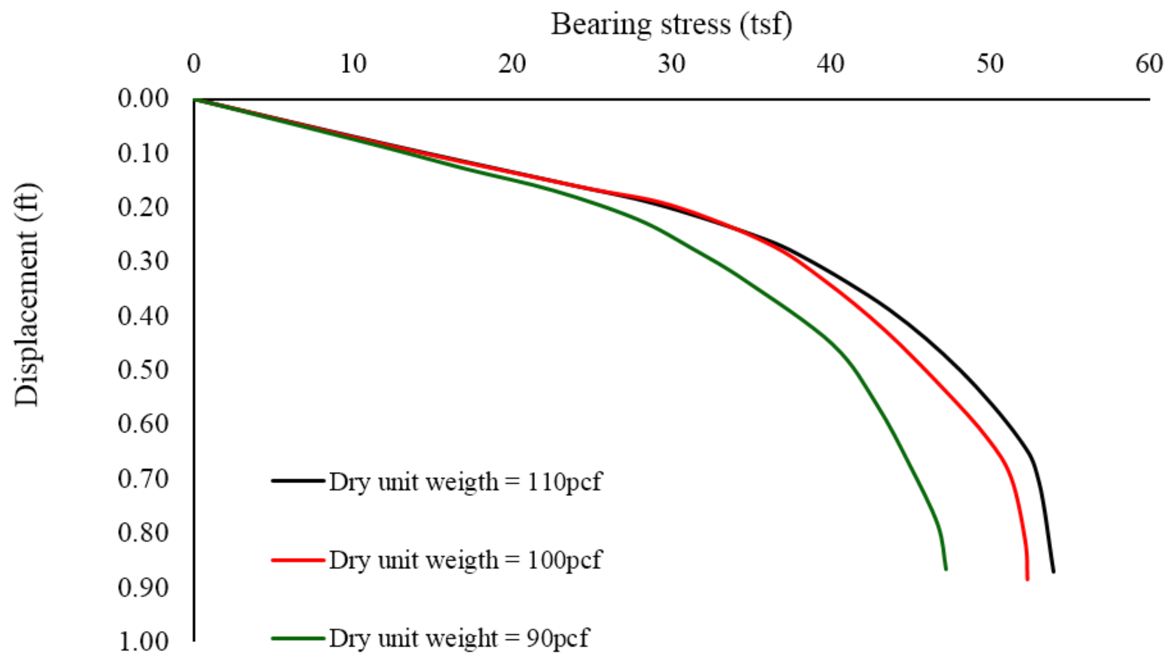


Simulating Florida Limestone– 3 Different Unit Weights



porosity	0.47	0.41	0.35
limestone density	$90pcf$	$100\ pcf$	$110pcf$
Young's modulus	$3530tsf$	$5266tsf$	$7855tsf$
Poisson's ratio	0.3	0.3	0.3
"Slope 1"	43.6°	45°	47.6°
"Slope 2"	-5.7°	0°	8°
p_{peak}	$24.0\ tsf$	$31.2\ tsf$	$41.4\ tsf$
cohesion	$5.33\ tsf$	$7.56tsf$	$10.73tsf$

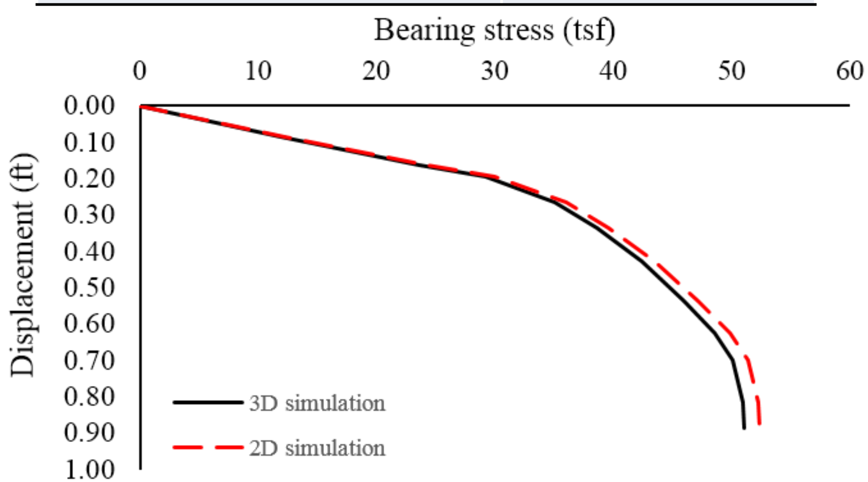
Simulating Florida Limestone– 3 Different Unit Weights

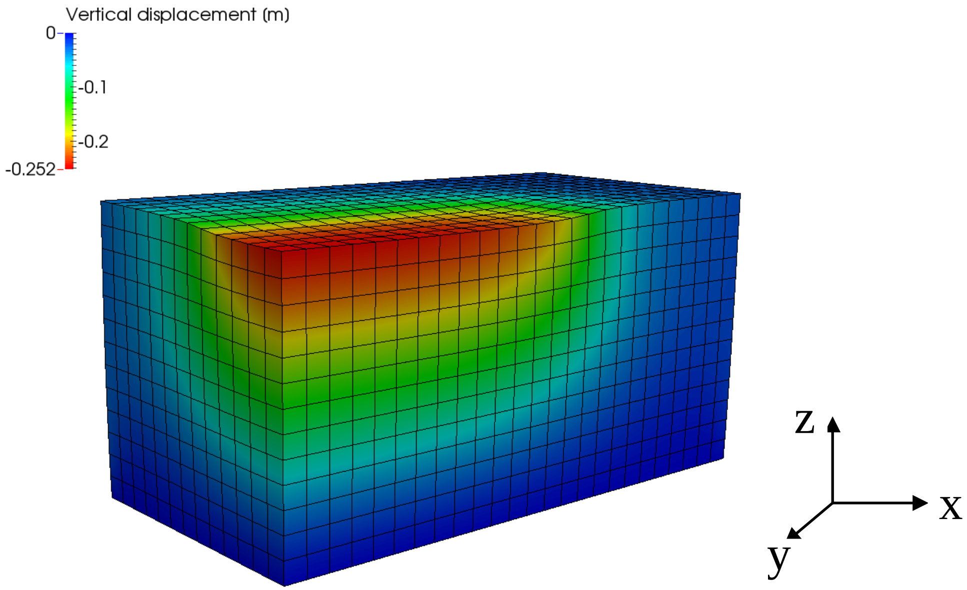


Numerical modeling

3D footing simulation

porosity	0.41
limestone density	$100pcf$
Young's modulus	$5266tsf$
Poisson's ratio	0.3
"Slope 1"	45°
"Slope 2"	0°
p_{peak}	$31.2\ tsf$
cohesion	$7.56\ tsf$





Deformed shape of soil layer (L/B=5)

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Future Numerical Work

I. Run Simulation of Bearing Capacity for Different Formations Based on Unit Weight (C, phi, P_{peak}, and slope 2) as function of L/B and Embedment Depth, D

II. Implement Sand Model into FEM and Repeat simulations of different formations [i.e. unit weight (C, phi, P_{peak}, and slope 2)] with T (limestone Thickness) function of B (footing width)

III. Develop Bearing Graphs for I & II