GMEC 2015

Relaxation of Driven Pile Resistance in Granular Soils

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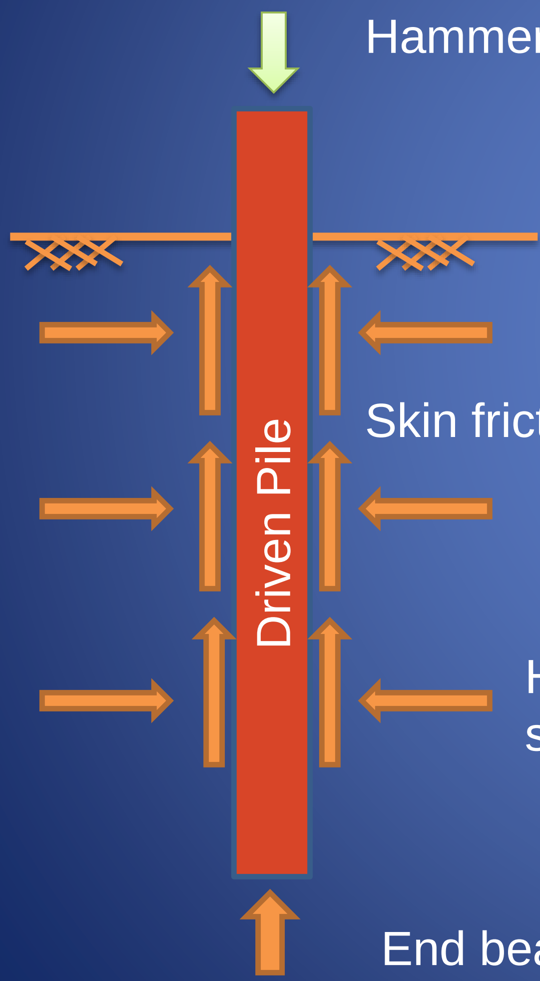
Relaxation of Driven Pile Resistance in Granular Soils

- Pile relaxation in granular soil
  - Pile driving may generate negative pore pressures in dilatant material (e.g. dense sand)
  - Negative pore pressure will produce a *temporary increase in soil strength*
  - Pile resistance decreases with time, as pore pressures return to hydrostatic conditions
  - In Florida, relaxation has been encountered typically in medium dense to very dense silty and shelly sands

OUTLINE

- Driven Pile Capacity (granular soils)
- Effective Stress
- Dilation & Relaxation
- Case Histories
- Design
- Construction

Driven Pile Capacity



Hammer impact

Granular soil

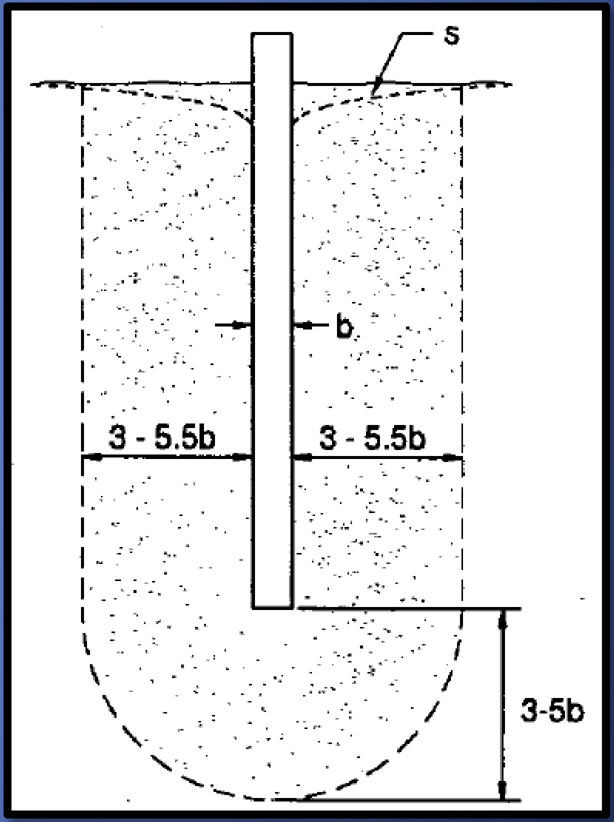
Skin friction = f($\boldsymbol{\sigma'_H}$, ϕ, ψ, δ, pile matl. OCR, time)

Horizontal stress (σ'_H)

End bearing = f($\boldsymbol{\sigma'_v}$, ϕ, ψ, δ, pile size, OCR, time)

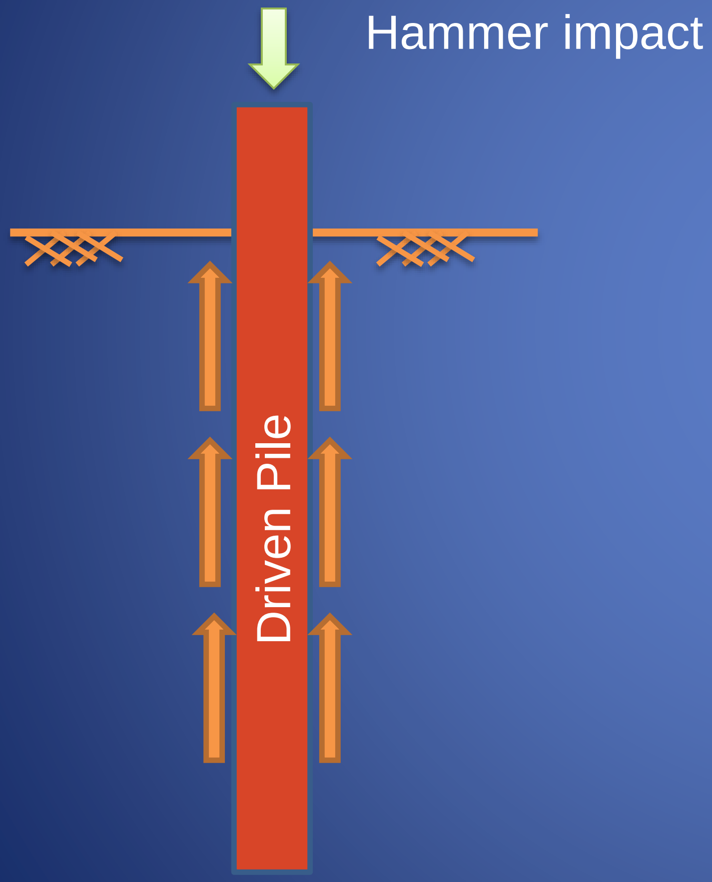
Driven Pile Capacity Densification Zone

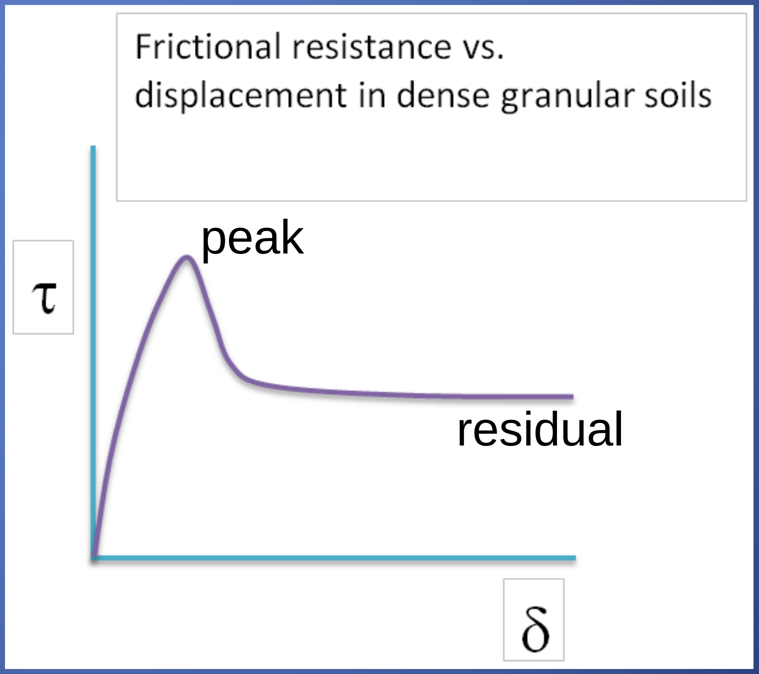
– Pile driving densifies the material (granular soils)



Zone of densification for granular soils due to pile driving. Broms, 1966

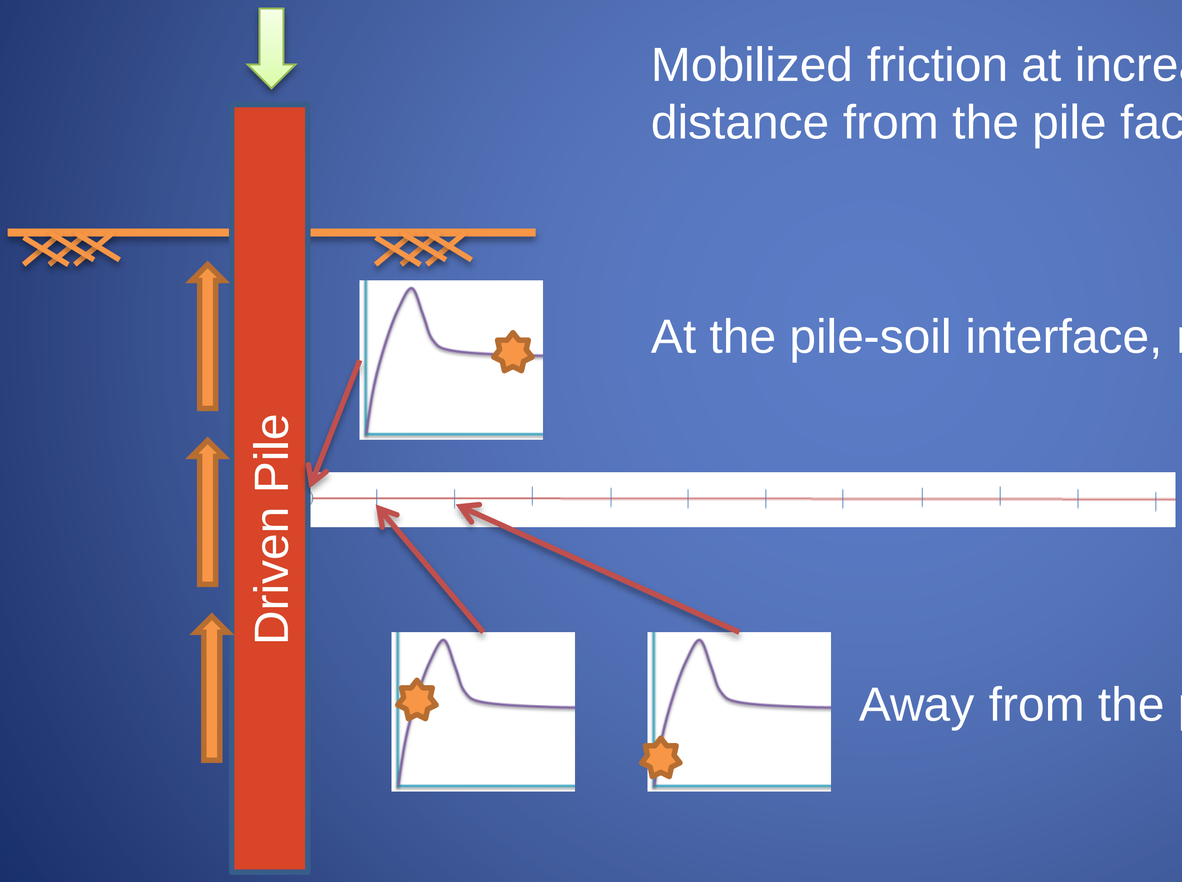
Driven Pile Capacity
Skin Friction





Shear resistance vs. **vertical** displacement

Driven Pile Capacity
Skin Friction



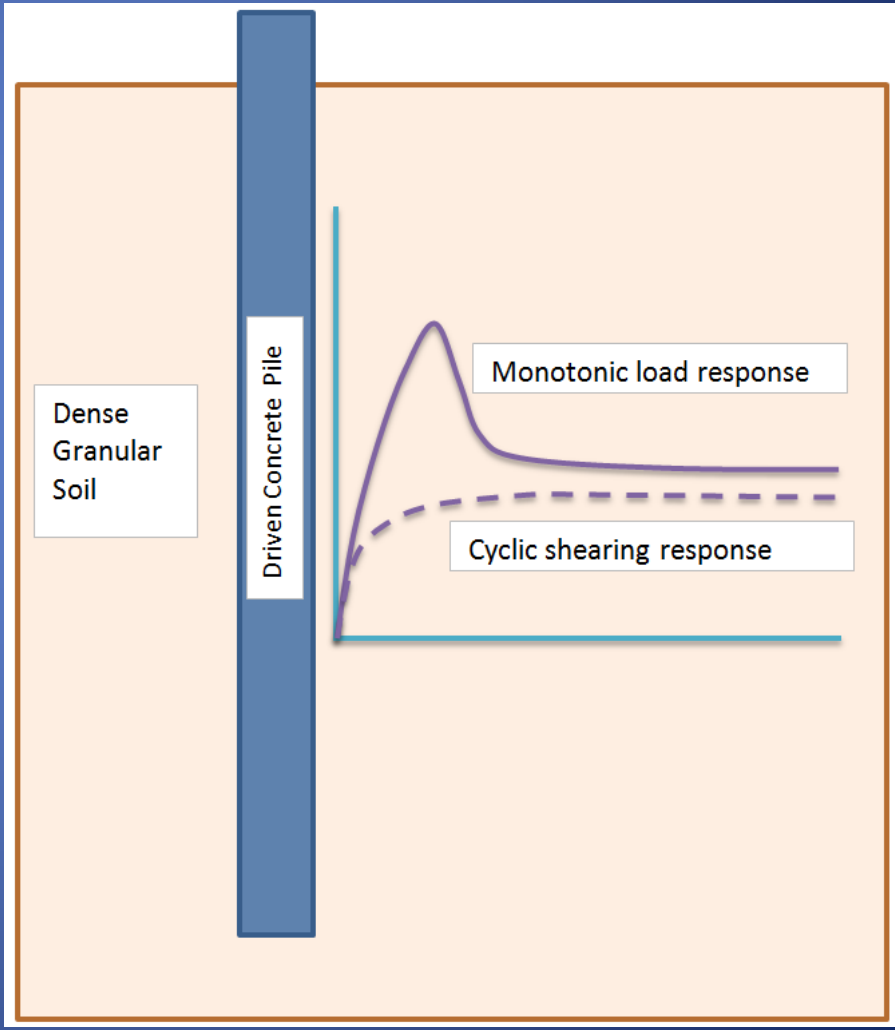
Mobilized friction at increasing **horizontal** distance from the pile face (not to scale!)

At the pile-soil interface, residual

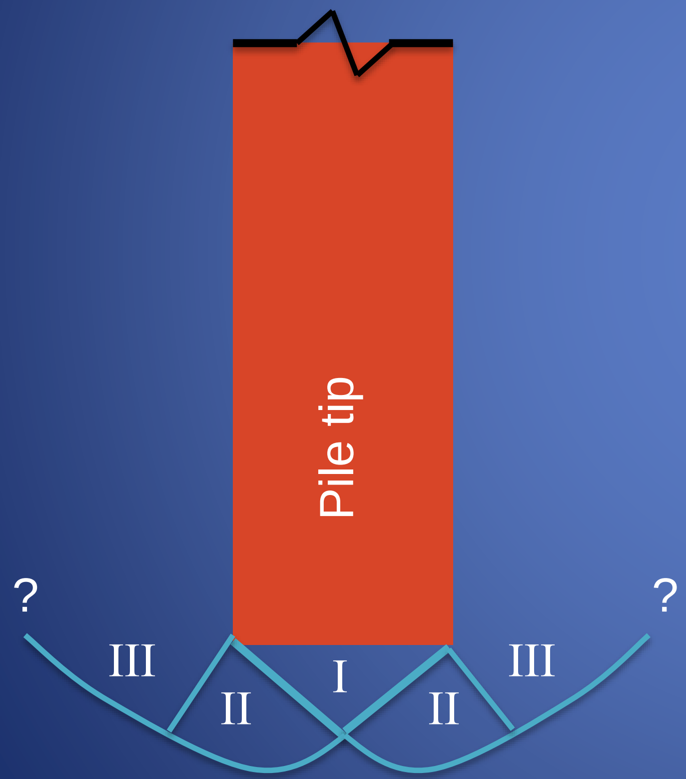
Away from the pile, from residual to zero

Driven Pile Capacity
Skin Friction

- Degradation of the residual response (fatigue)
- Possible scenario for long piles
- Not uncommon to apply > 1,000 blows



Driven Pile Capacity
End Bearing

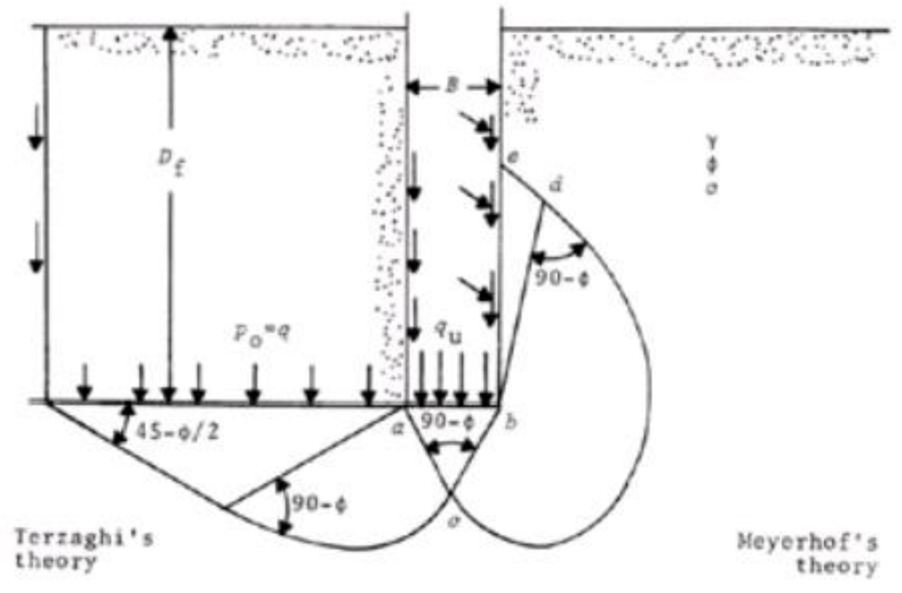


Zone I: Conical section that moves with the pile and pushes material outward during driving

Zone II: Radial shear (log-spiral surface)

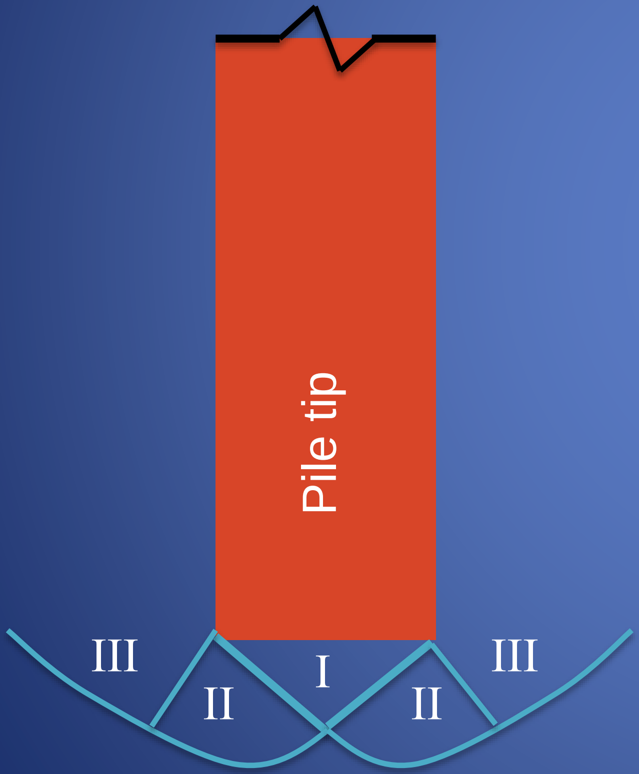
Zone III: Rankine passive

Driven Pile Capacity
End Bearing

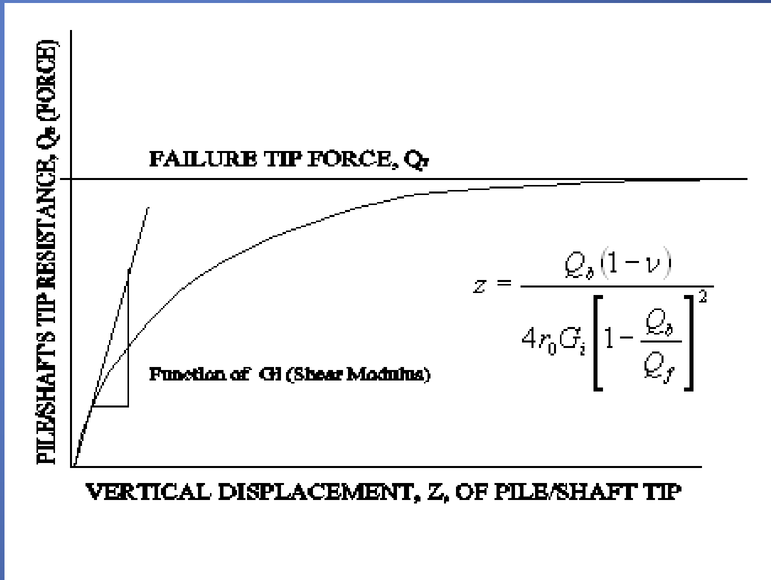


(b) Deep foundation

Driven Pile Capacity
End Bearing

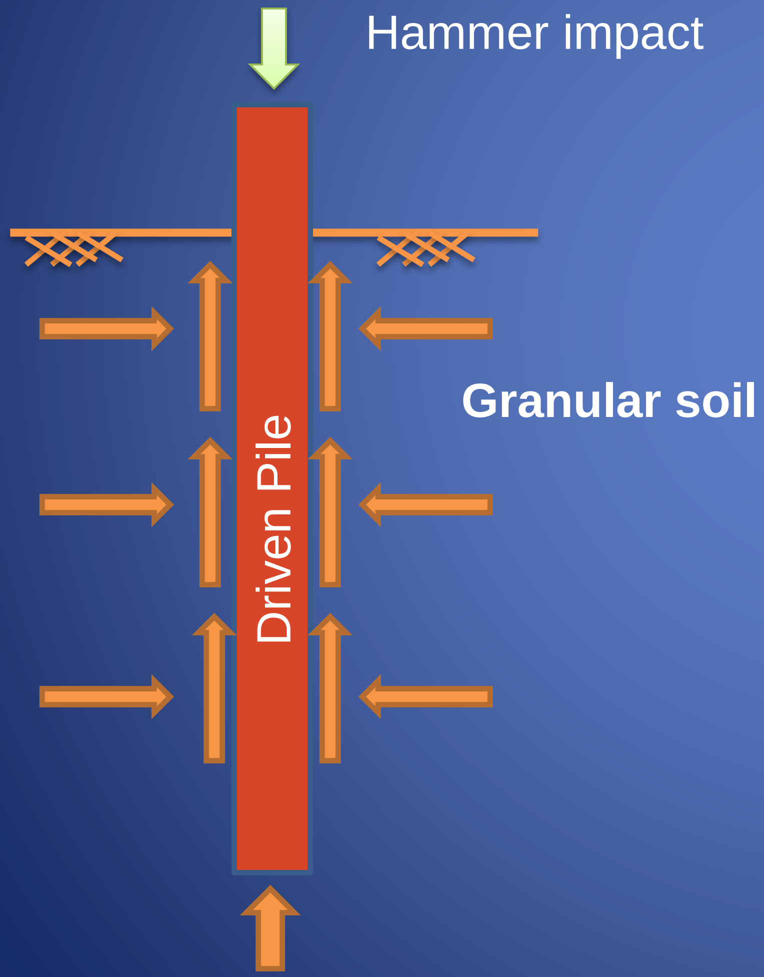


End bearing as a function of shear modulus (FB Multipier)



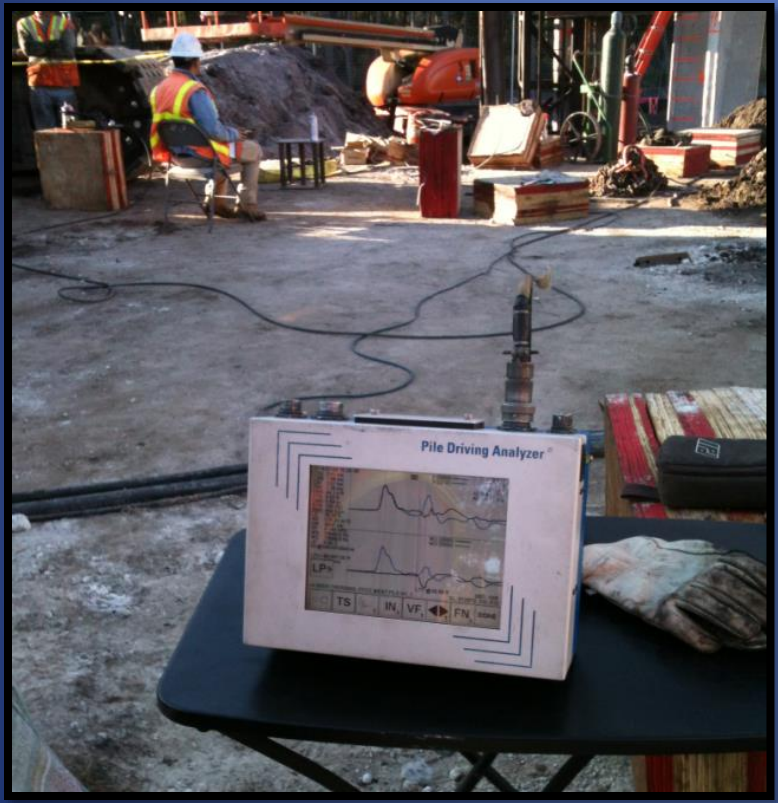
Axial T-Z (Q-Z) Curve for Driven Pile

Driven Pile Capacity Preliminary (static) Design



- In Florida; ***FBDEEP*** (empirical correlations of SPT "N" to static load tests)
- Beta method
- Nordlund/Thurman
- Vesic
- Tomlinson
- Others..

Driven Pile Capacity Construction QC



Pile Driving Analyzer (PDA)

Embedded Data Collector (EDC)

Driven Pile Capacity Design

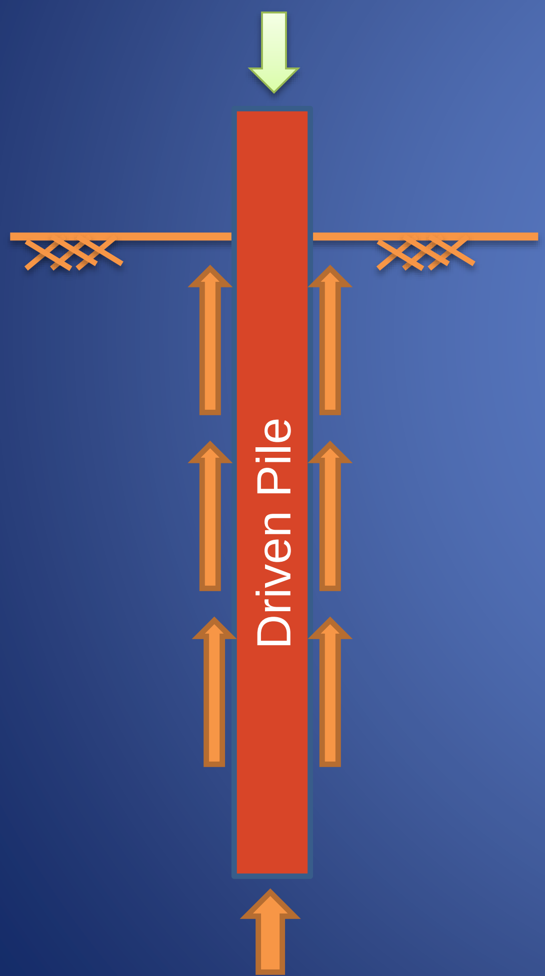


TABLE 1 Factor of safety on ultimate axial geotechnical capacity based on level of construction control (AASHTO, 1997)

Basis for Design and Type of Construction Control	Increasing Design/Construction Control				
Subsurface Exploration	X	X	X	X	X
Static Calculation	X	X	X	X	X
Dynamic Formula	X				
Wave Equation		X	X	X	X
CAPWAP Analysis			X		X
Static Load Test				X	X
Factor of Safety (FS)	3.50	2.75	2.25	2.00*	1.90

*For any combination of construction control that includes a static load test, FS =2.0.

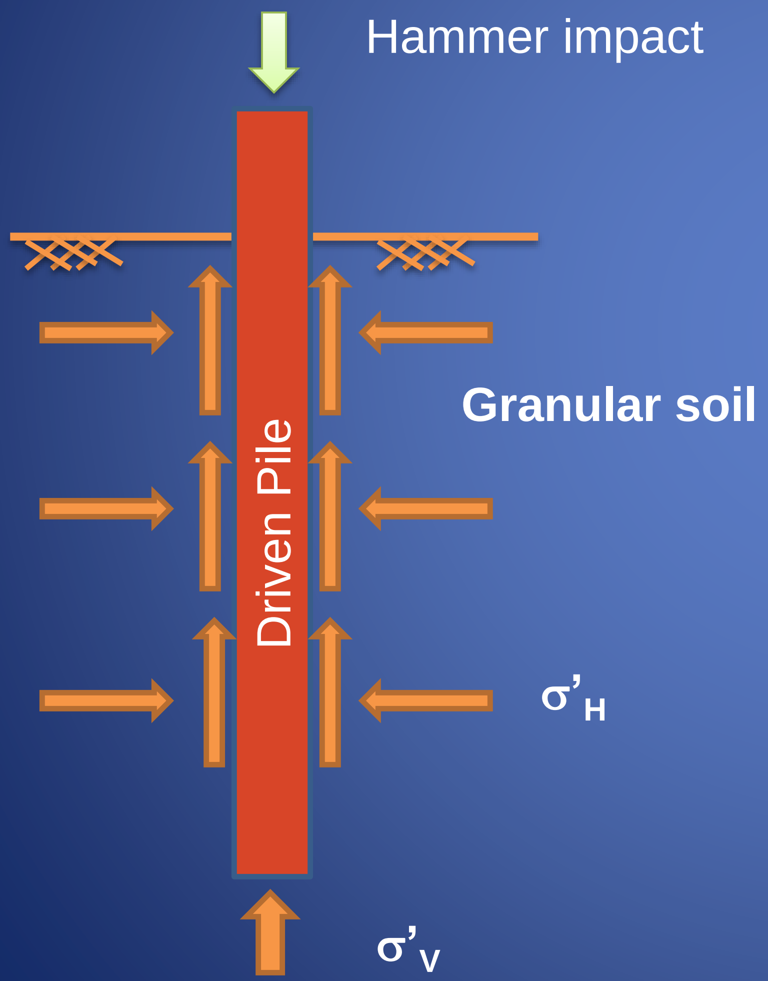
ASD (factor of safety)

Table 3.5.6-1 Resistance Factors for Piles (all structures)

Pile Type	Loading	Design Method	Construction QC Method	Resistance Factor, φ
Driven Piles with 100% Dynamic Testing	Compression	Davisson Capacity	EDC or PDA & CAPWAP	0.75
			EDC or PDA & CAPWAP & Static Load Testing	0.85
			EDC or PDA & CAPWAP & Statnamic Load Testing	0.80
	Uplift	Skin Friction	EDC or PDA & CAPWAP	0.60
			EDC or PDA & CAPWAP & Static Uplift Testing	0.65
Driven Piles with ≥5% Dynamic Testing	Compression	Davisson Capacity	Driving criteria based on EDC or PDA & CAPWAP	0.65
			Driving criteria based on EDC or PDA & CAPWAP & Static Load Testing	0.75
			Driving criteria based on EDC or PDA & CAPWAP & Statnamic Load Testing	0.70
	Uplift	Skin Friction	Driving criteria based on EDC or PDA & CAPWAP	0.55
			Driving criteria based on EDC or PDA & CAPWAP & Static Load Testing	0.60
All piles	Lateral (Extreme Event)	FBPier[1]	Standard Specifications	1.00
			Lateral Load Test[2]	1.00

LRFD (resistance factors - FDOT)

Driven Pile Capacity



- Soil resistance is a function of effective stress

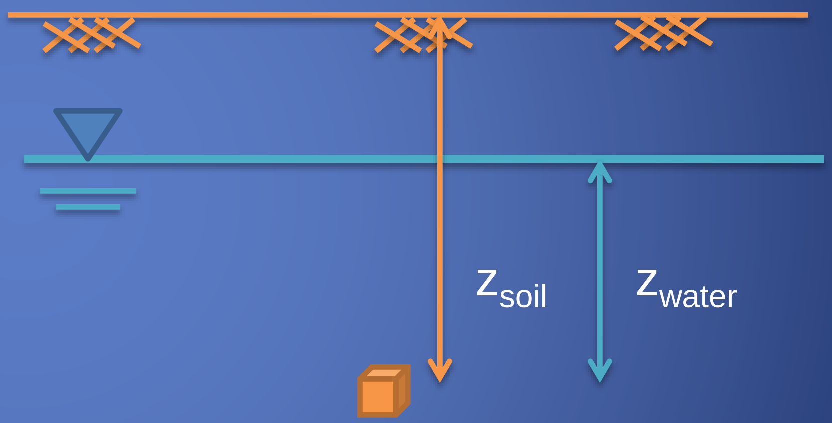
$$\sigma' = \sigma_t - U$$

Effective Stress

- $\sigma' = \sigma_t - U$

  $\sigma_t = \gamma_t \times z_{soil}$

  $U = \gamma_w \times z_{water}$



- σ' = Eff. Stress (stress carried by the soil skeleton)

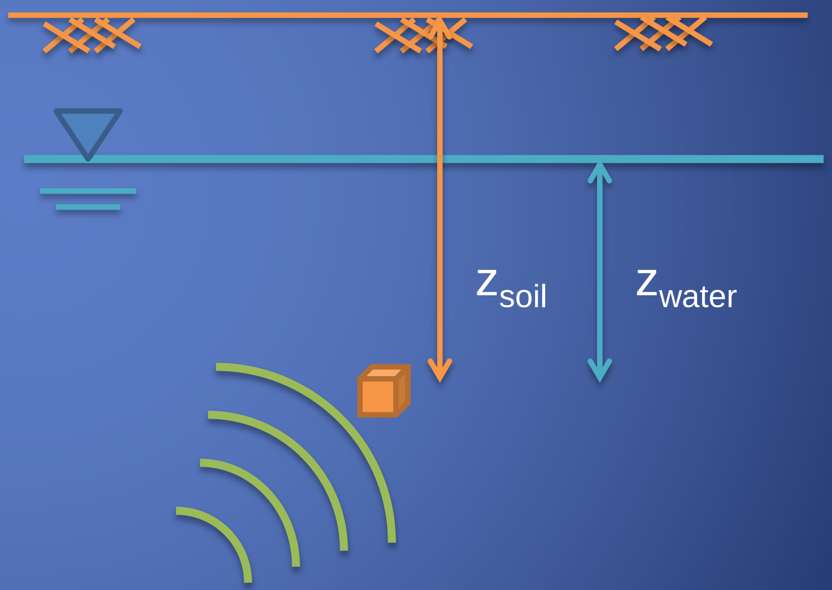
Effective Stress

- Earthquake/Blasting

  Increase in pore pressure "U"

  Temporary reduction in effective stress

  $\sigma' = \sigma_t - U$

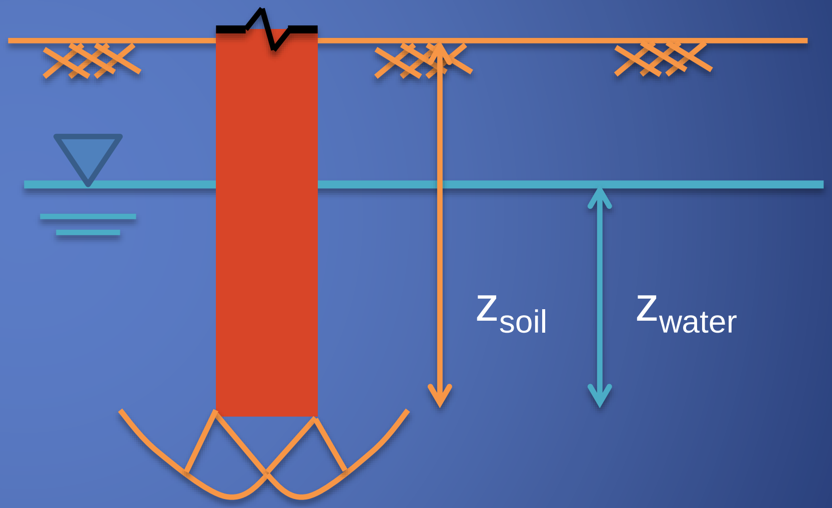


If $U = \sigma_t \rightarrow \sigma' = 0$…liquefaction

Effective Stress

- Pile Driving

Soil is densified and subsequently sheared. ***In dilatant soils***, negative pore pressures cause a temporary increase in effective stress



$$\sigma' = \sigma_t - (-U)$$

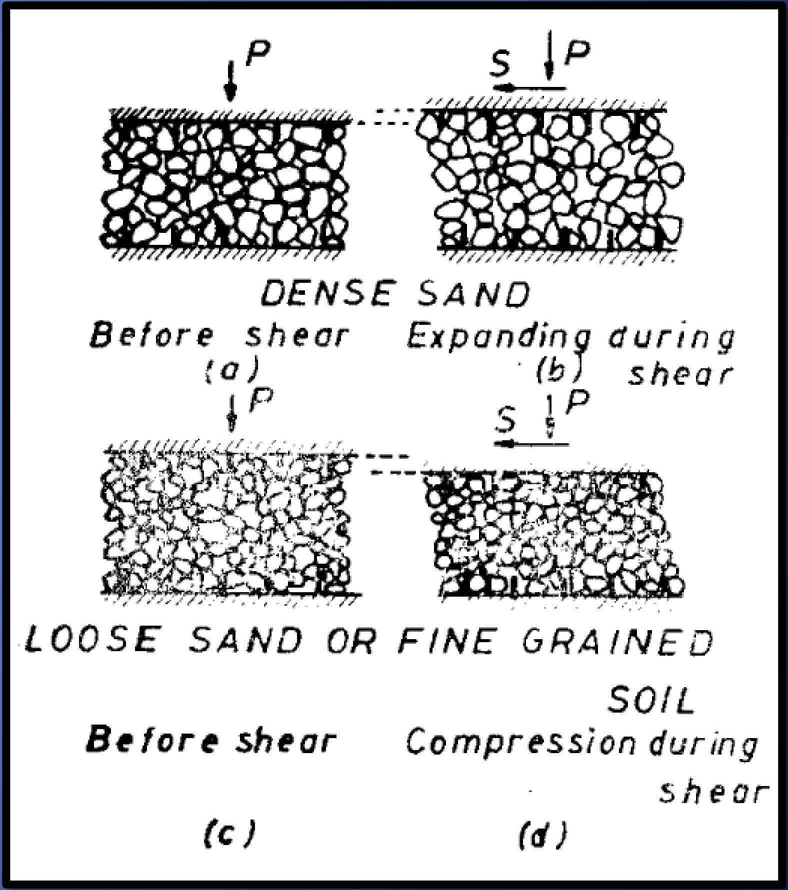
Looking at dilatant material in more detail….

Dilation

- Dilation is the observed tendency of dense granular material to expand in volume as it is sheared.
- In densely packed arrangements interlocking prevents the grains from moving around each other and they are forced to either shear or "roll" over each other. It is the rolling action that can generate suction into the void spaces created during displacement.

Dilation

- “It is a remarkable fact that a dense sand, when compressed in one direction actually increases in volume.” (Lambe and Whitman 1969)

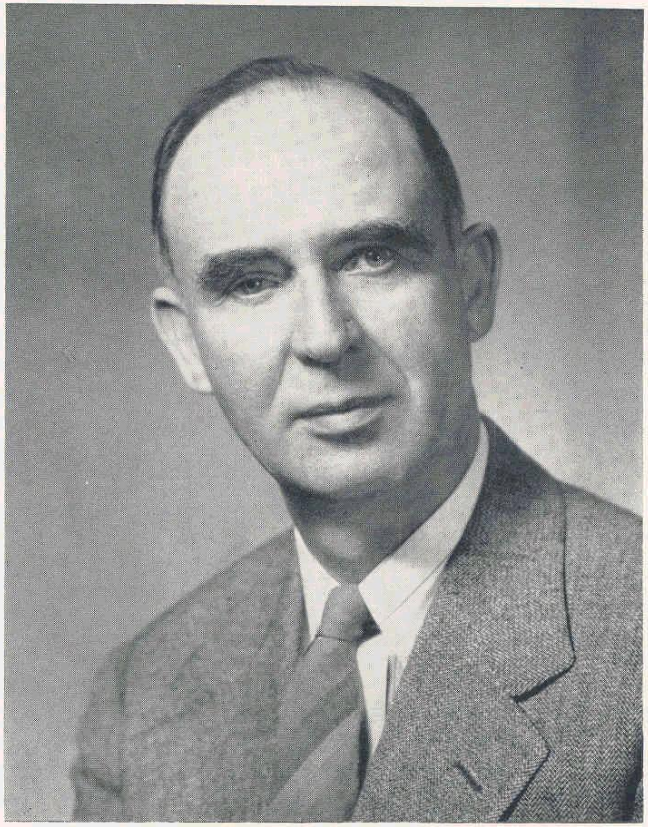


Dilation

- "The angle of internal friction, in spite of its name, does not depend solely on internal friction, since a portion of the shearing stress on a plane of failure is utilized in overcoming interlocking."

$$\tau/\sigma' = (\delta y/\delta x) + \mu$$

Fundamentals of Soil Mechanics, 1948



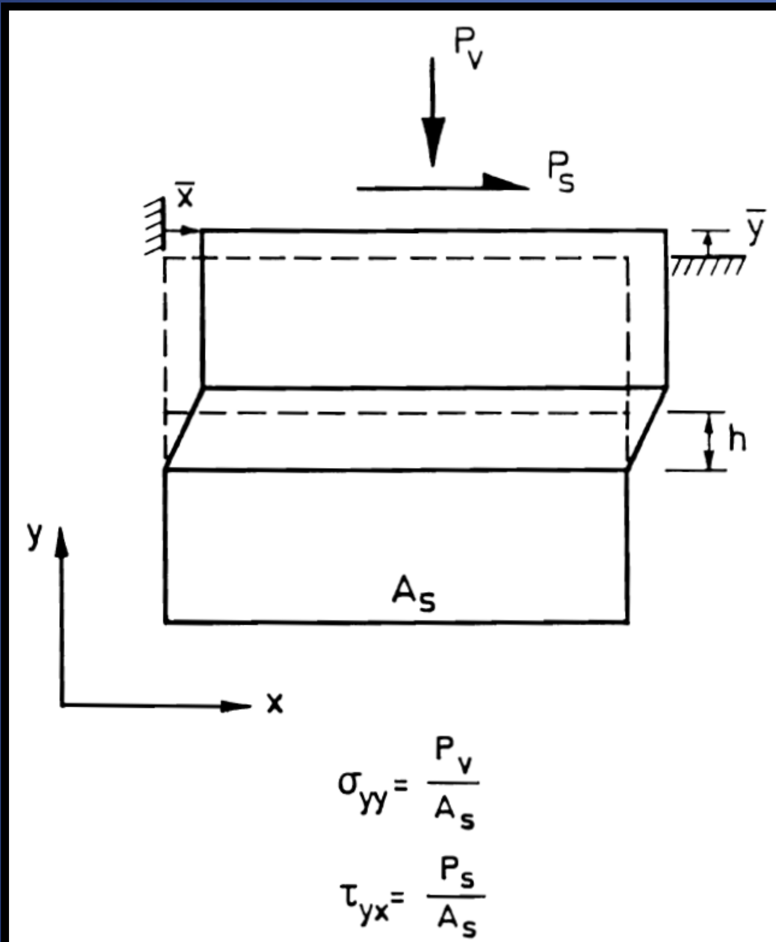
Donald W. Taylor

Dilation

Coulomb's description of shear strength does not explicitly state the influence of dilation

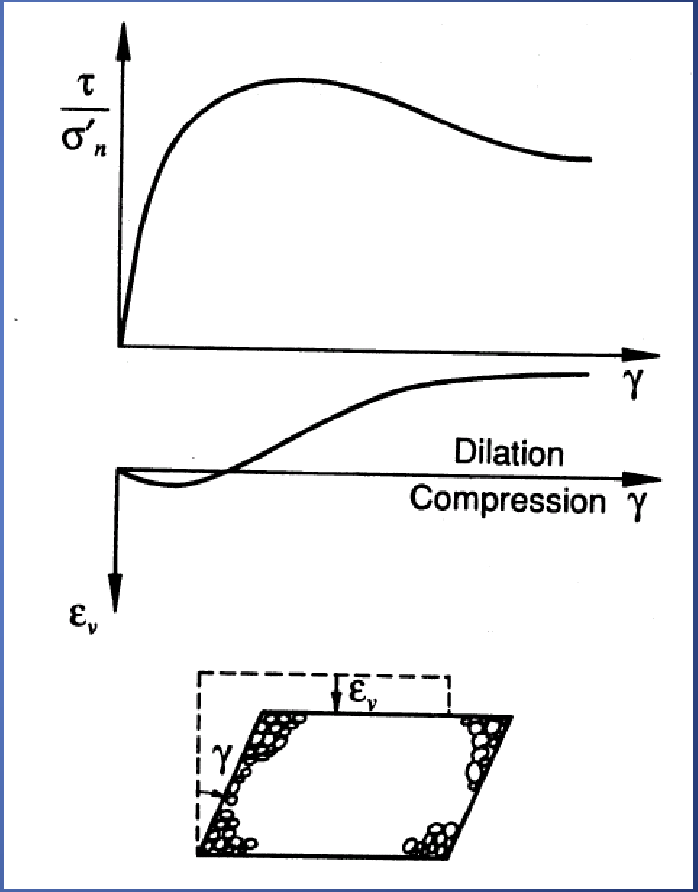
$\tau = c' + \sigma' \tan \phi'$

Laboratory Shear Resistance



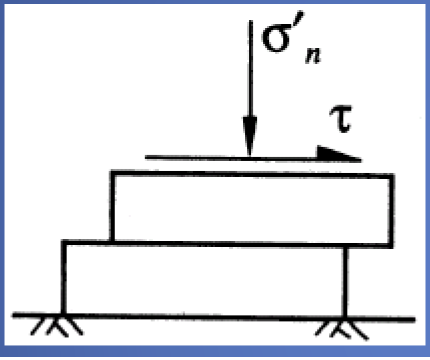
Dilation

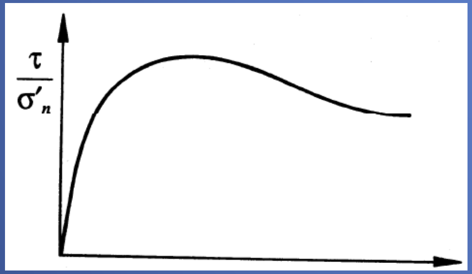
- Dense sands tend to dilate
- Initial compression followed by an expansion in volume
- Denser samples dilate faster
- When the test is performed under large pressure ϕ'_{max} approaches ϕ'_{crit}

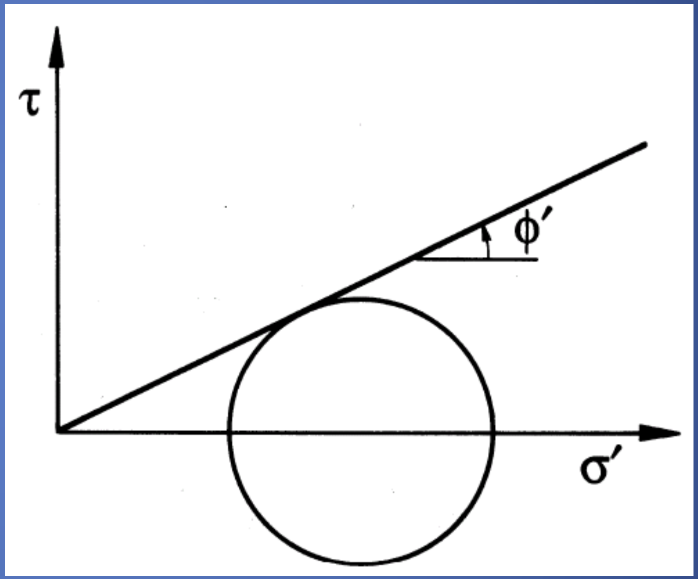


After Houlsby

Dilation



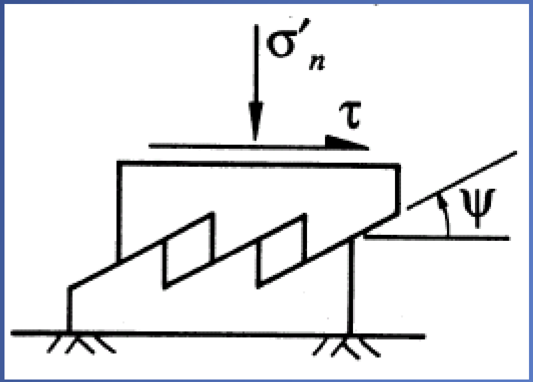




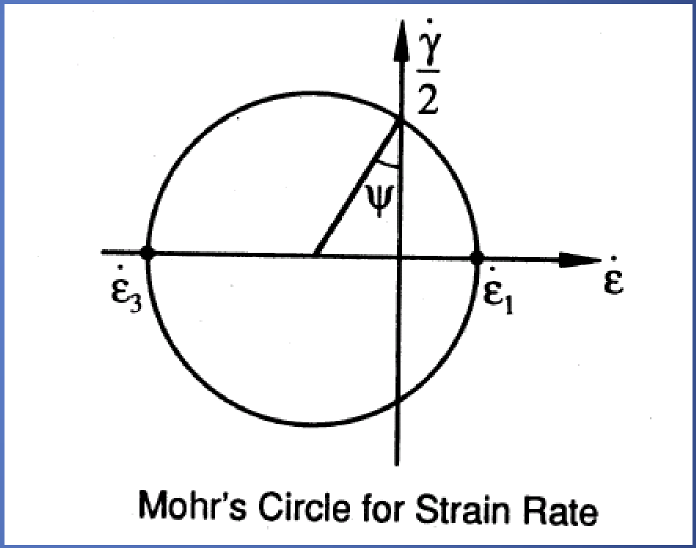
$\tau/\sigma = \tan \phi'$

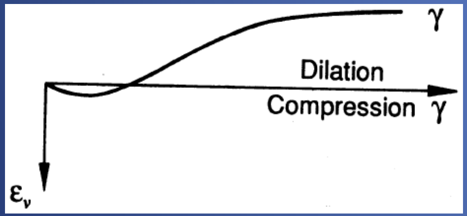
the ratio of shear stress to normal stress

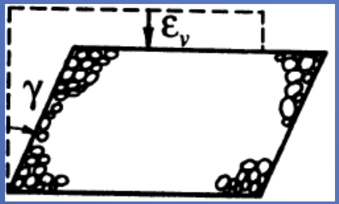
Dilation



Houlsby

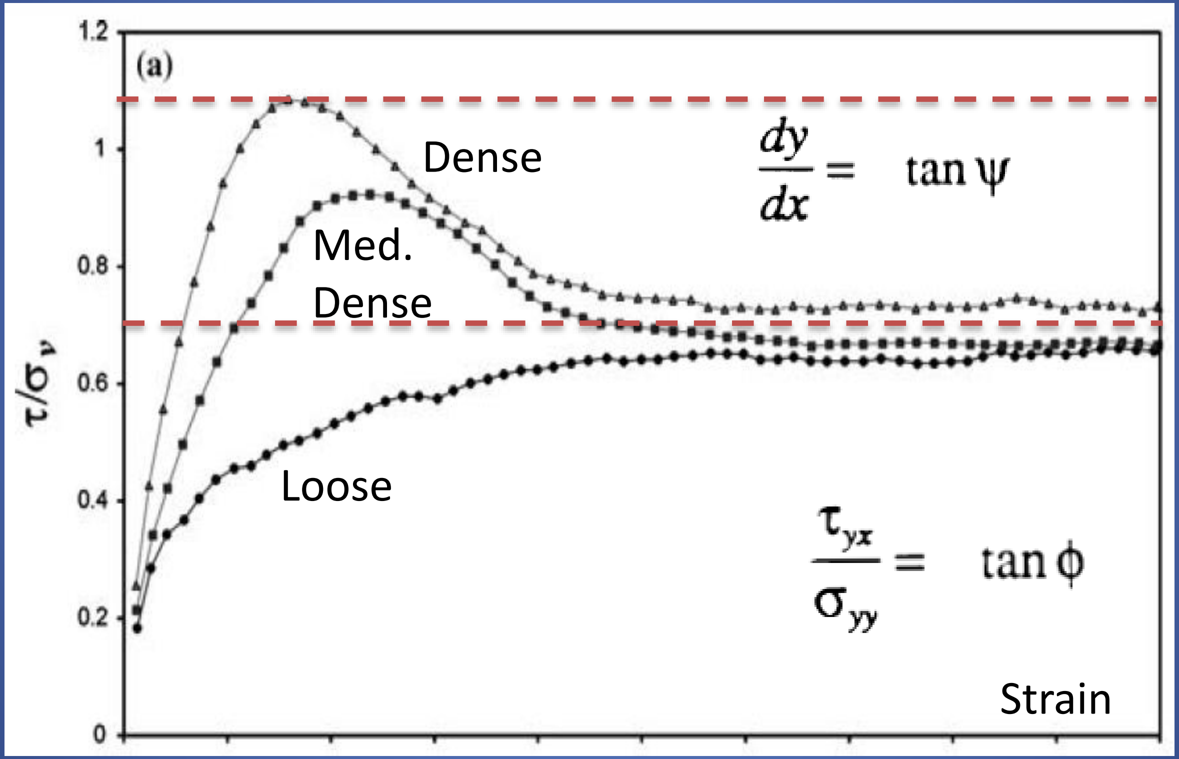






tan ψ is the ratio between a volumetric strain rate and a shear strain rate

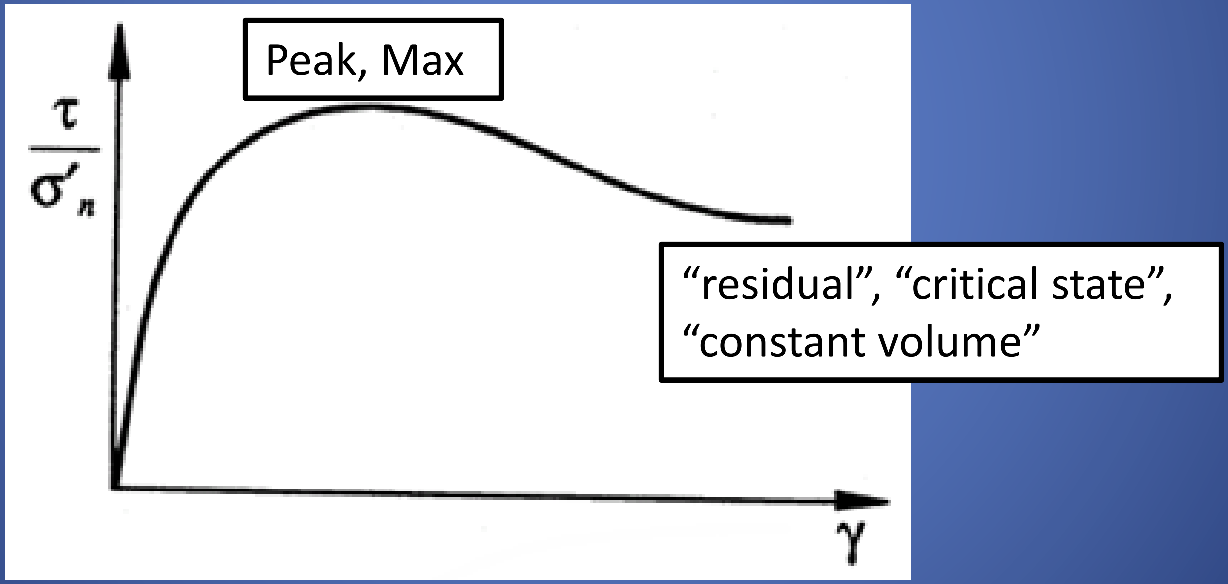
Dilation



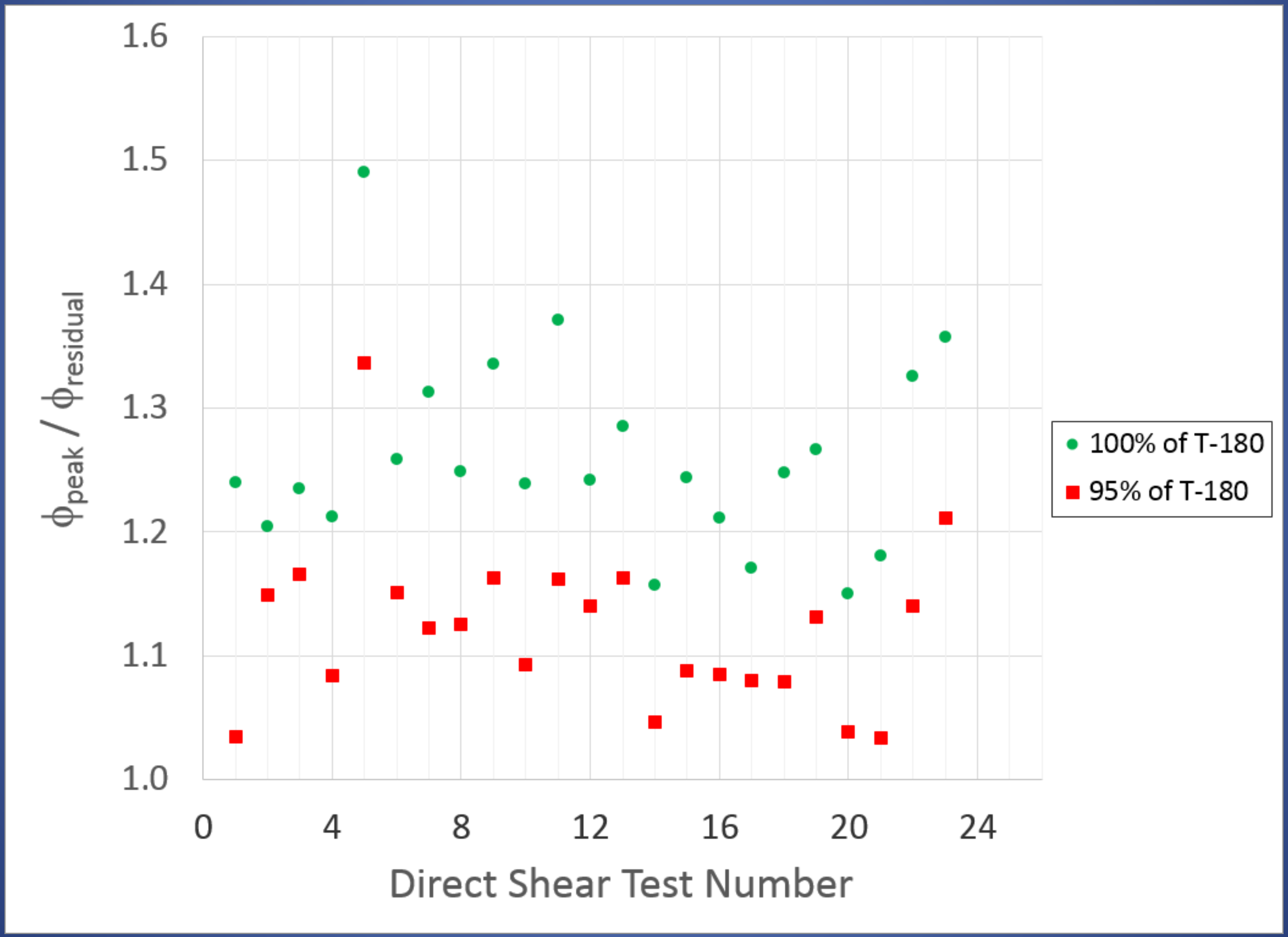
$\tan \phi'_p = \tan(\phi'_{cv} + \psi)$ (General form)

Dilation

– Terminology



Dilation

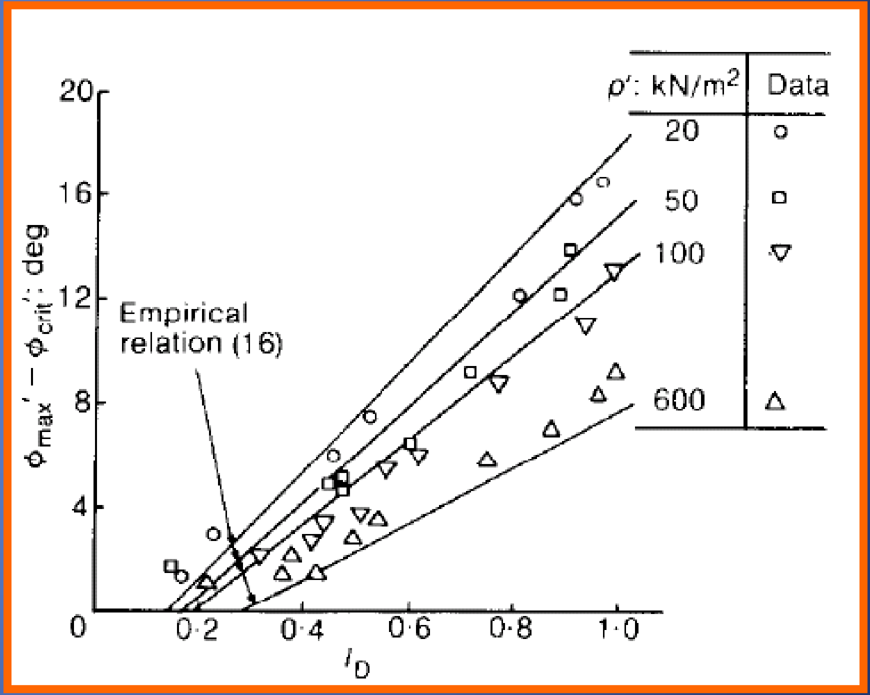


1.6
1.5
1.4
1.3
1.2
1.1
1.0
$\phi_{peak} / \phi_{residual}$
0
4
8
12
16
20
24
Direct Shear Test Number
100% of T-180
95% of T-180

Dilation

- The angle of dilation tends to increase with density, that process is highly dependent on soil mineralogy

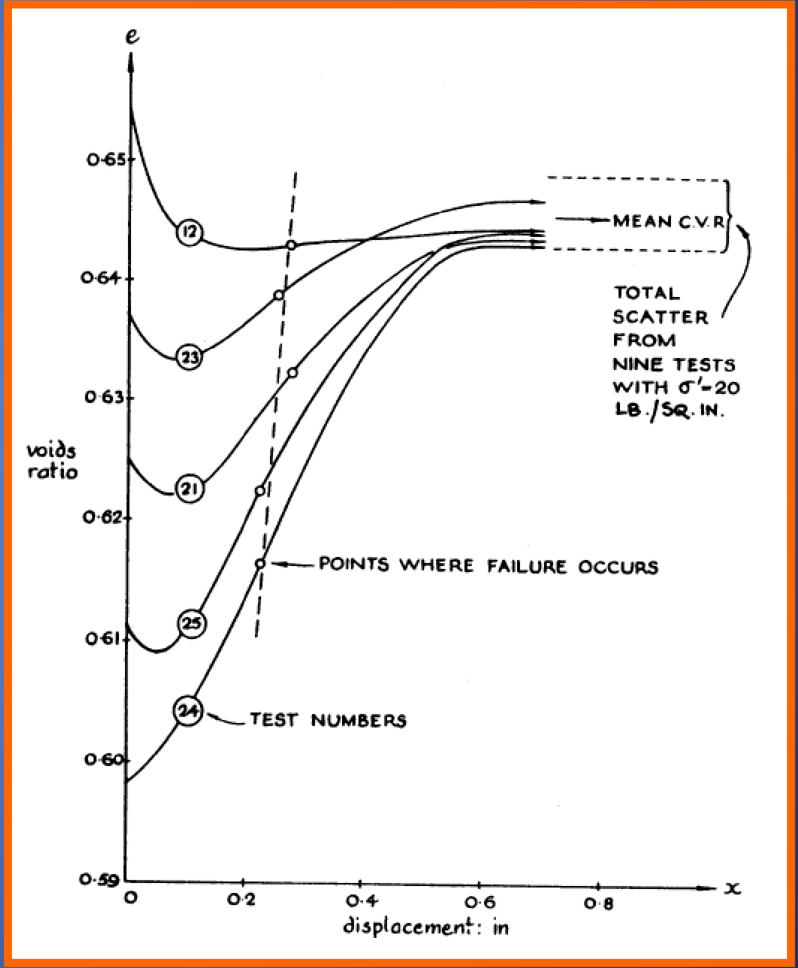
Dilatancy-density relationship



Berlin Sand (DeBeer, 1965)

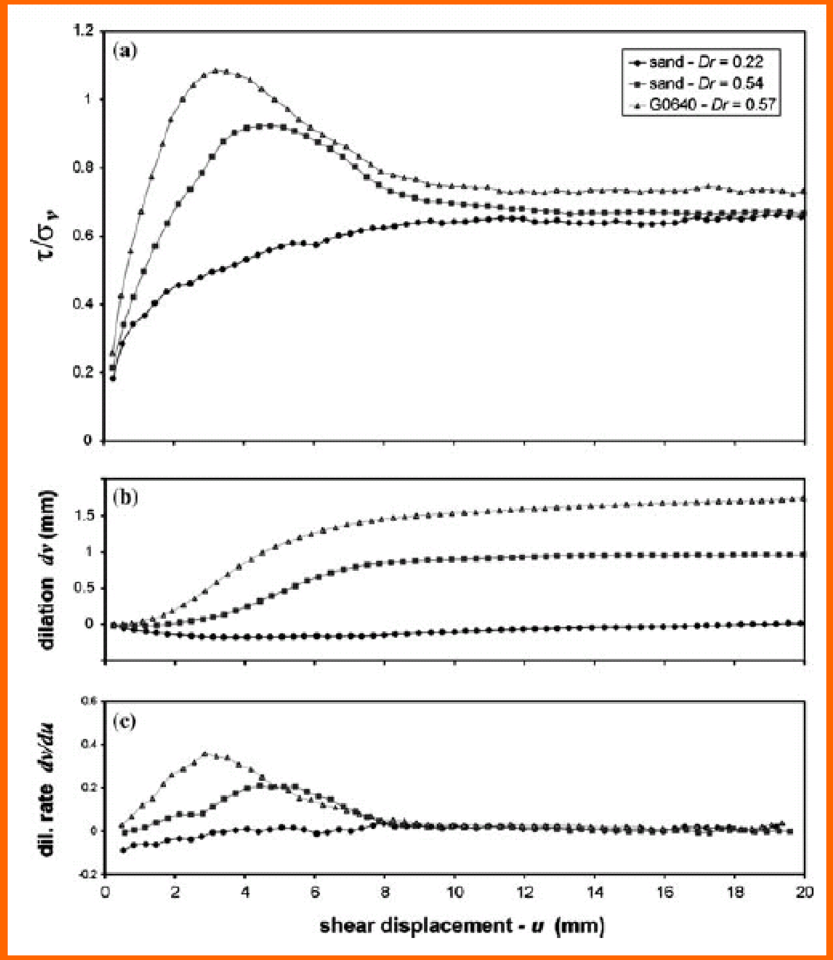
Dilation

– Samples dilate until they reach a constant volume void ratio regardless of their initial density



Dilation

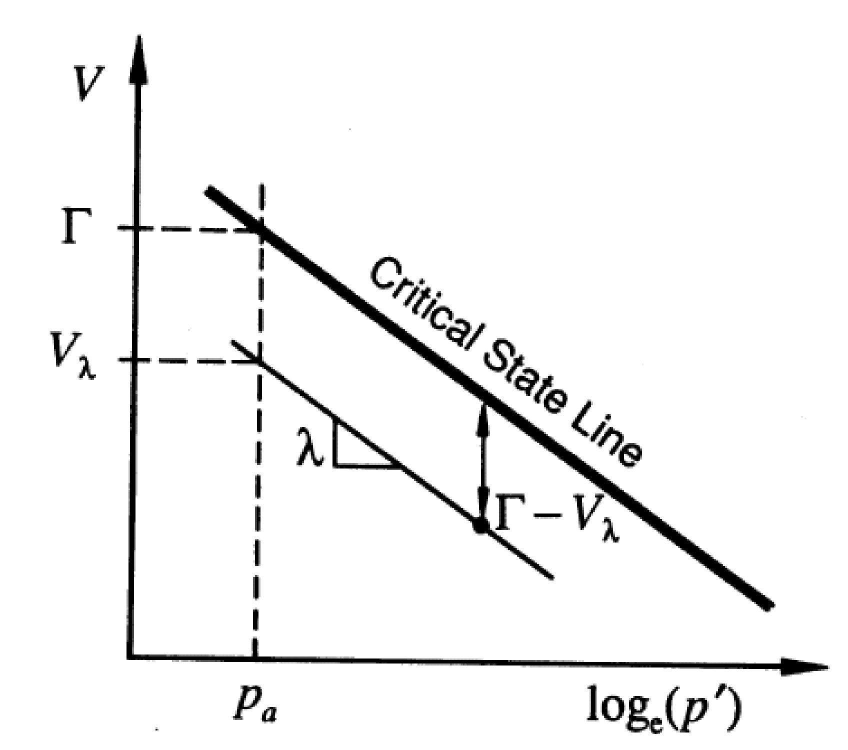
- The largest rate of dilation tan ψ coincides with ϕ'_{peak}.
- dilation rate approaches zero as ϕ' approaches ϕ'_{cv}



Simoni & Houlsby

Dilation

- On the Critical State Line (CSL) the rate of dilation is zero
- Dilation will only occur a certain “distance” away from the CSL in terms of stress/strain



Wroth and Basset, 1965

Dilation

- For direct shear

$$\frac{\sigma_1'}{\sigma_3'} = \left(\frac{\sigma_1'}{\sigma_3'}\right)_{crit}\left(1 - \frac{d\varepsilon_v}{d\varepsilon_1}\right)$$

Rowe

$$\phi' = \phi'_{crit} + 0{\cdot}8\psi$$

Bolton

Dilation

- Stress-dilatancy relationship → *"flow rule"*
- Using Taylor's energy correction equation;

$$\frac{\tau_{yx}}{\sigma_{yy}} + \frac{d\varepsilon_{yy}}{d\gamma_{yx}} = m$$

$$\frac{\tau_{yx}}{\sigma_{yy}} - \tan\psi = \sin\phi_{cv}$$

$$\tan\phi_{ds} - \tan\psi = \sin\phi_{cv}$$

Stroud, 1971

Dilation

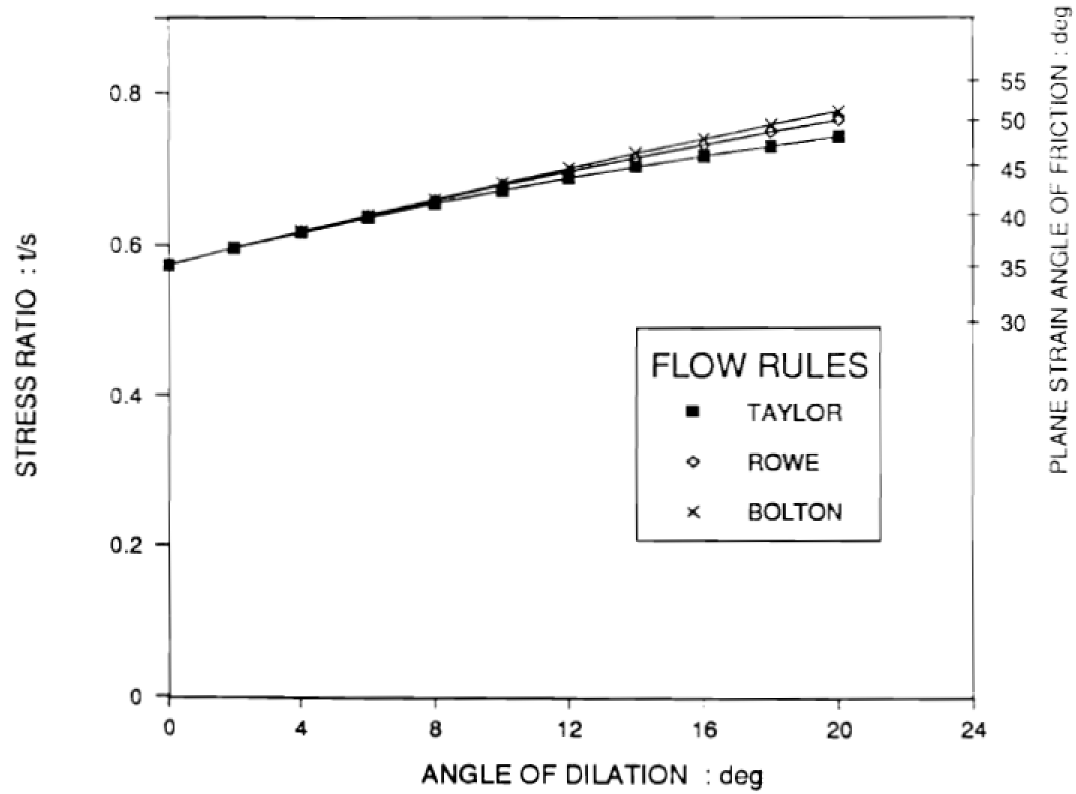
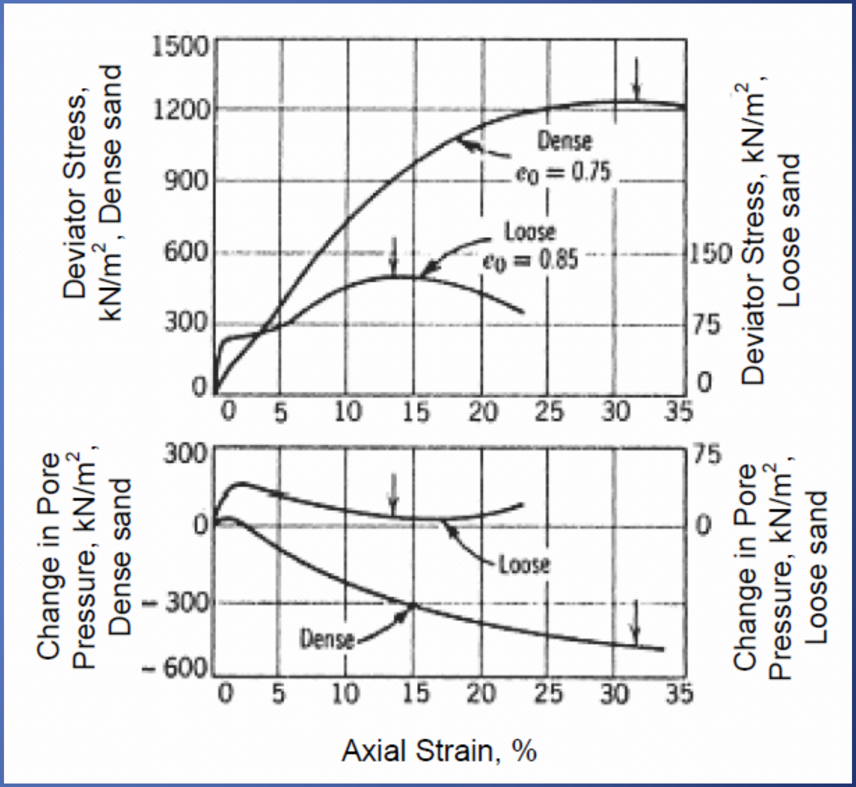


Fig. 6. Comparison of flow rules in terms of the principal stress ratio and the angle of dilation in the soil. Drawn for dense sand $\phi_{cv} = 35°$.

Jewell, 1988

THE PROBLEM

– Dilatant soil can generate negative pore water pressure, having an overall effect of a **temporary increase in effective stress**



After Leonards

THE PROBLEM

- At rest (at t_0)

$$\sigma'_{v\ t=0} = \sigma_T - U_o$$

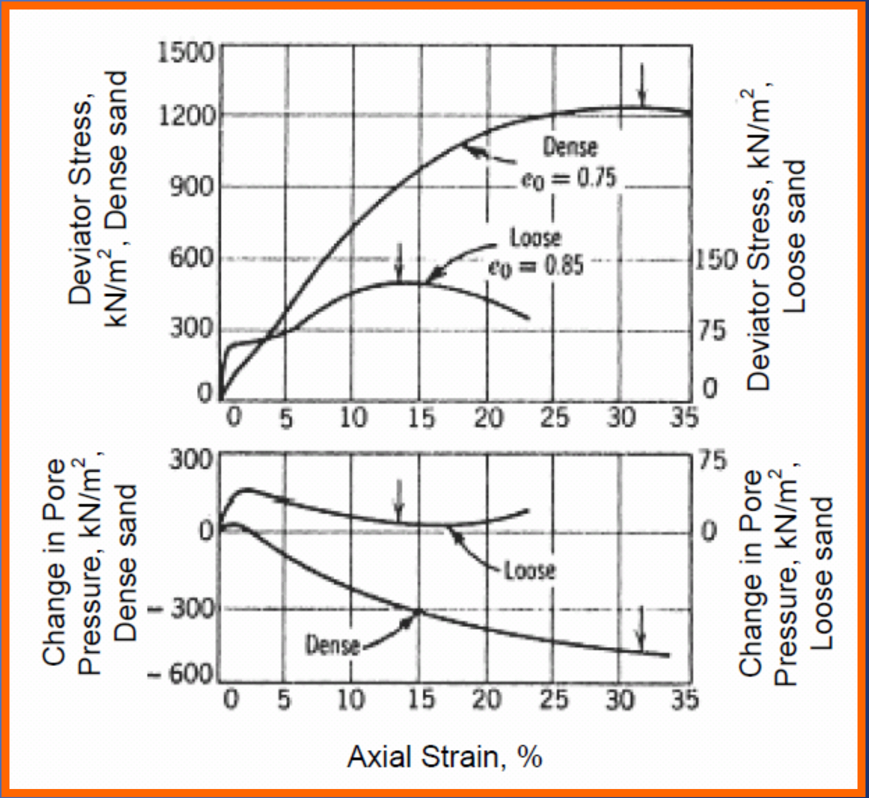
- During dilation (at t_1)

$$\sigma'_{v\ t=1} = \sigma_T - (-U_1)$$

$$\sigma'_{v\ t=1} > \sigma'_{v\ t=0}$$

- After some time "t"
  U_1 will revert to U_o

$$\sigma'_{v\ t=2} < \sigma'_{v\ t=1}$$



Effect of Density and Confining Stress

- Bolton
  - For Plane Strain

$$\phi'_{max} - \phi'_{crit} = 0.8\psi_{max} = 5\ I_R$$

  - For Triaxial strain

$$\phi'_{max} - \phi'_{crit} = 3\ I_R$$

[16]

  - Relative dilatancy index I_R

$$I_R = I_D\ (10 - \log \sigma') - 1$$

Effect of Density and Confining Stress

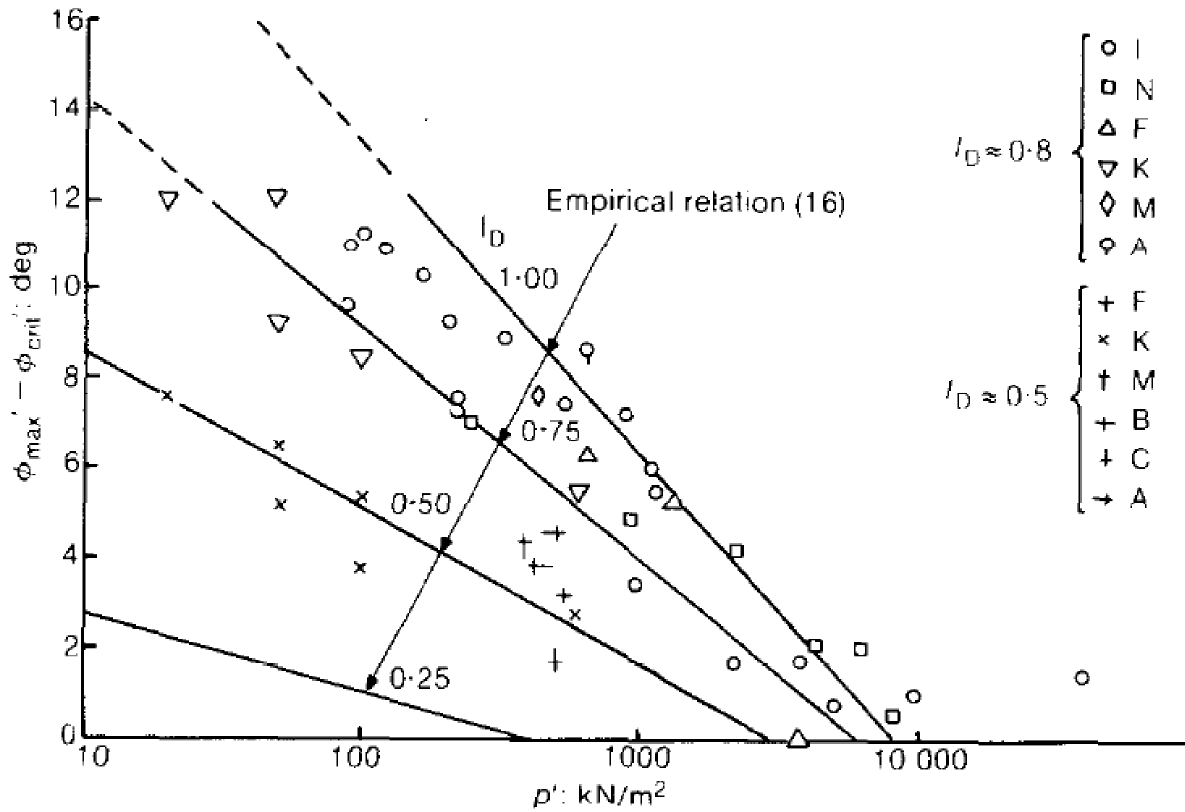
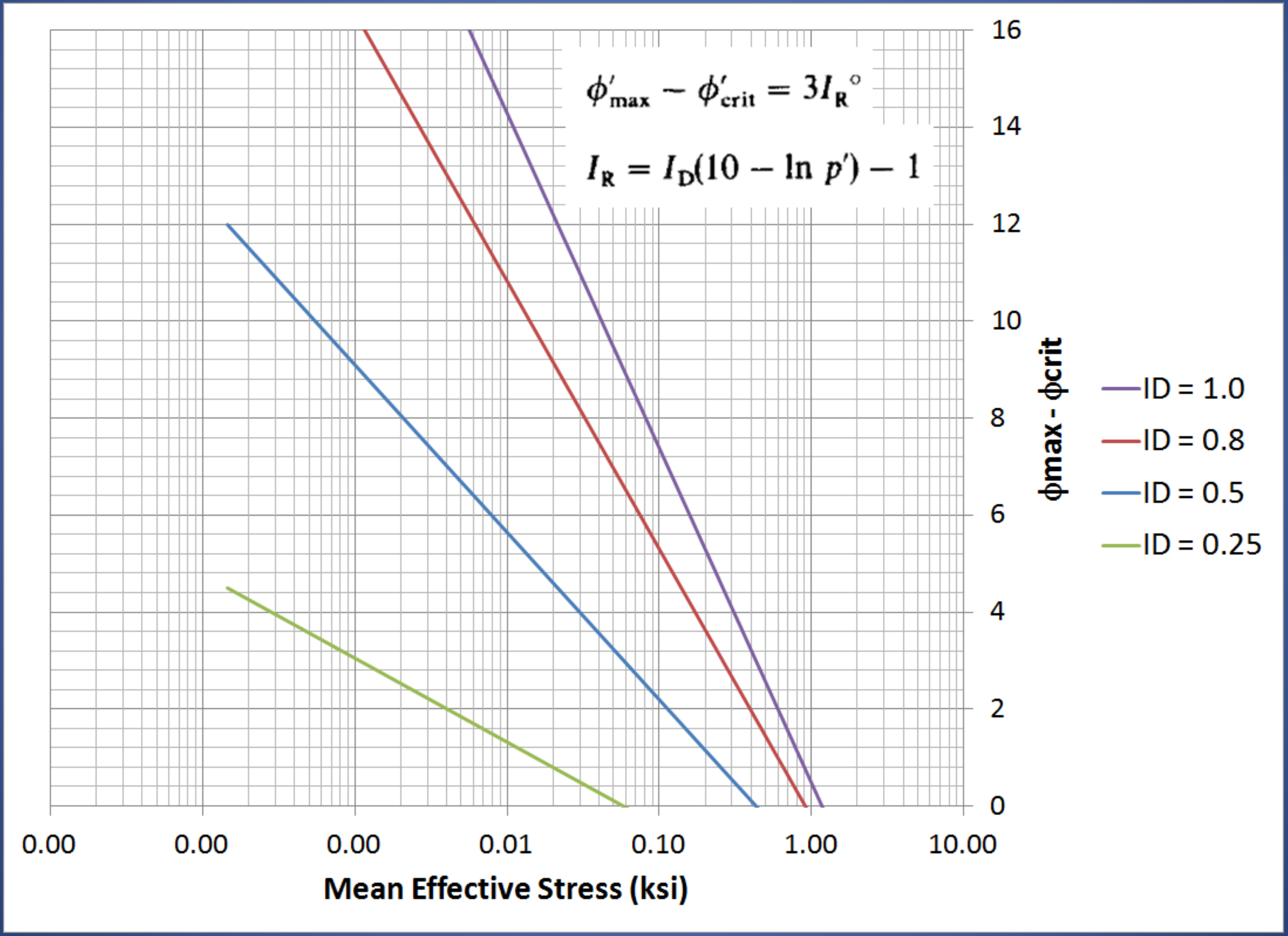


Fig. 10. Triaxial test data for sands in Table 1 failing at various mean effective stresses

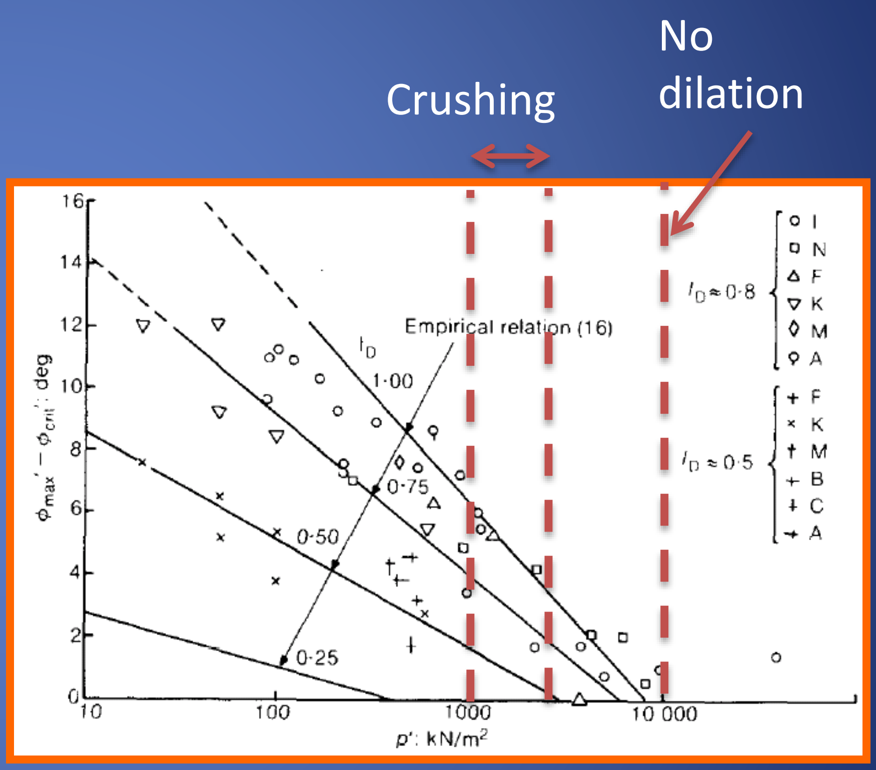
Effect of Density and Confining Stress



ID = Relative density

Effect of Density and Confining Stress

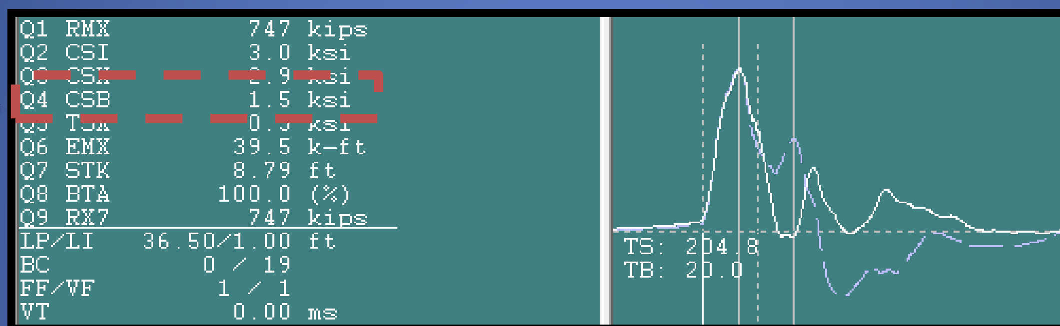
- Sands have been found to begin crushing at pressures ranging from 0.15 to 0.58 ksi (Bolton).
- The Triaxial data collected indicates zero dilation for the materials tested at approximately 1.45 ksi (10,000 kN/m^2)

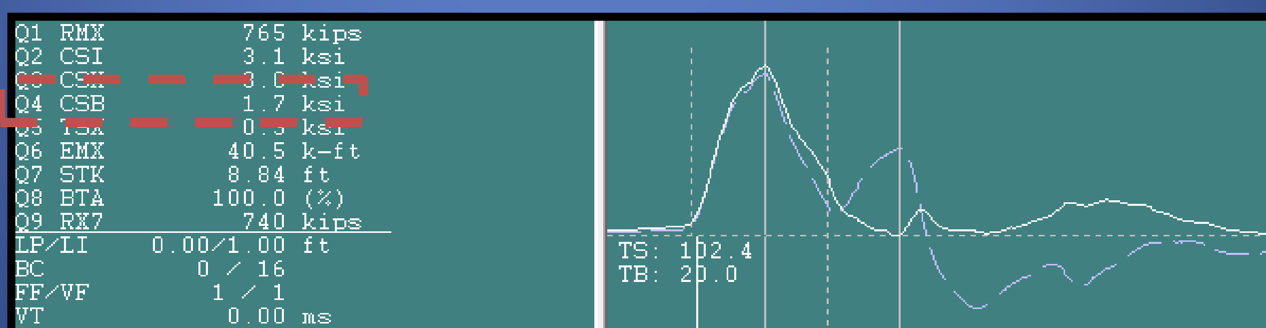


Triaxial data of Lee & Seed

Effect of Density and Confining Stress

- Tip stresses from two different piles in sands

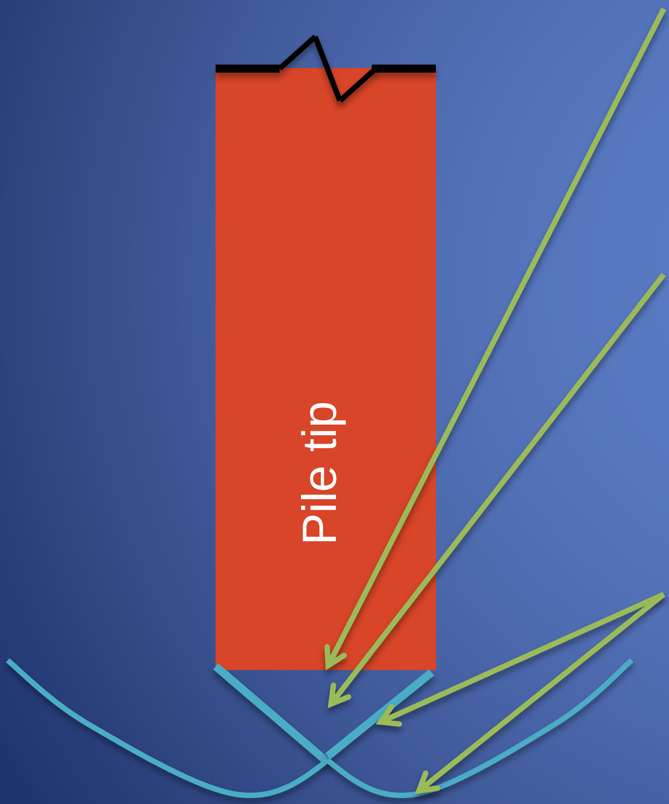




Tip stresses from PDA (CSB) > 1.45 ksi >> 0.58 ksi

No dilation and possible soil crushing at the soil-pile tip interface

Dilation at Pile Tip Shear Surfaces

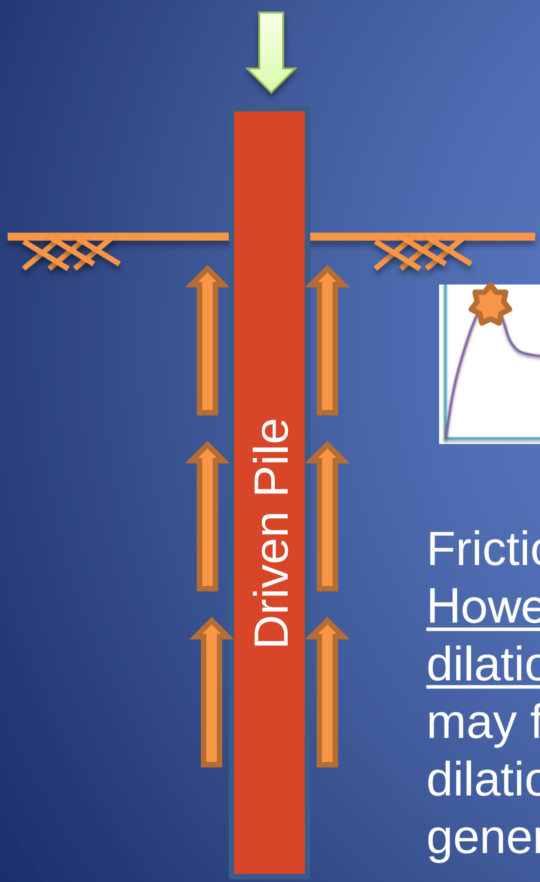


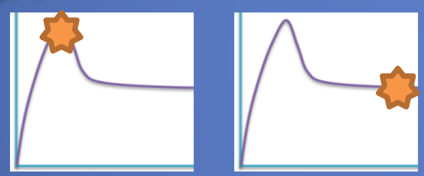
Granular soil is crushed at the soil-pile interface (depends on mineralogy) $\phi = \phi_{crit}$ and $\psi = 0$

Possible development of positive excess pore pressure below the tip ($V_{pile} > k_{soil}$), with a <u>temporary decrease</u> in effective stress (temporary reduction in end bearing)

Shear surfaces dilate as deformation occurs developing $\psi > 0$ and –U, generating a <u>temporary increase</u> in effective stress (temporary increase in end bearing)

Dilation Along the Sides

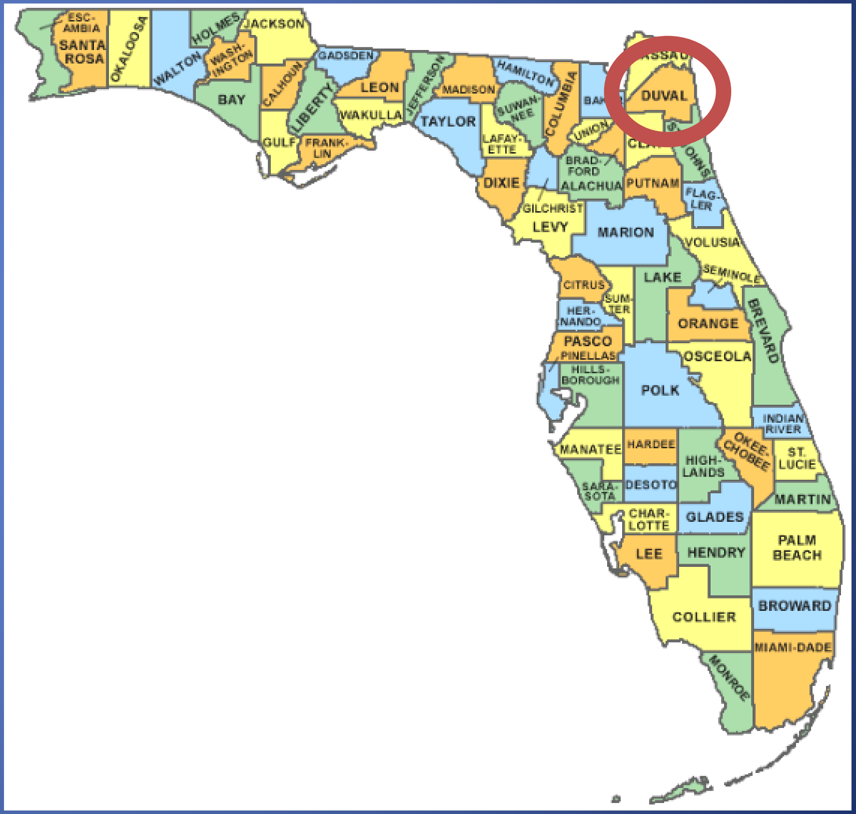




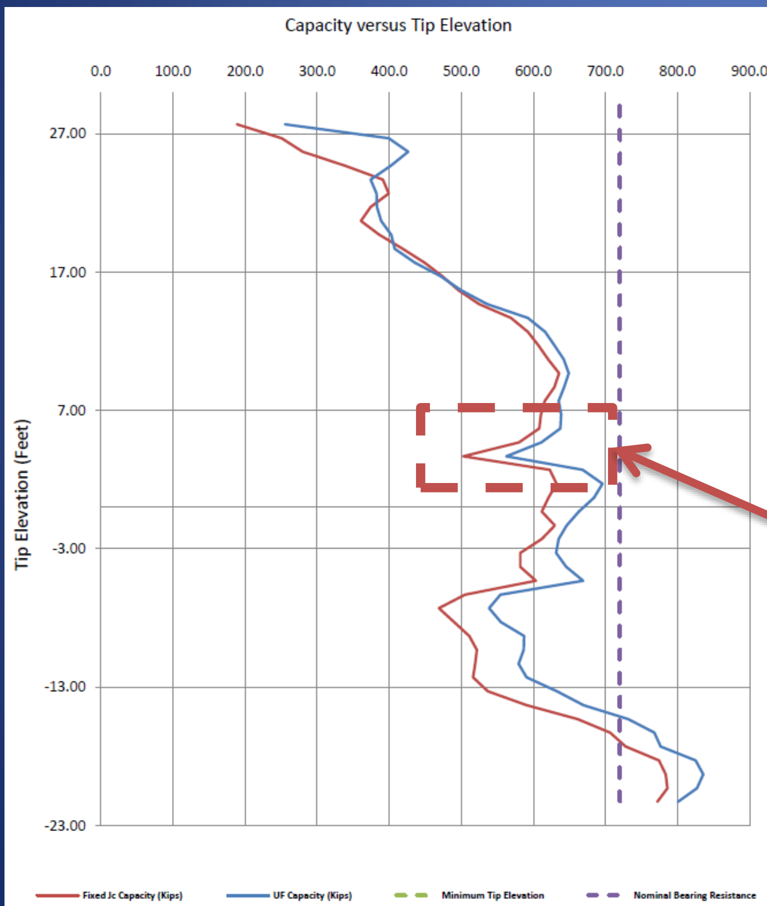
Frictional response goes from peak to residual, $\psi > 0$, However, every “segment” along the pile experiences dilation only once during the drive, unlike the pile tip which may feel the effect after every blow. The influence of dilation along the side of the pile on the overall capacity is generally minor due to this.

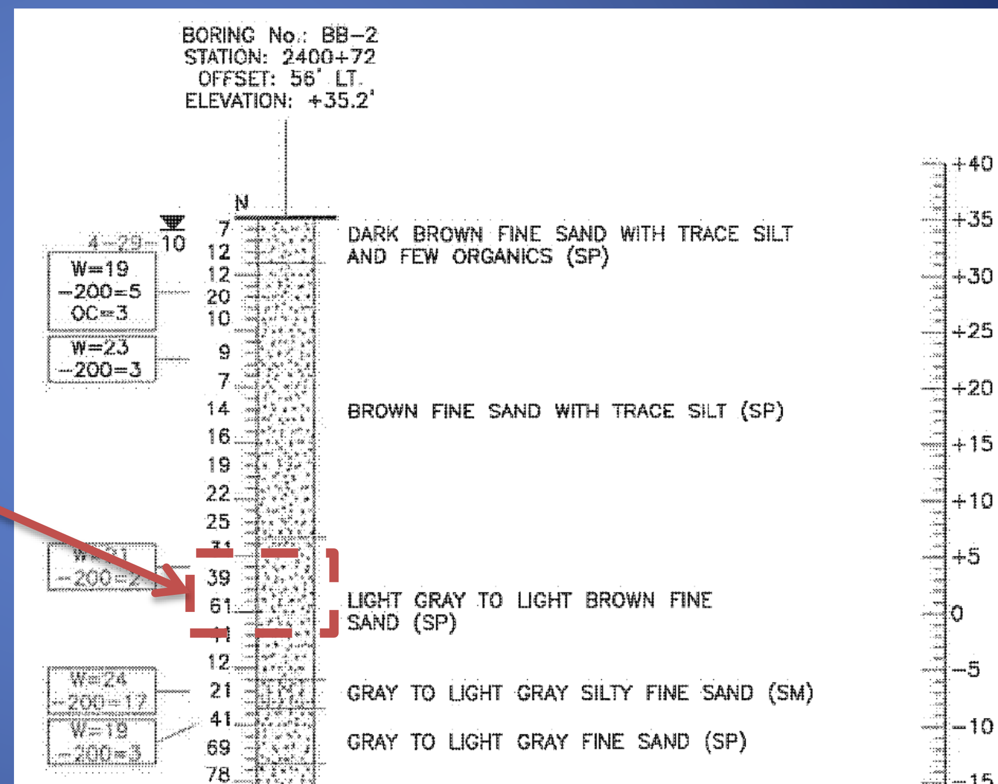
Case History
SR 9B Phase I – Duval County

- Nominal Bearing Resistance “NA” (Research pile)
- 24” Pre-stressed concrete pile
- Delmag D46-32
  - Open end diesel



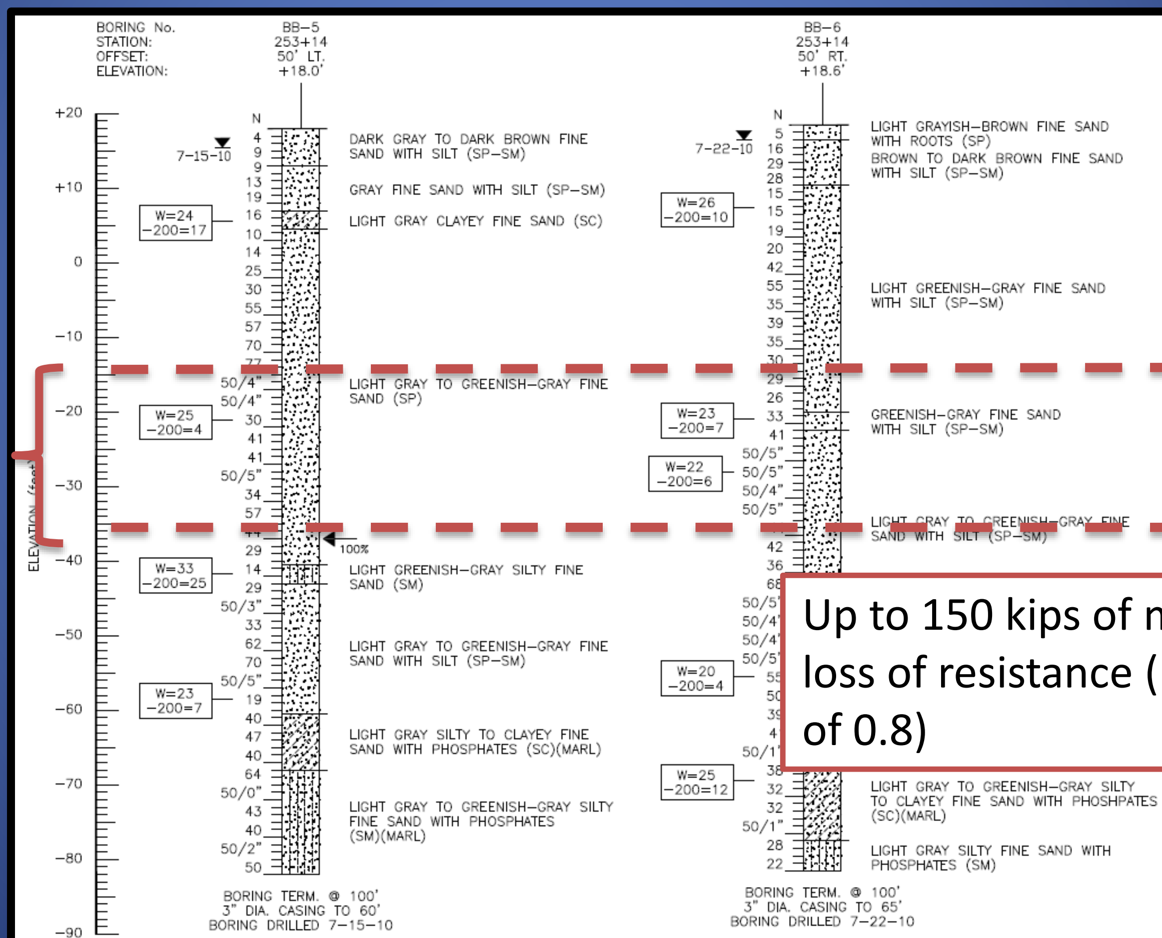
Case History
SR 9B Phase I – Duval County





EDC data

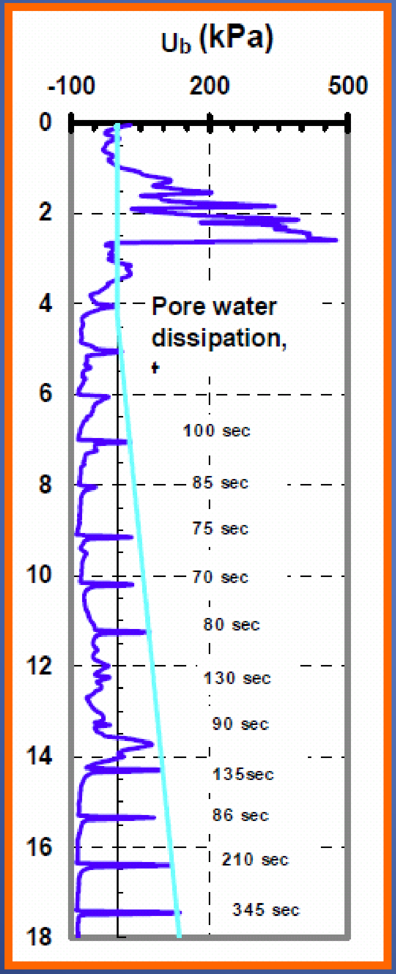
Case History
SR 9B Phase I – Duval County



Zone of measured relaxation

Up to 150 kips of measured loss of resistance (BOR/EOID of 0.8)

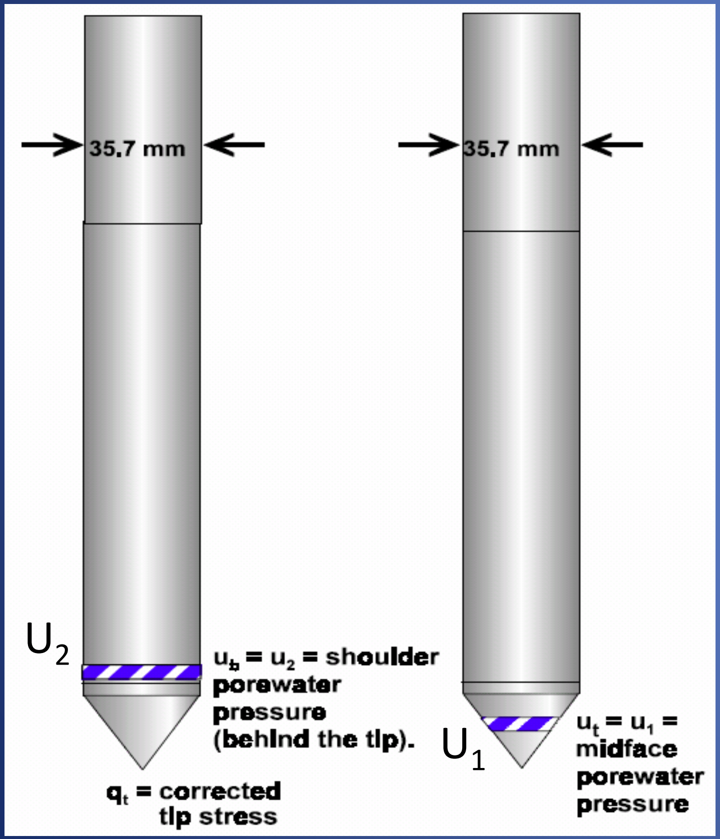
Case History
SR 9B Phase I – Duval County



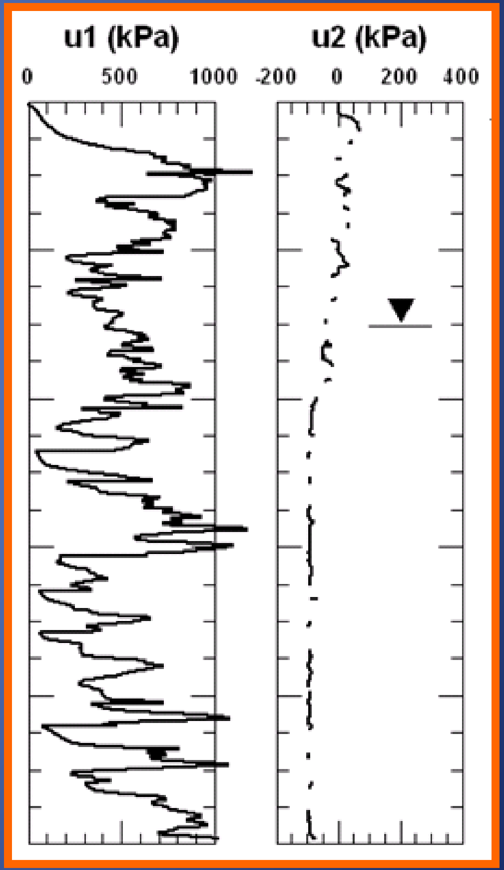
- Seismic piezocone soundings in piedmont soils (ML and SM) indicate negative pore pressure generated from dilation is not a permanent condition, and "U" will return to a hydrostatic stress level after a period of time

After Mayne

Case History
SR 9B Phase I – Duval County

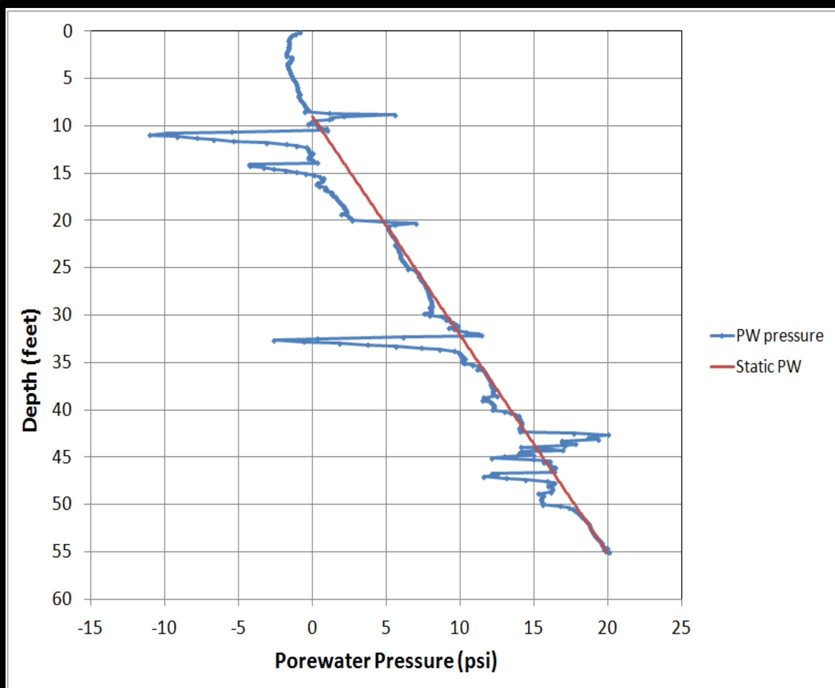


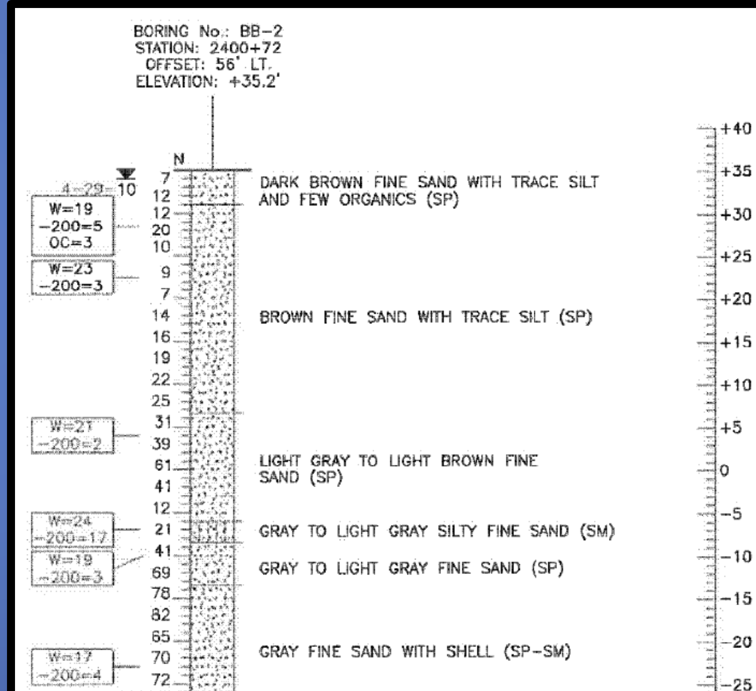
U2 position is required



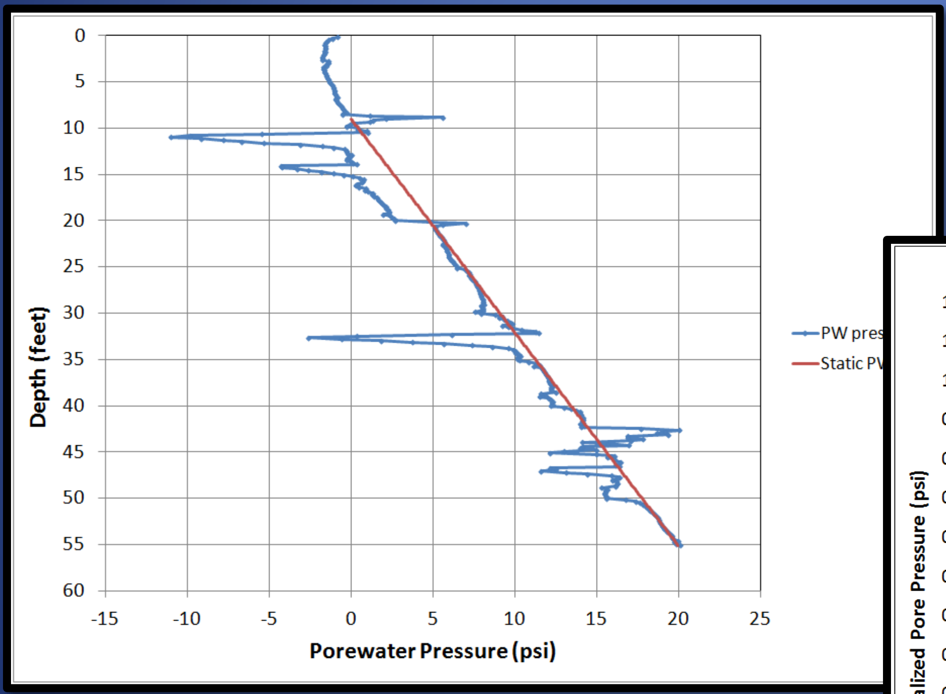
Case History
SR 9B Phase I – Duval County

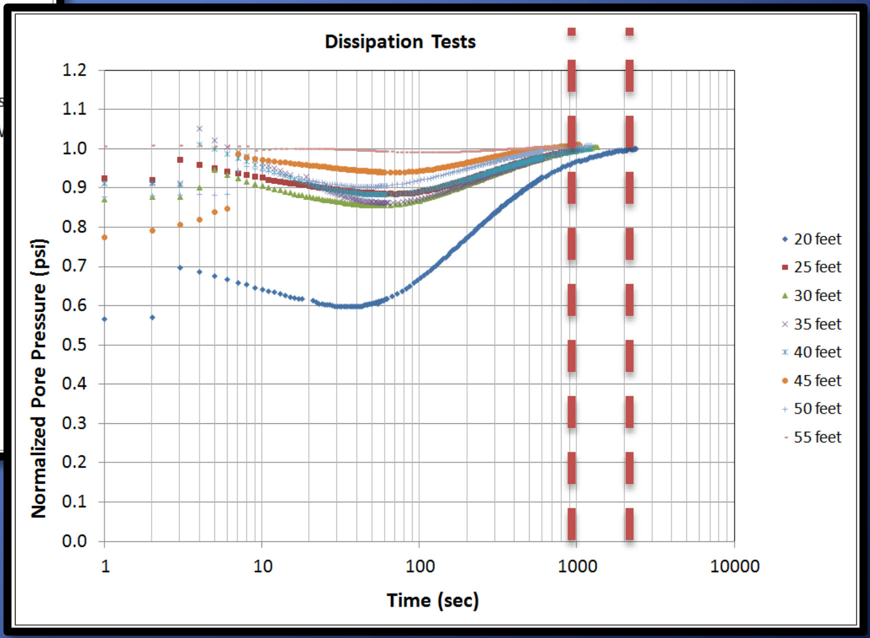
Safety Hammer





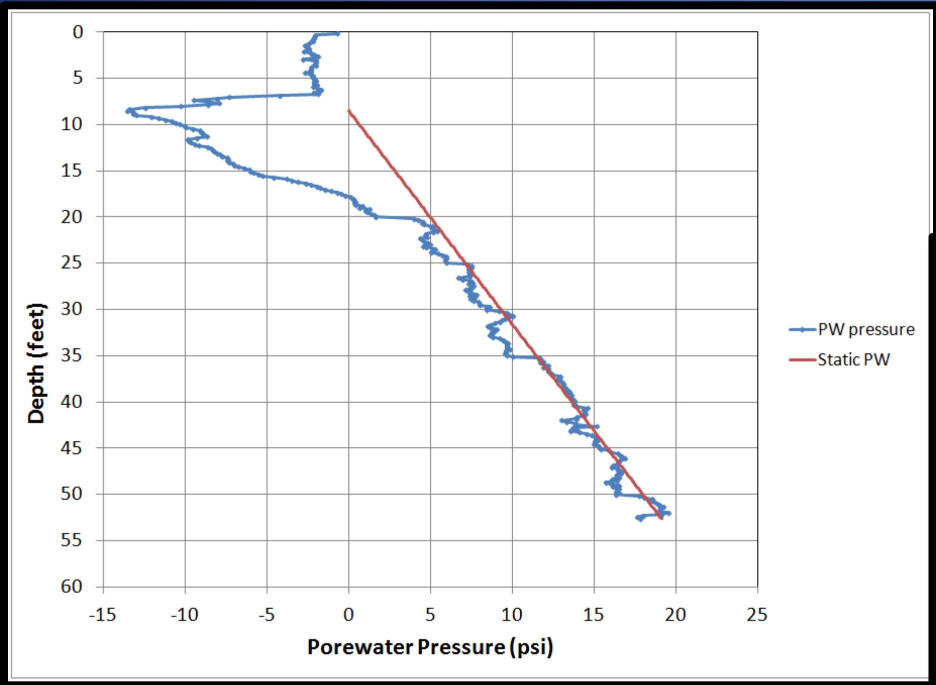
Case History
SR 9B Phase I – Duval County

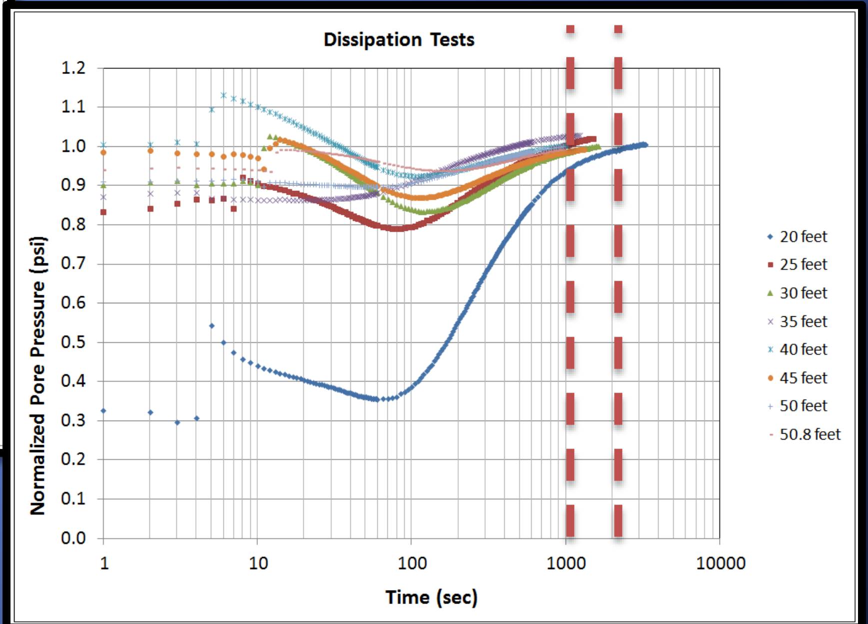




15 - 30 minutes

Case History
SR 9B Phase I – Duval County

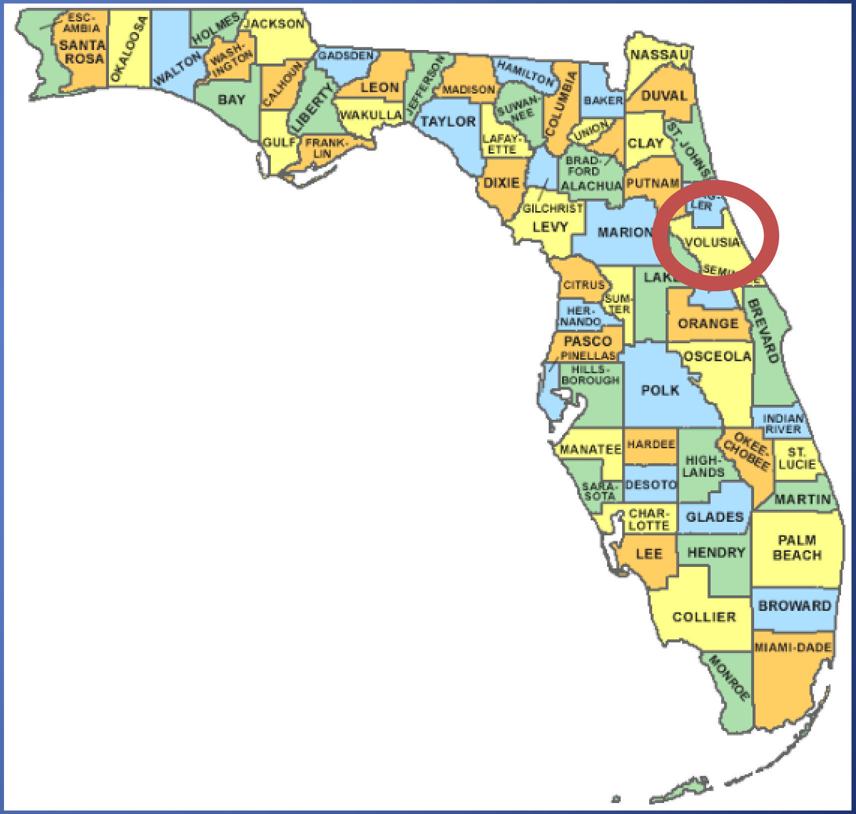




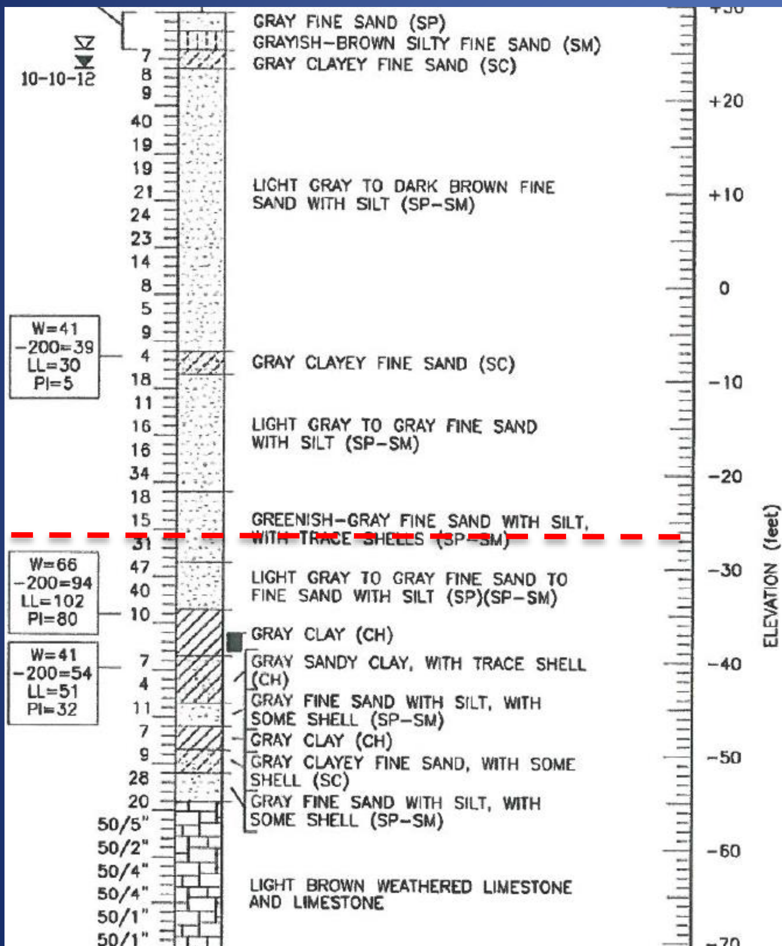
15 - 30 minutes

Case History
SR 400 Widening – Volusia County

- Nominal Bearing Resistance ranged from 520 to 734 Kips
- 24” Pre-stressed concrete piles
- APE D-46-32
  - Open end diesel



Case History
SR 400 Widening – Volusia County

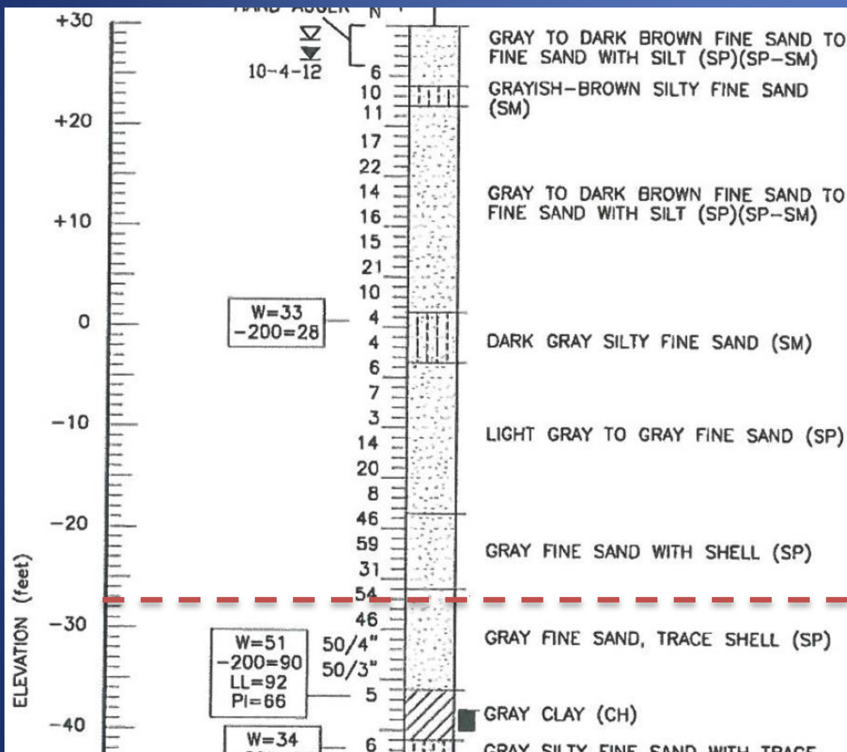


RB-1 (≈ -26.5)

Bent 6-1, Pile 3, EOID					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
9/30/13	948	426	522	0.26	30.7
BOR					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
10/9/13	859	330	529	0.44	34.7
	BOR/EOID				
Time (days)	Total	Skin	E.B.	J_C	
9	0.91	0.78	1.01	1.7	

CAPWAP Results
(EMX from PDA)

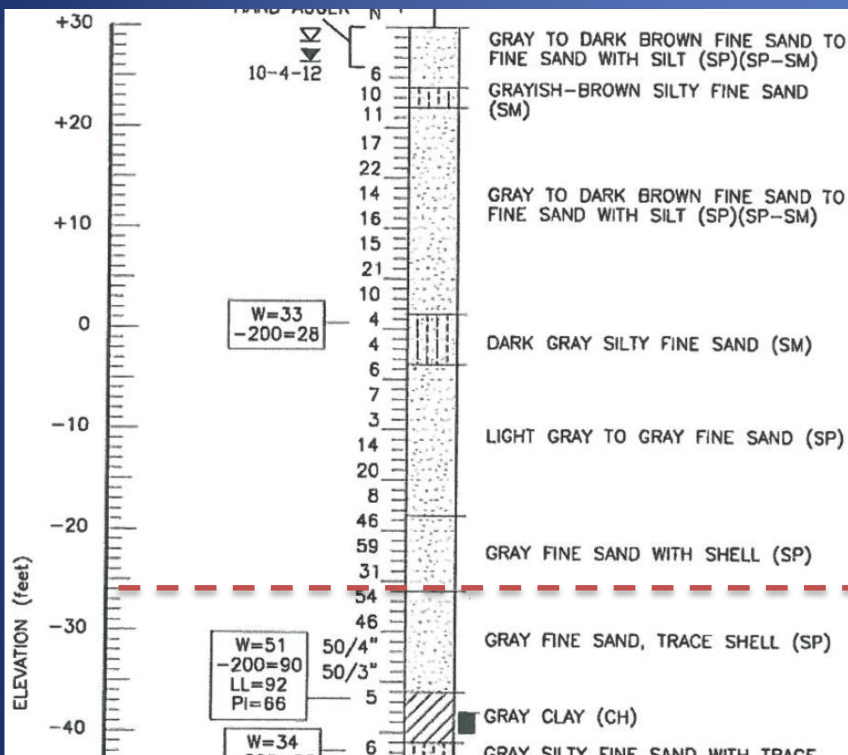
Case History
SR 400 Widening – Volusia County



RB-2 (≈ -27.3)

Bent 6-2, Pile 8, EOID					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
10/05/13	1154	406	749	0.28	37.7
BOR					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
10/12/13	910	519	390	0.53	34.5
	BOR/EOID				
Time (days)	Total	Skin	E.B.	J_C	
7	0.79	1.28	0.52	1.89	

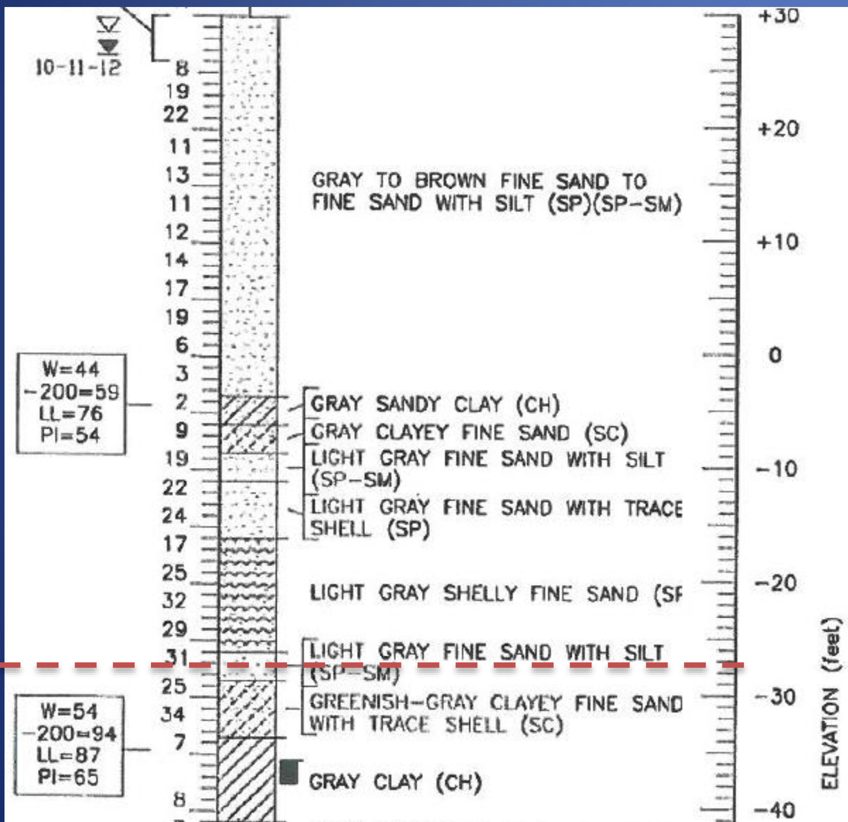
Case History
SR 400 Widening – Volusia County



RB-2 (≈ -26)

Bent 6-2, Pile 12, EOID					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
10/9/13	1100	471	630	0.4	36.5
BOR					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
10/15/13	831	483	348	0.59	36.6
	BOR/EOID				
Time (days)	Total	Skin	E.B.	J_C	
3	0.76	1.03	0.55	1.48	

Case History
SR 400 Widening – Volusia County



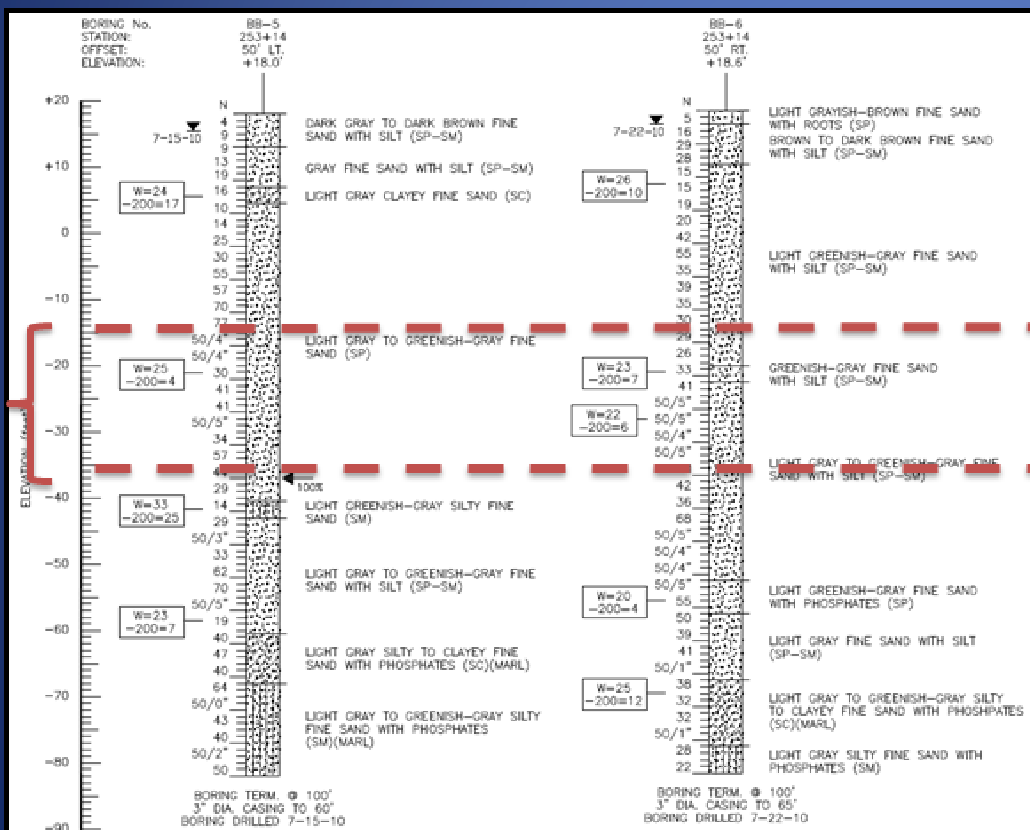
RB-3 (≈ -27.3)

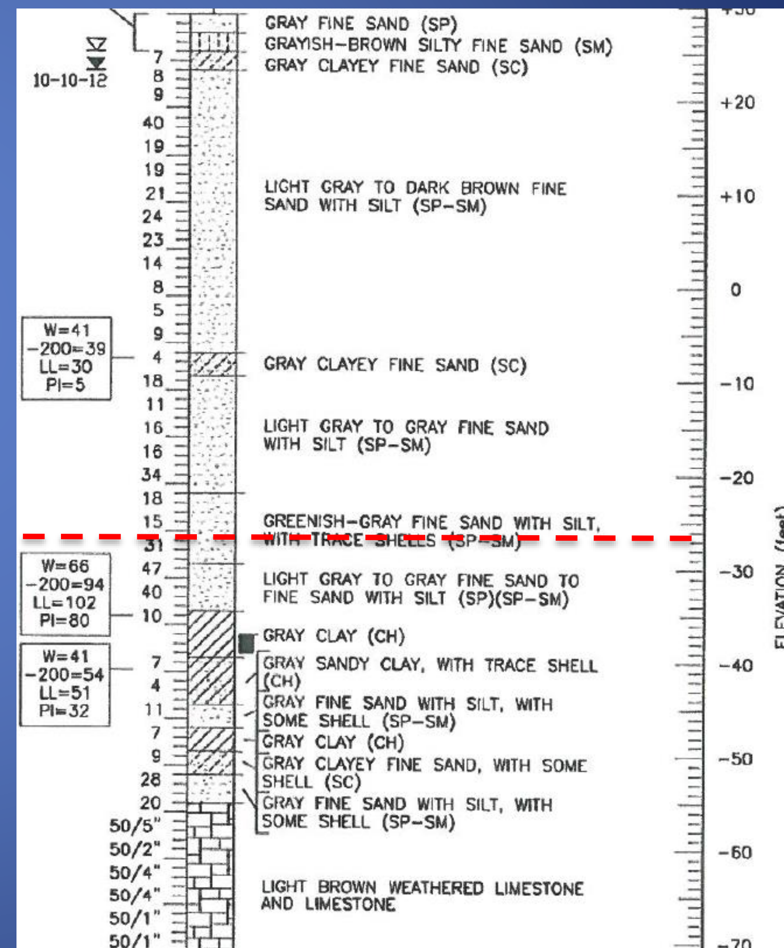
Bent 6-3, Pile 7, EOID					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
9/18/13	1079	198	881	0.33	41.4
BOR					
Date	Total (kips)	Skin (kips)	E.B. (kips)	J_C	EMX (ft-k)
10/23/13	945	575	370	0.5	37.2
	BOR/EOID				
Time (days)	Total	Skin	E.B.	J_C	
35	0.88	2.9	0.42	1.52	

Relaxation - Design

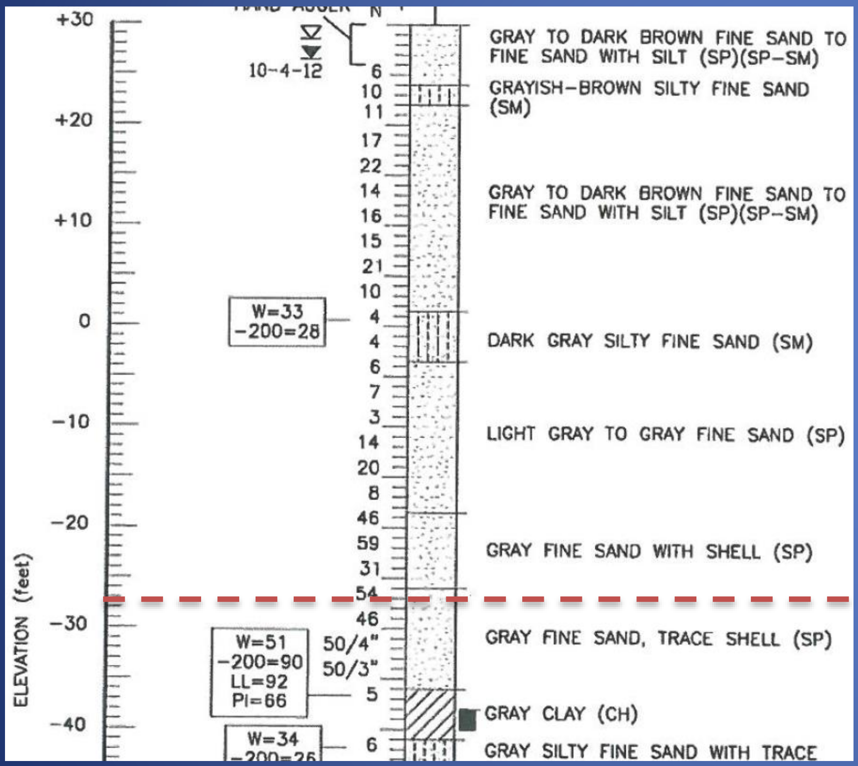
- Detection
  - In order to address the issue in design it must be recognized during the field exploration
  - SPT borings are the most common In-Situ testing method used in Florida for preliminary investigations

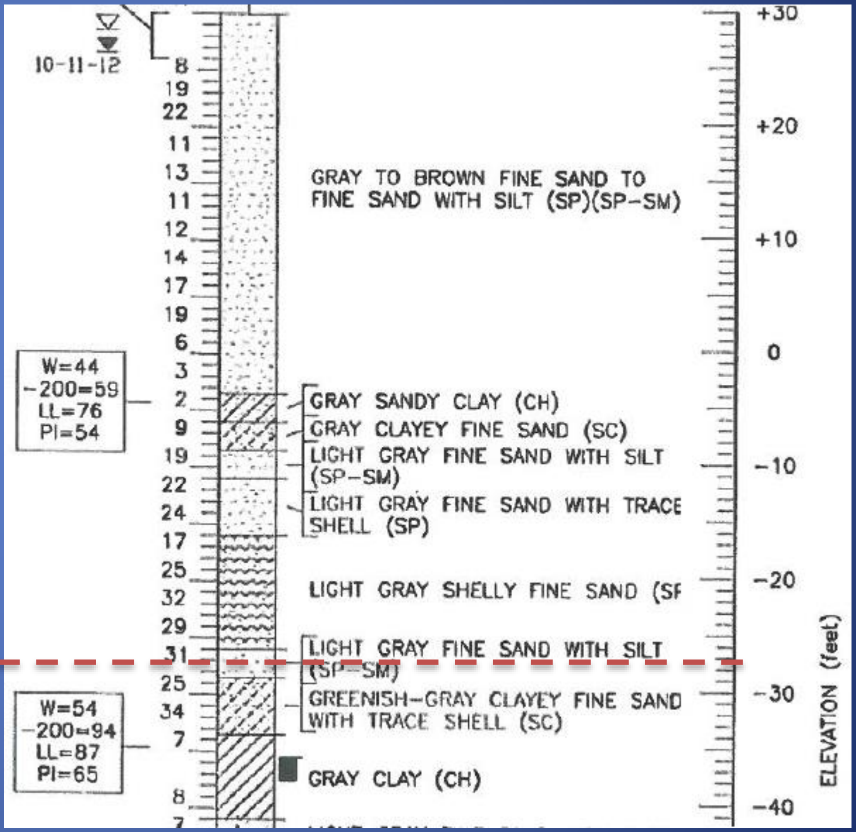
Relaxation - Design





Relaxation - Design





Relaxation - Design

Granular Materials		
Relative Density	**Safety Hammer SPT N-Value (Blow/Foot)**	**Automatic Hammer SPT N-Value (Blow/Foot)**
Very Loose	Less than 4	Less than 3
Loose	4 – 10	3 – 8
Medium Dense	10 – 30	8 – 24
Dense	30 – 50	24 – 40
Very Dense	Greater than 50	Greater than 40

May dilate during pile driving

Relaxation - Design

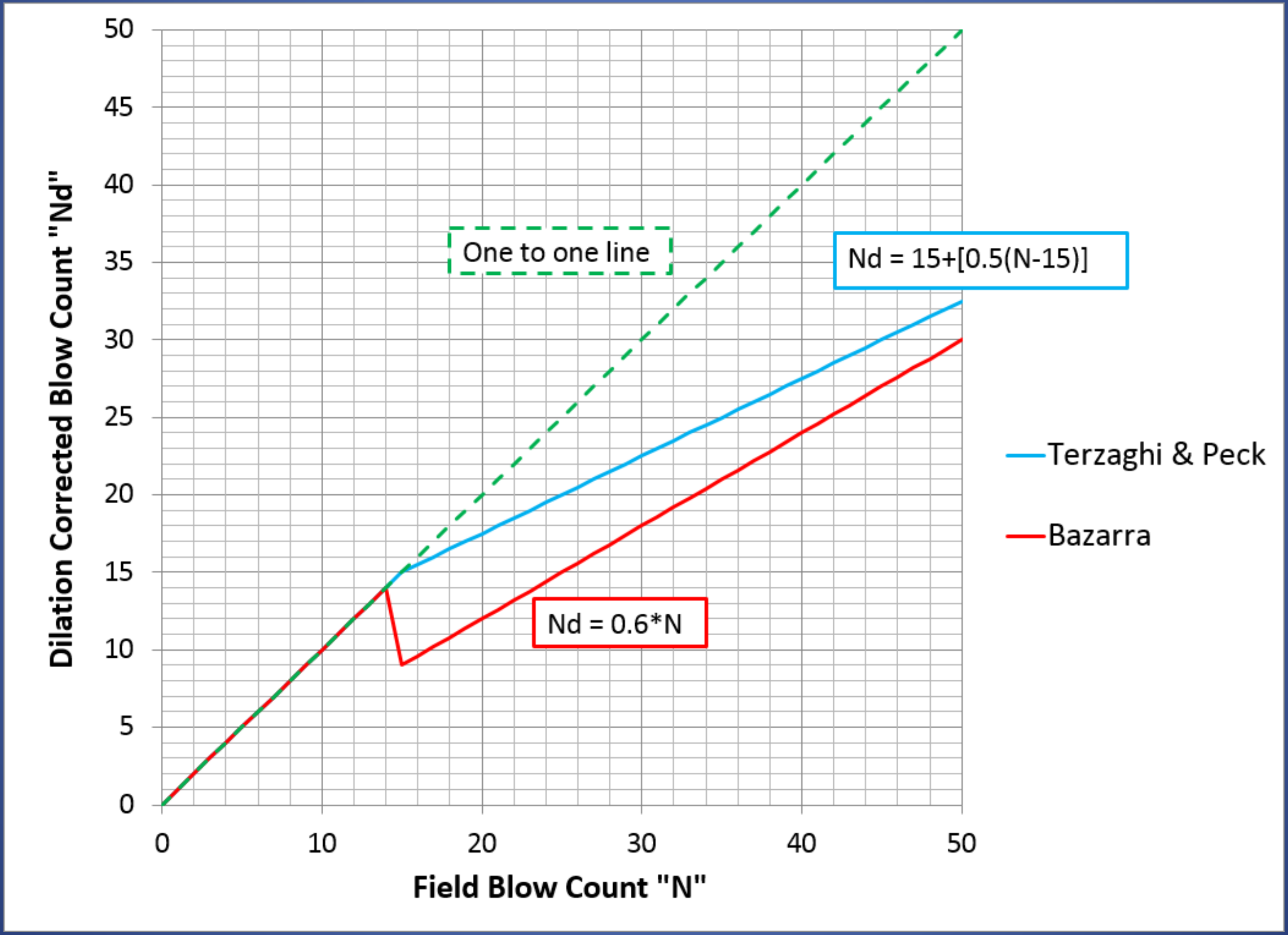
- Corrections to SPT "N" for preliminary design in dilatant soils
  - Terzaghi. For N > 15

$$N_d = 15 + 0.5(N-15)$$

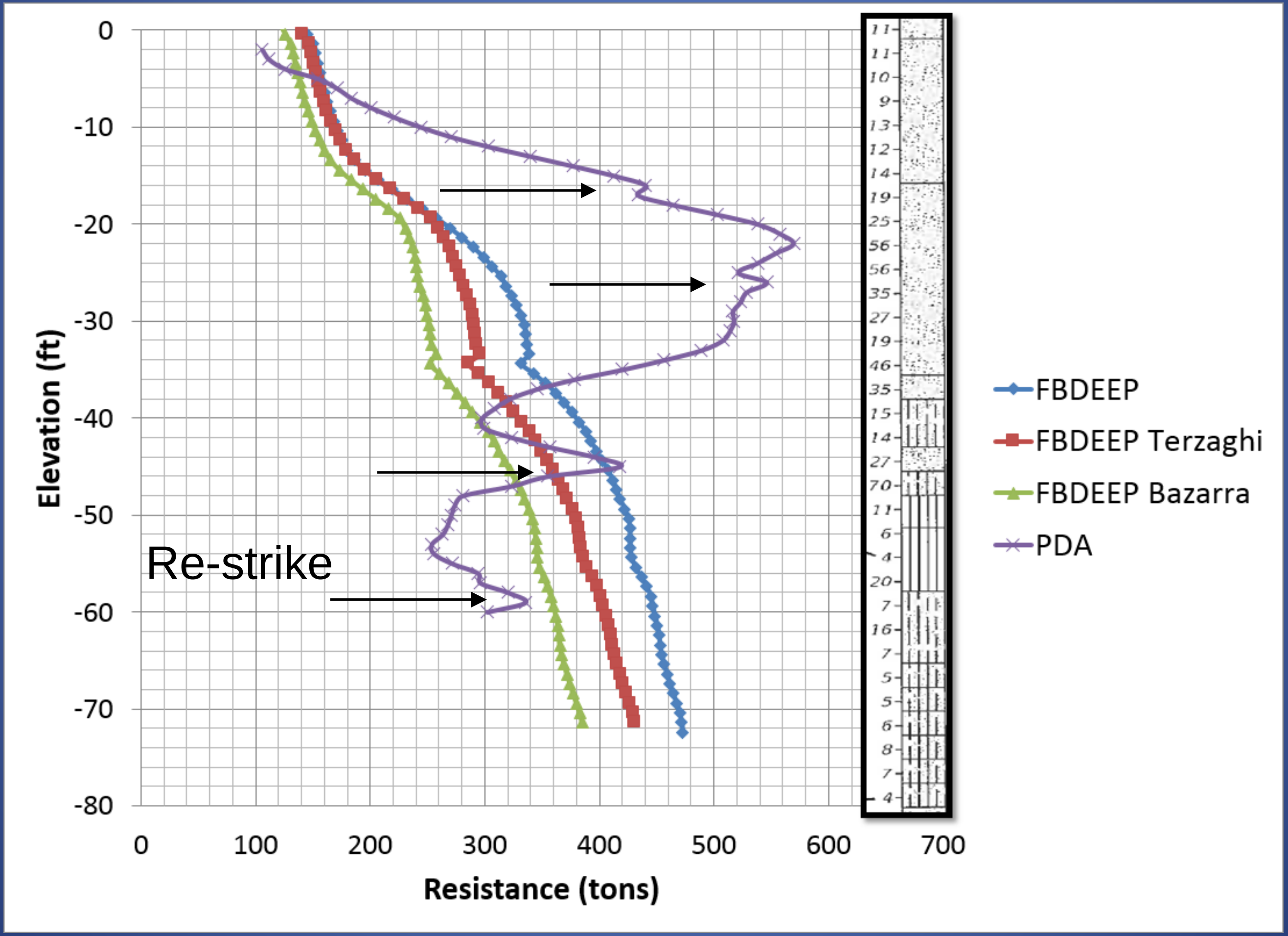
  - Bazarra. For N > 15

$$N_d = 0.6(N)$$

Relaxation - Design

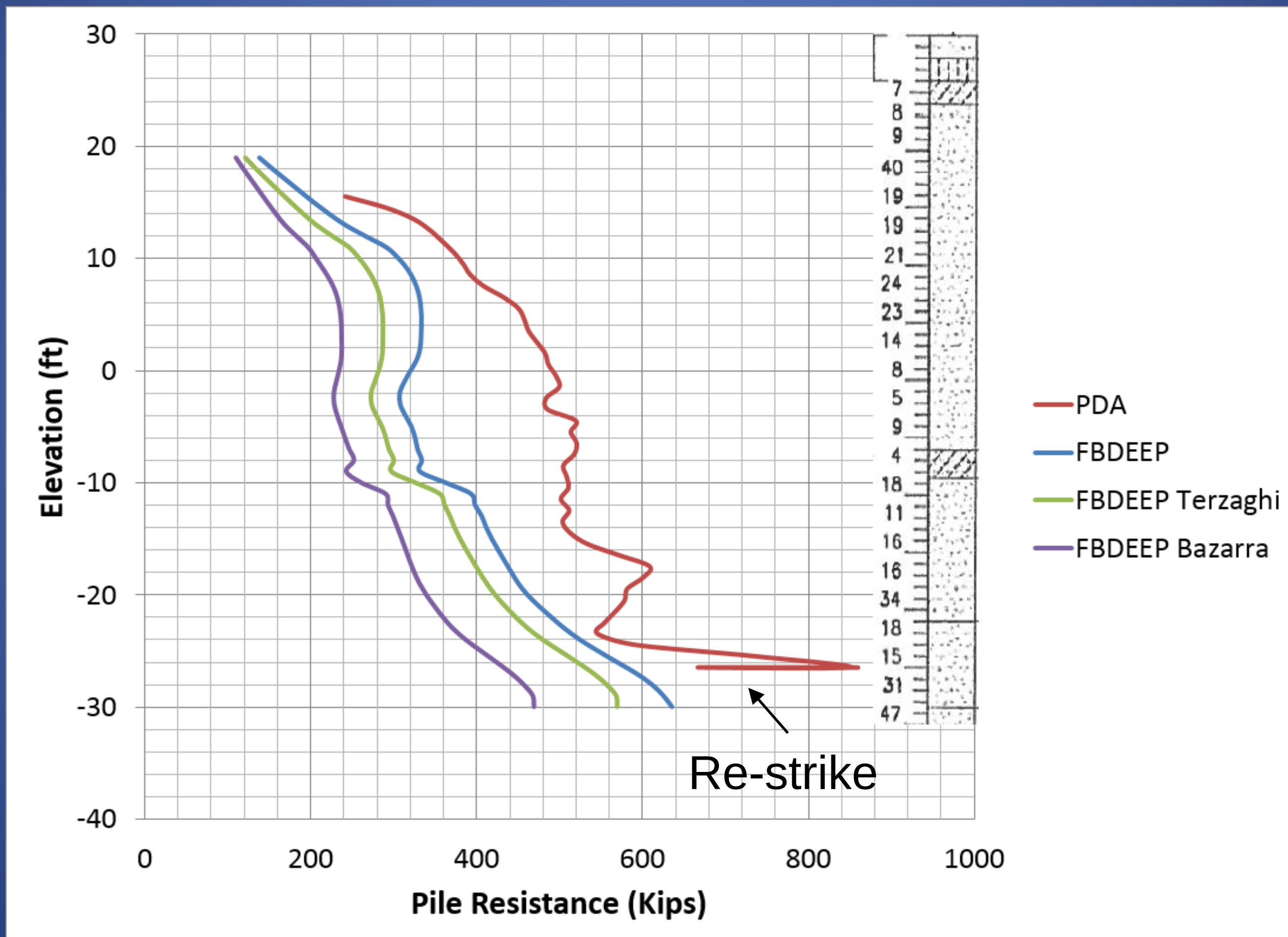


Relaxation - Design

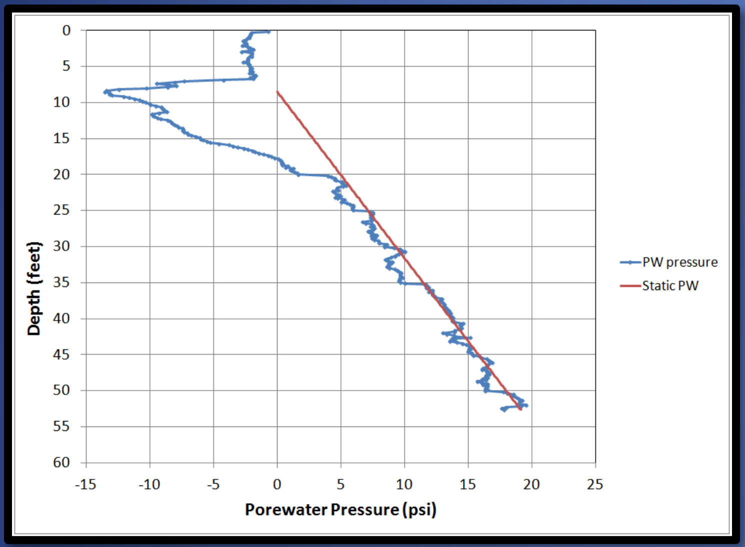


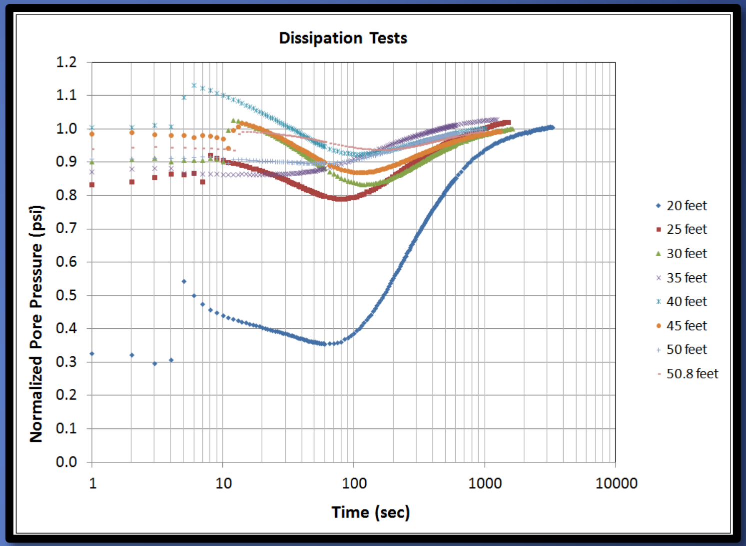
Four re-strikes at different elevations

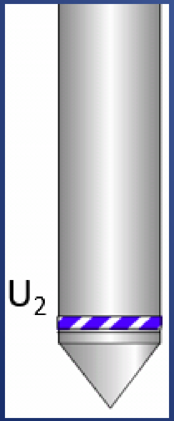
Relaxation - Design



Relaxation - Design

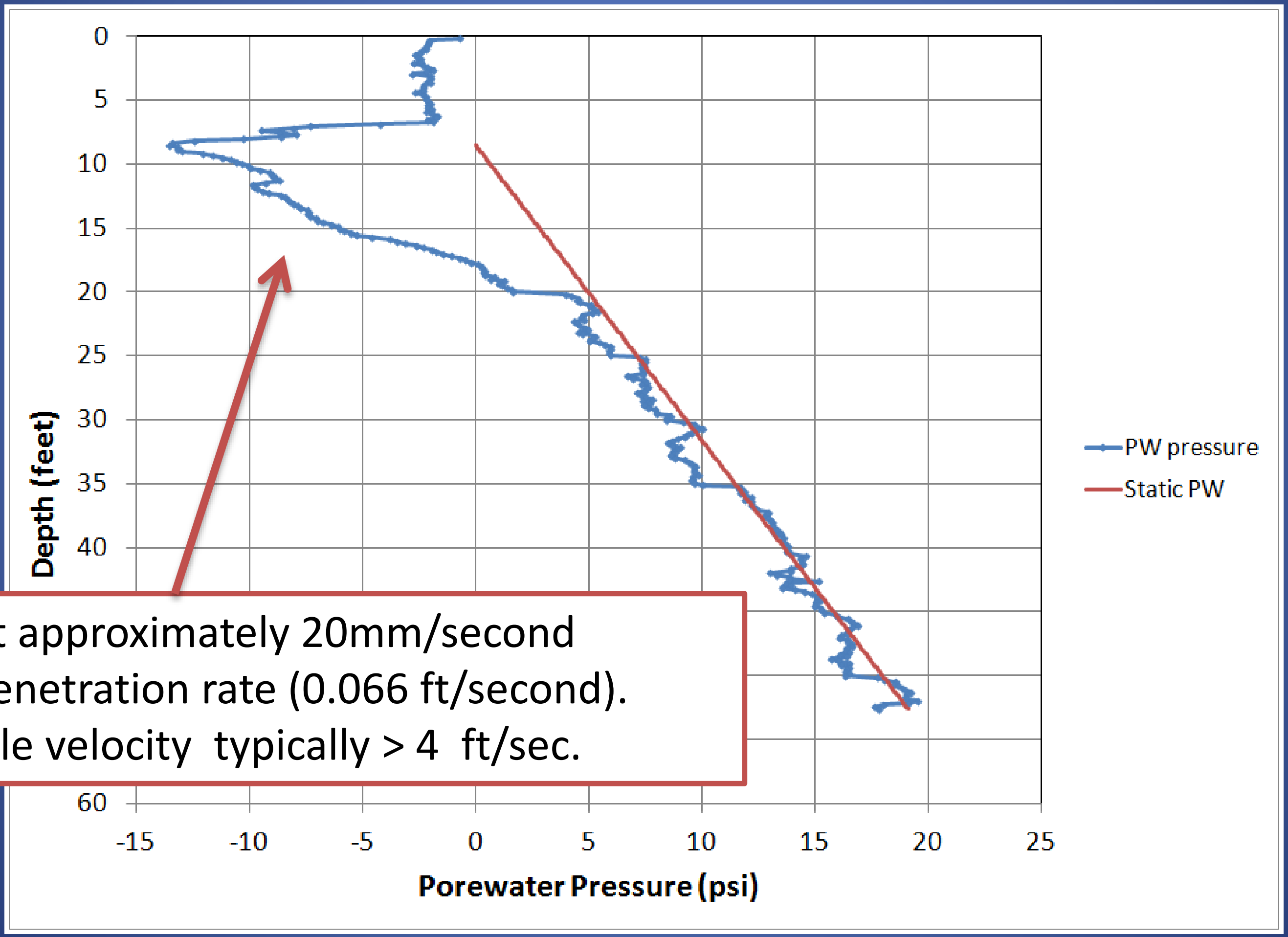






- CPT-U with shoulder pore pressure transducer. Check for areas where U goes negative
- Perform dissipation tests at depths of concern

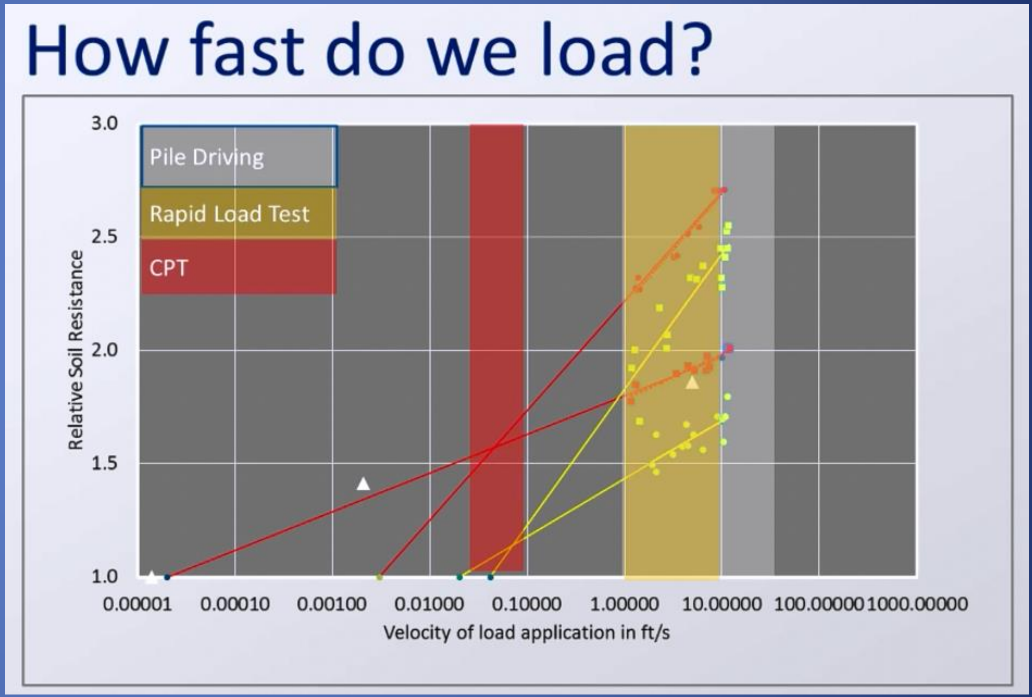
Case History
SR 9B Phase I – Duval County



At approximately 20mm/second penetration rate (0.066 ft/second). Pile velocity typically > 4 ft/sec.

Relaxation - Design

- What is the impact of shearing velocity on dilation?
  - CPTu vs. pile driving?



Source: Dr. Frank Rausche
(2015 DFI Osterberg Lecture)

Relaxation - Construction

- Set check (re-strike) during installation of Test Piles
  - As soon as the nominal bearing resistance (NBR) is reached (below minimum tip)
  - Use CPT-U results as a general guide on where to perform set-checks

Relaxation - Construction

Production Pile Capacities at Various Time of Testing
Max. NBR of 608 kips

Pile No.	* Time of Testing	Date	Blow No.	Ref. Elev (ft)	Ground Elev. (ft)	Depth below Ref. Elev. (ft)	Pile Tip Elev (ft)	Pen. (ft)	**Jc	WS (ft/sec)	WC (ft/sec)	CAPWAP Pile Capacity (kips)	Blow Count (Bpf)	CAPWAP MQ
Pile 1	EOD	10/1/13	EOD	40.54	28.20	67.00	-26.46	54.66	0.36	13966	13966	930	105	1.72
	8-day RS	10/9/13	3	40.54	28.20	67.02	-26.48	54.68	0.42	13966	13966	840	240	1.45
Pile 2	EOD	9/30/13	EOD	40.54	28.20	68.00	-27.46	55.66	0.30	13966	13966	810	146	2.42
	9-day RS	10/9/13	3	40.54	28.20	68.02	-27.48	55.68	0.49	13966	13966	790	160	1.55
Pile 3	EOD	9/30/13	EOD	40.54	28.20	67.00	-26.46	54.66	0.26	13966	13966	948	109	2.1
	9-day RS	10/9/13	3	40.54	28.20	67.02	-26.48	54.68	0.44	13966	13966	859	160	1.59
Pile 4	EOD	9/30/13	EOD	40.45	28.20	67.00	-26.55	54.75	0.30	13966	13966	870	109	1.55
	9-day RS	10/9/13	3	40.45	28.20	67.02	-26.56	54.76	0.44	13966	13966	882	160	1.72
Pile 5	EOD	9/30/13	EOD	40.55	28.20	67.00	-26.46	54.66	0.31	13966	13966	931	100	1.97
	9-day RS	10/9/13	4	40.55	28.20	67.03	-26.48	54.68	0.54	13966	13966	811	160	1.63
Pile 6	EOD	9/27/13	EOD	31.38	28.24	56.99	-25.61	53.84	0.24	13966	13966	969	87	1.53
	3-day RS	9/30/13	3	31.38	28.24	57.03	-25.65	53.89	0.50	13966	13966	910	96	1.55
	4-day RS	10/1/13	4	31.38	28.24	57.21	-25.83	54.06	0.41	13966	13966	929	96	2.08
	12-day RS	10/9/13	3	31.38	28.24	57.23	-25.85	54.09	0.40	13966	13966	940	120	1.91
Pile 8	EOD	9/27/13	EOD	31.38	28.24	57.00	-25.62	53.86	0.22	13966	13966	910	98	1.76
	3-day RS	9/30/13	3	31.38	28.24	57.03	-25.65	53.89	0.45	13966	13966	794	108	2.16
	4-day RS	10/1/13	4	31.38	28.24	57.19	-25.81	54.05	0.50	13966	13966	750	96	1.85
	12-day RS	10/9/13	2	31.38	28.24	57.31	-25.93	54.17	0.49	13966	13966	705	120	1.79
											Max.	969		
											Min	705		

*EOD: End of initial drive, XX-day: Elapsed time after initial drive
** determined based on CAPWAP analyses.

Terracon (M. Kim)

Relaxation - Summary

- Recognize potential problem areas during SPT exploration
- Perform CPT-U, with dissipation tests at various elevations
- Use corrections on blow count to estimate test pile length in FBDEEP
- Re-strike piles below minimum tip where NBR is achieved to confirm capacity. Revise minimum tip elevation if required.

Relaxation of Driven Pile Resistance in Granular Soils

- Questions?