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(S. Olson)

Liquefied shear strength & flow slides – challenges and paths forward

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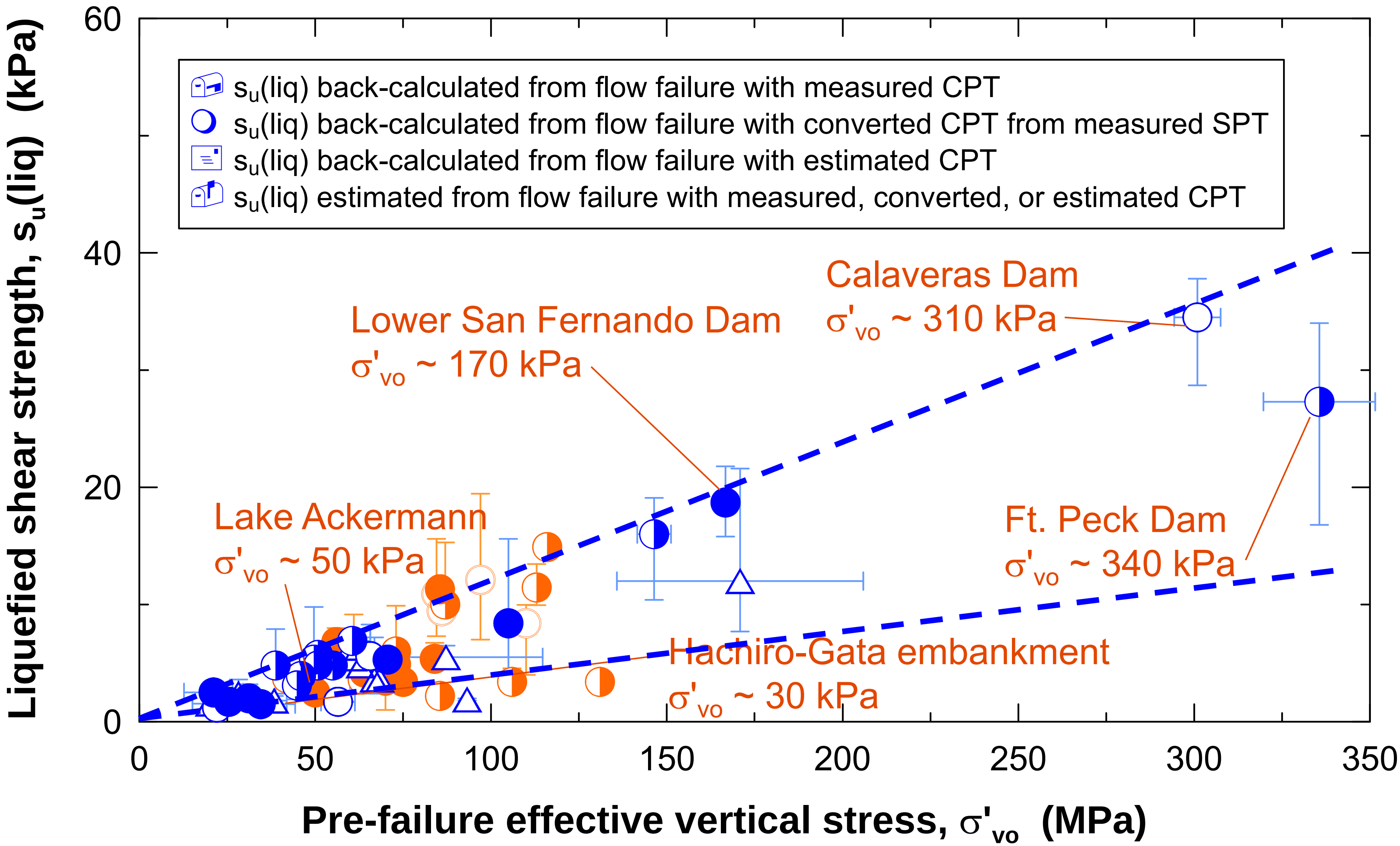
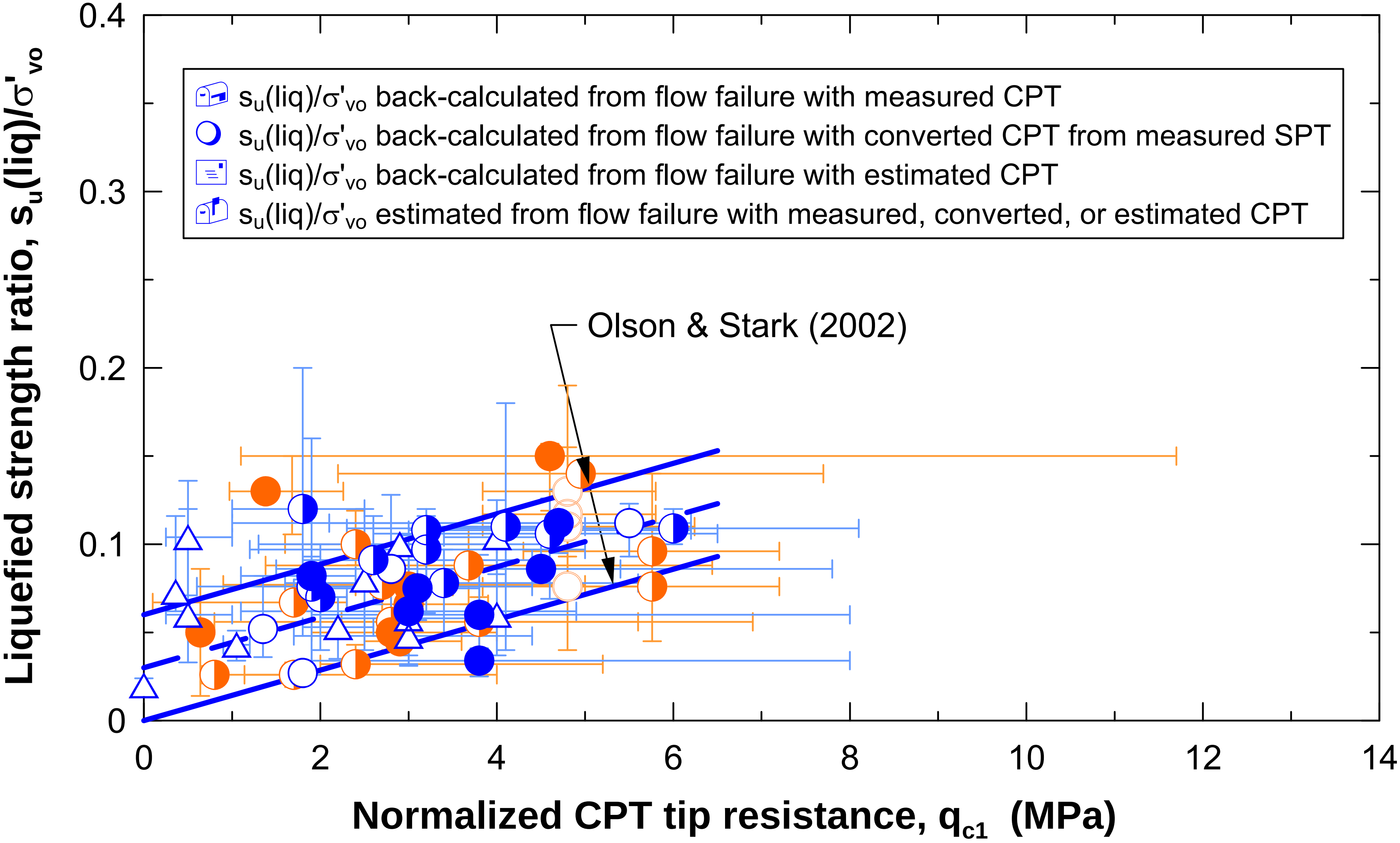
U.S.-New Zealand-Japan International Workshop
Liquefaction-Induced Ground Movements Effects
2 – 4 November 2016

C1. Does s_u(liq) vary with σ'_{vo} directly?

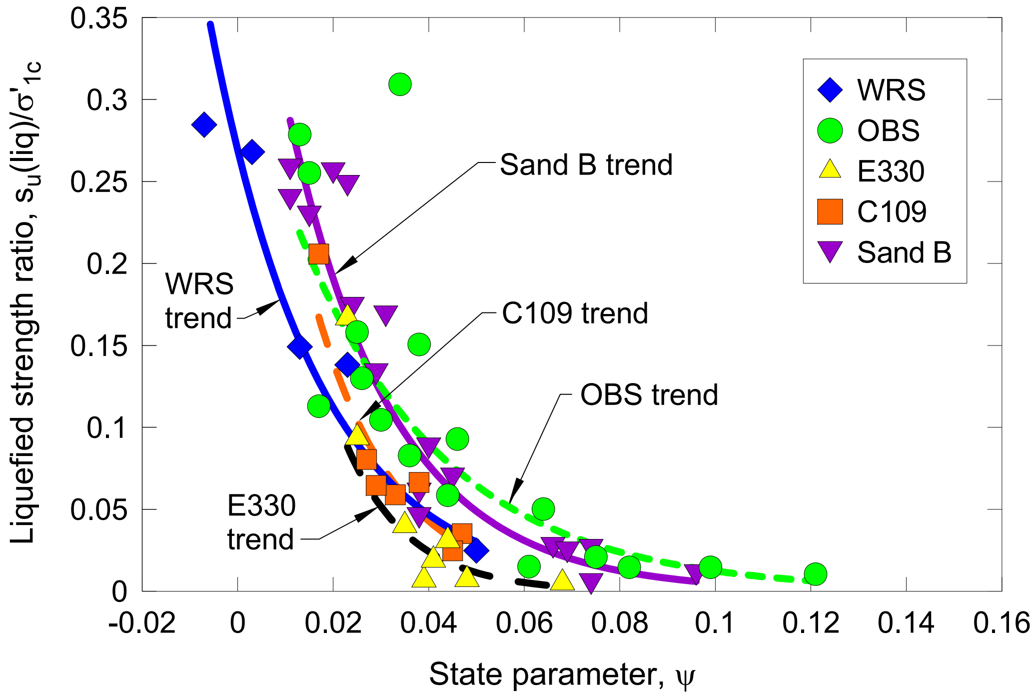


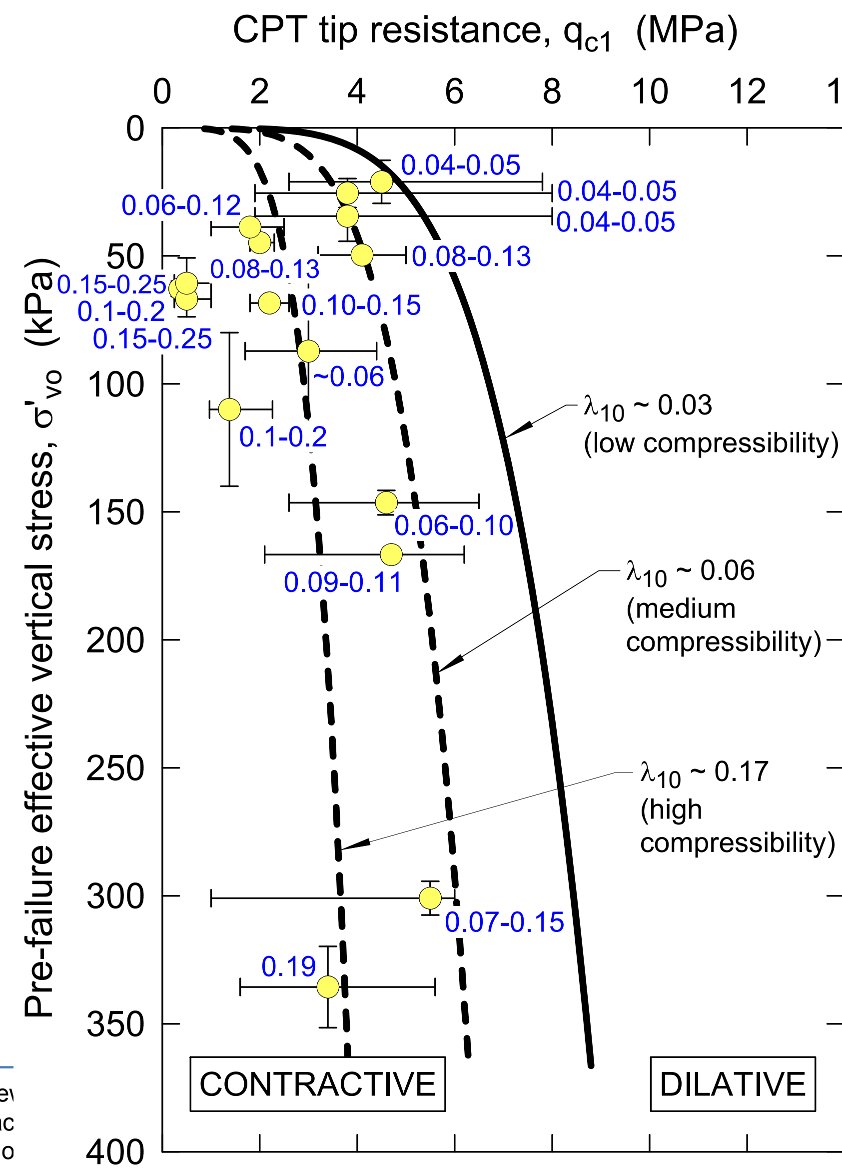
C2. Does $s_u(liq) / \sigma'_{vo}$ vary with penetration resistance?



Olson & Stark (2002);
Olson & Muhammad (2016, in review)

C3. Can $s_u(liq)/\sigma'_{vo}$ be extrapolated indefinitely?





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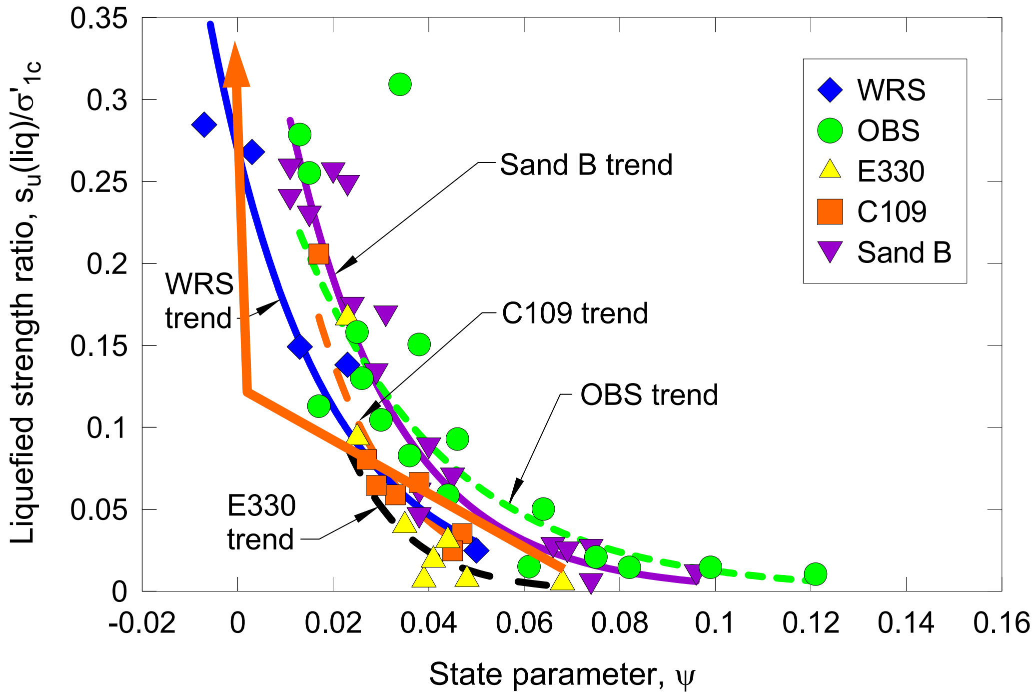

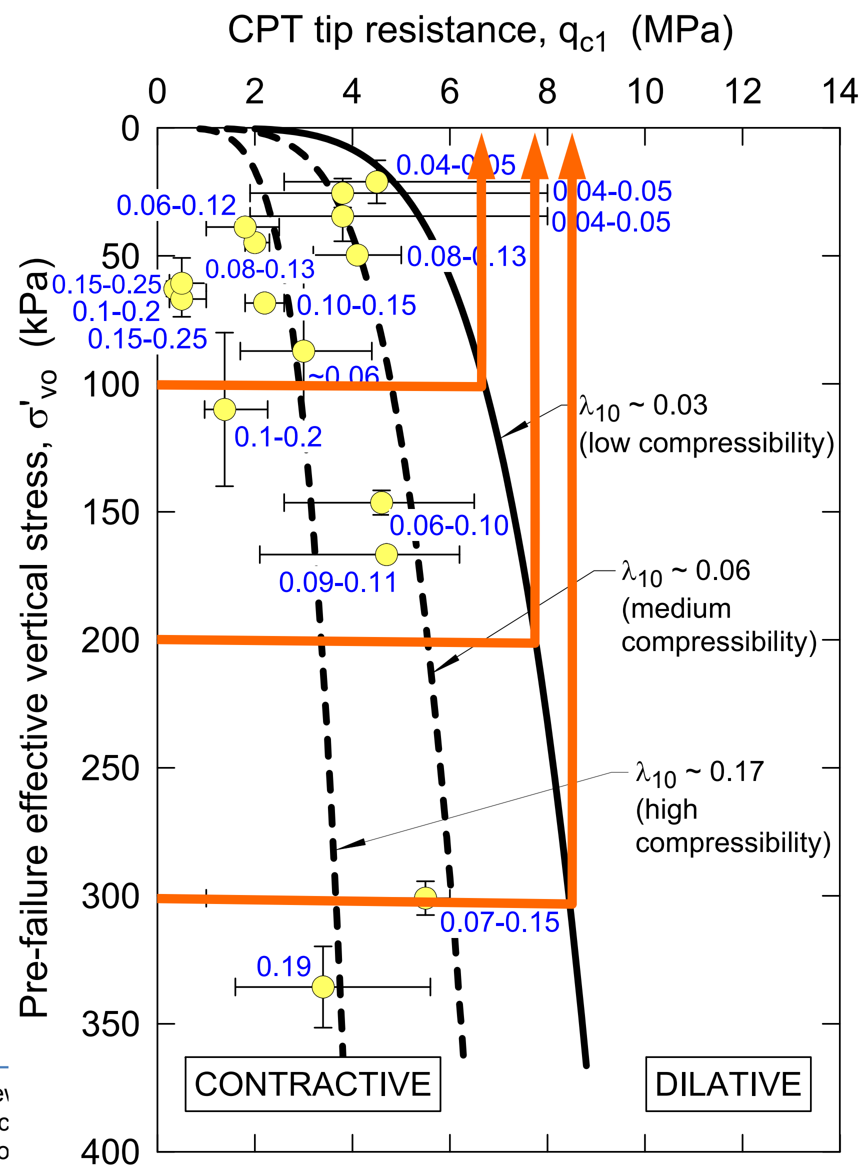

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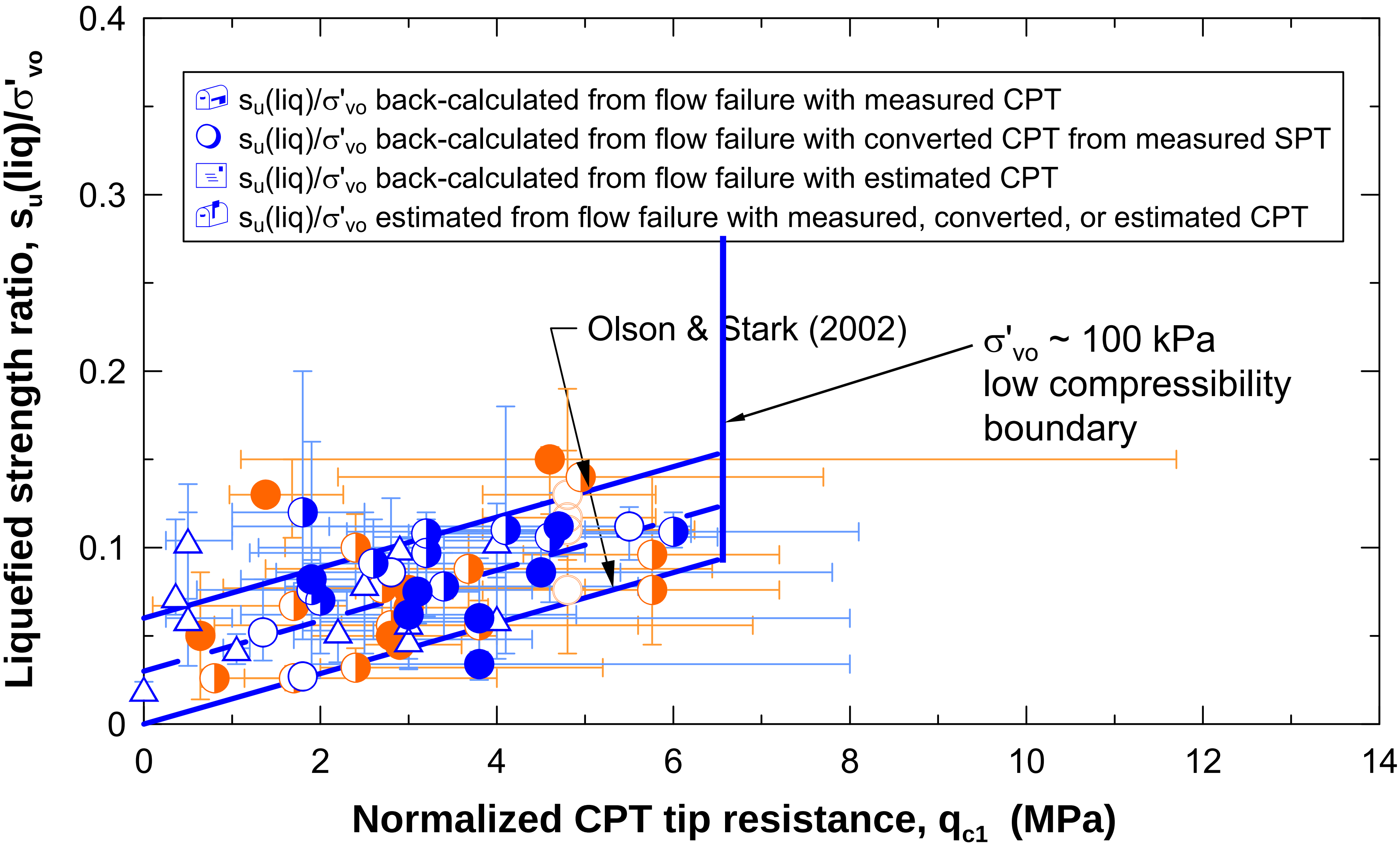

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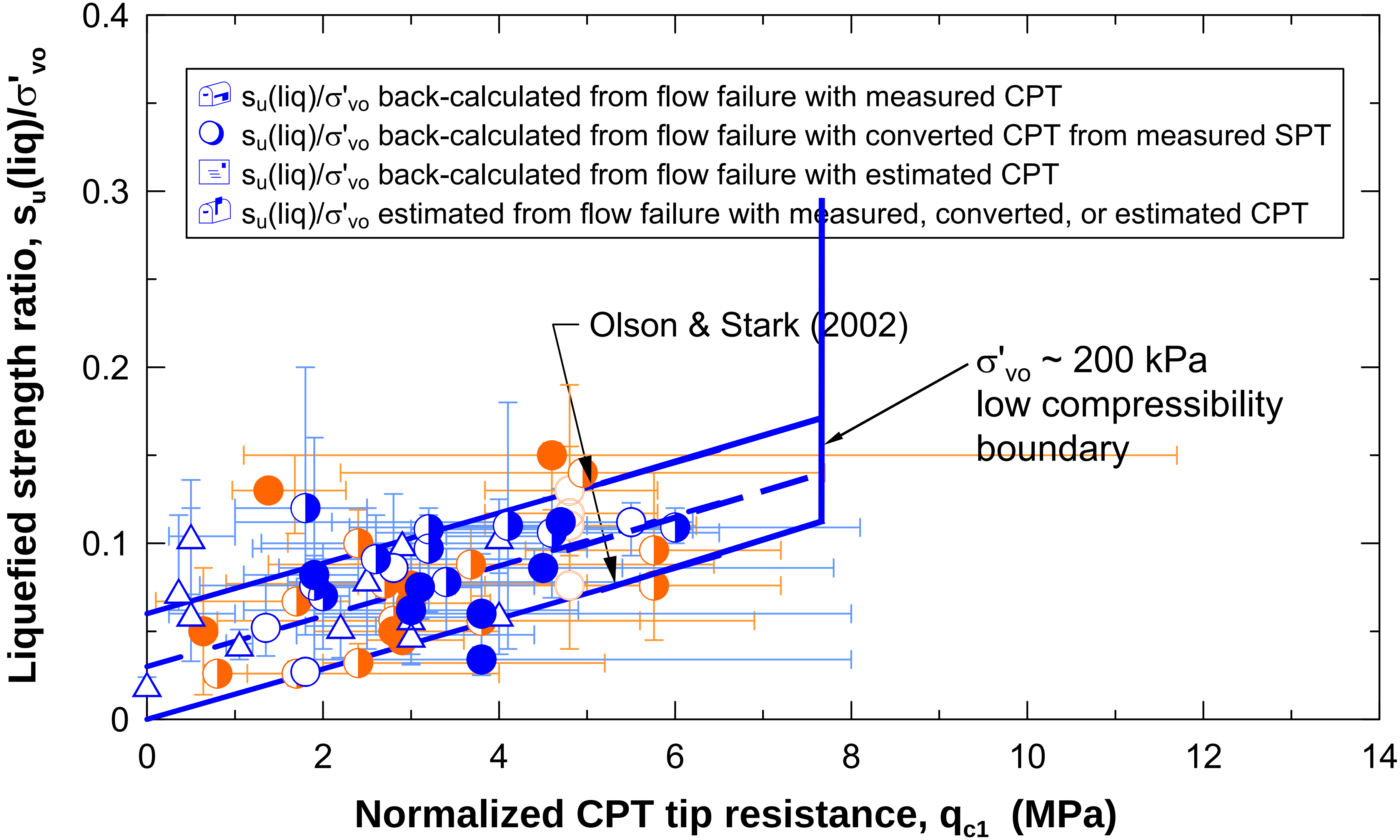

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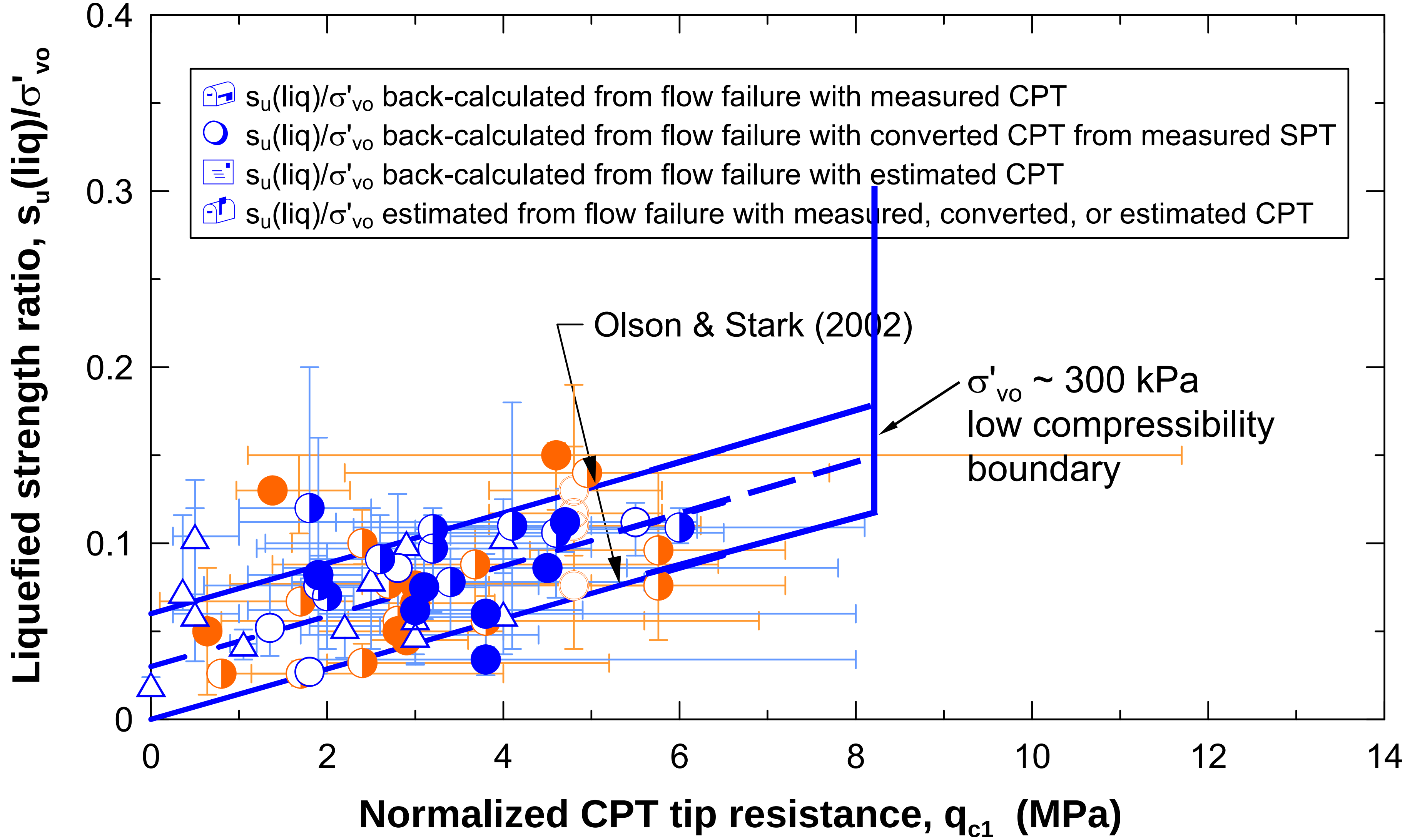
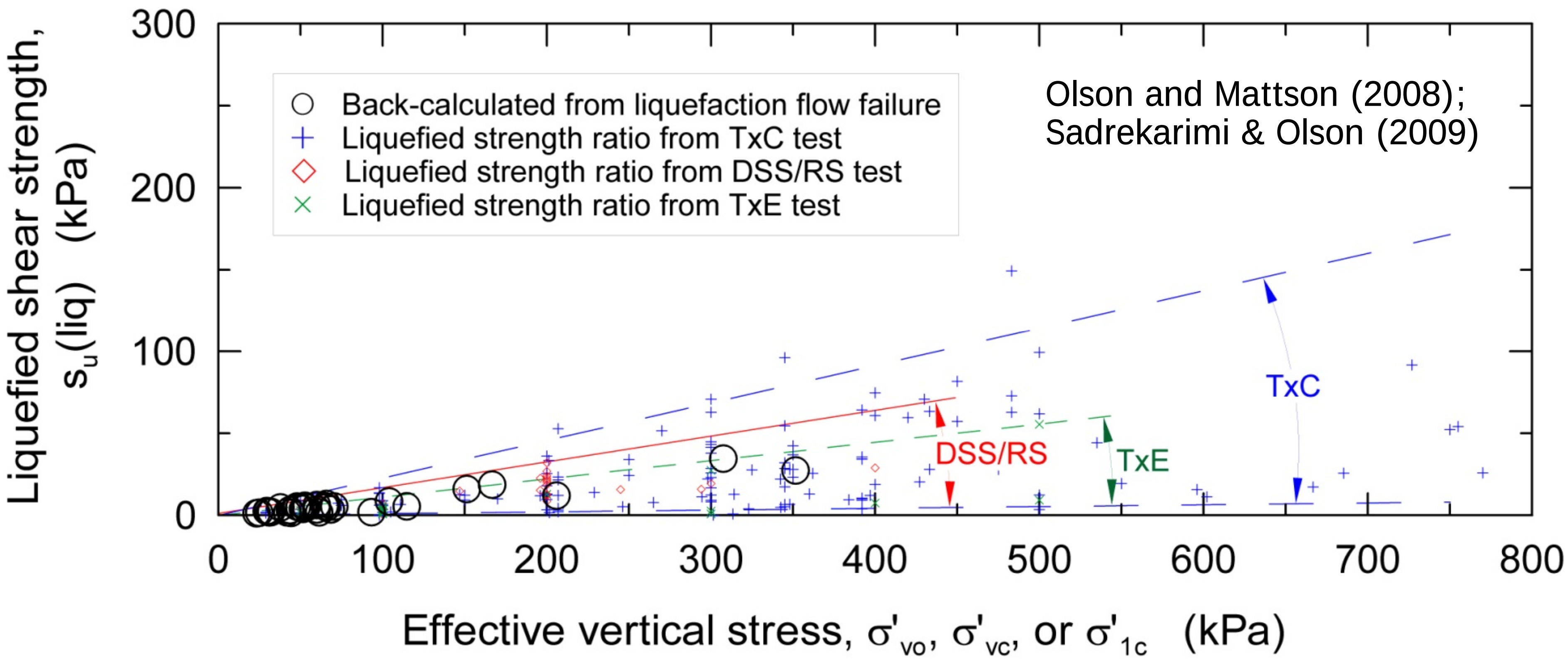
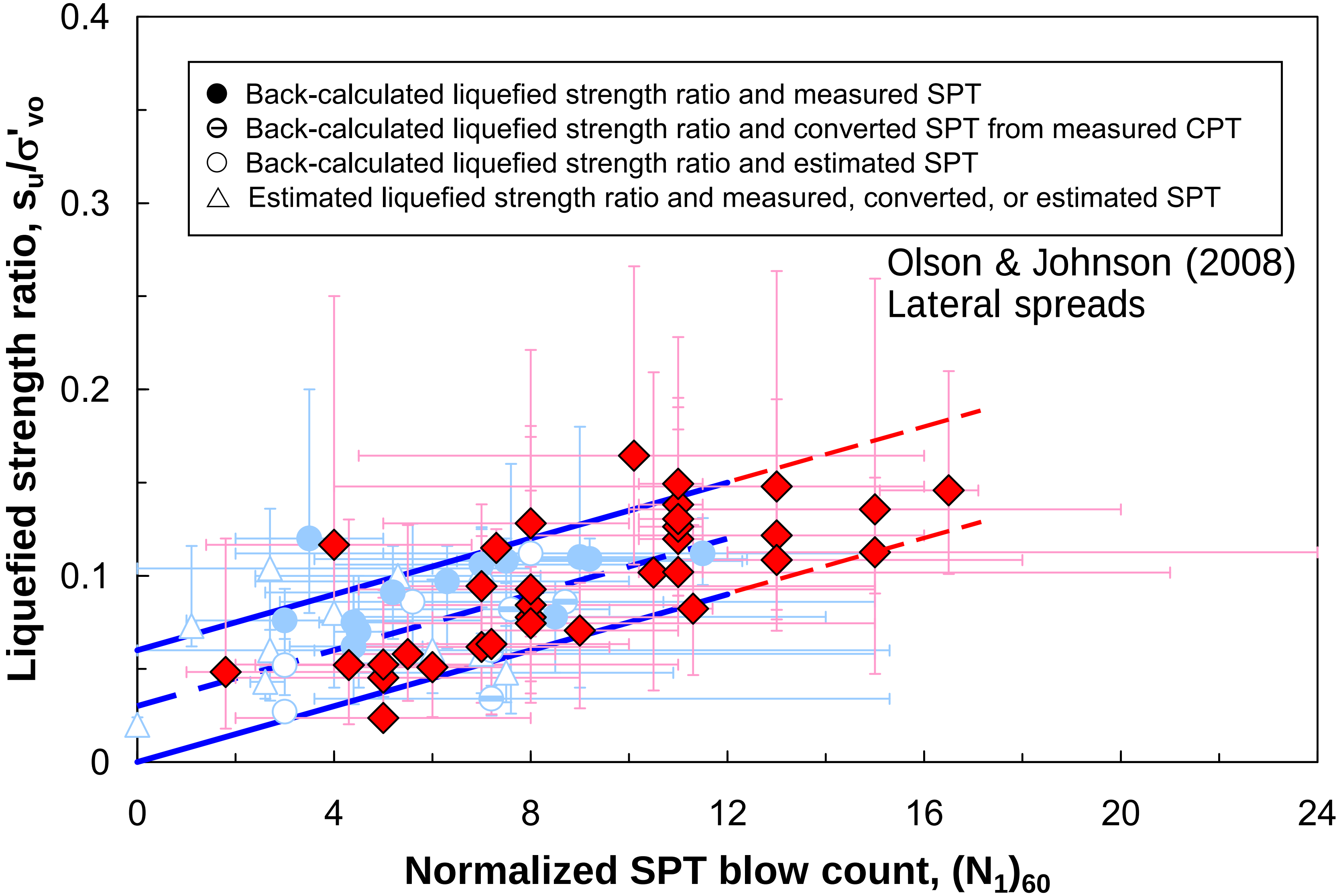


Olson & Stark (2002);
Olson & Muhammad (2016, in review)

C4. Does lab data approximate field case histories?



C5. How does PWP / void redistribution affect s_u(liq)?



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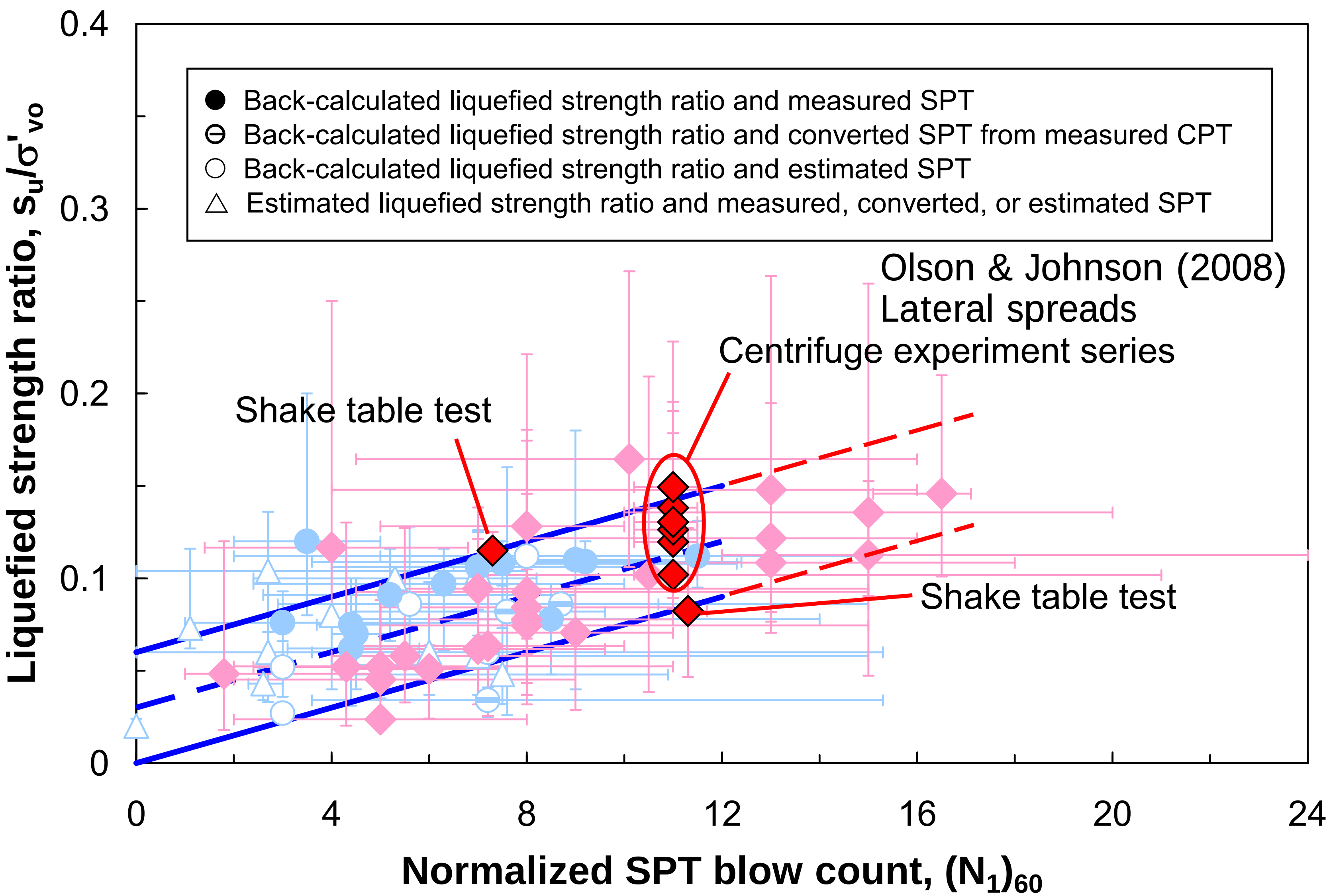
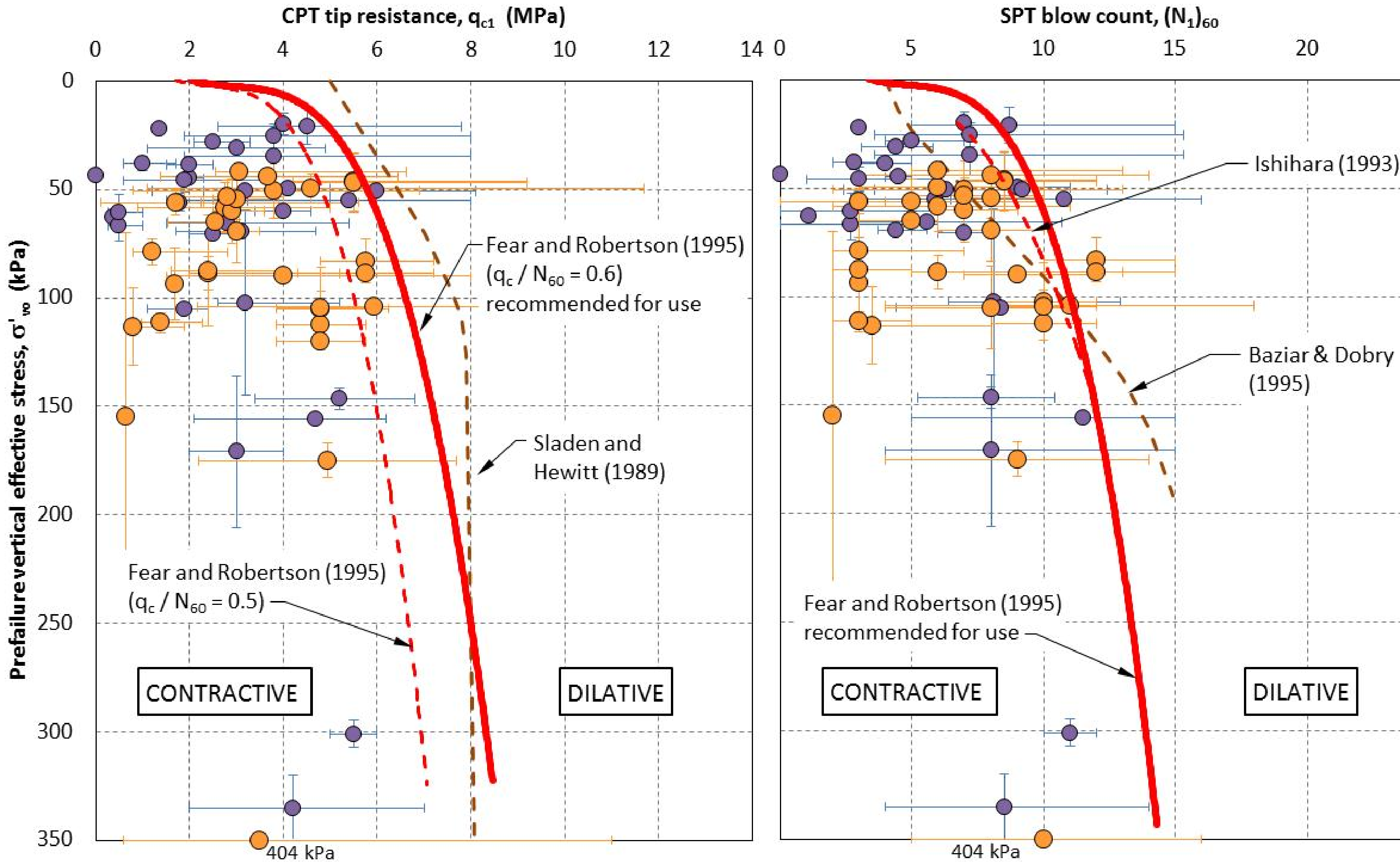


C6. What is $s_u(liq)$ in medium dense to dense sandy soils?



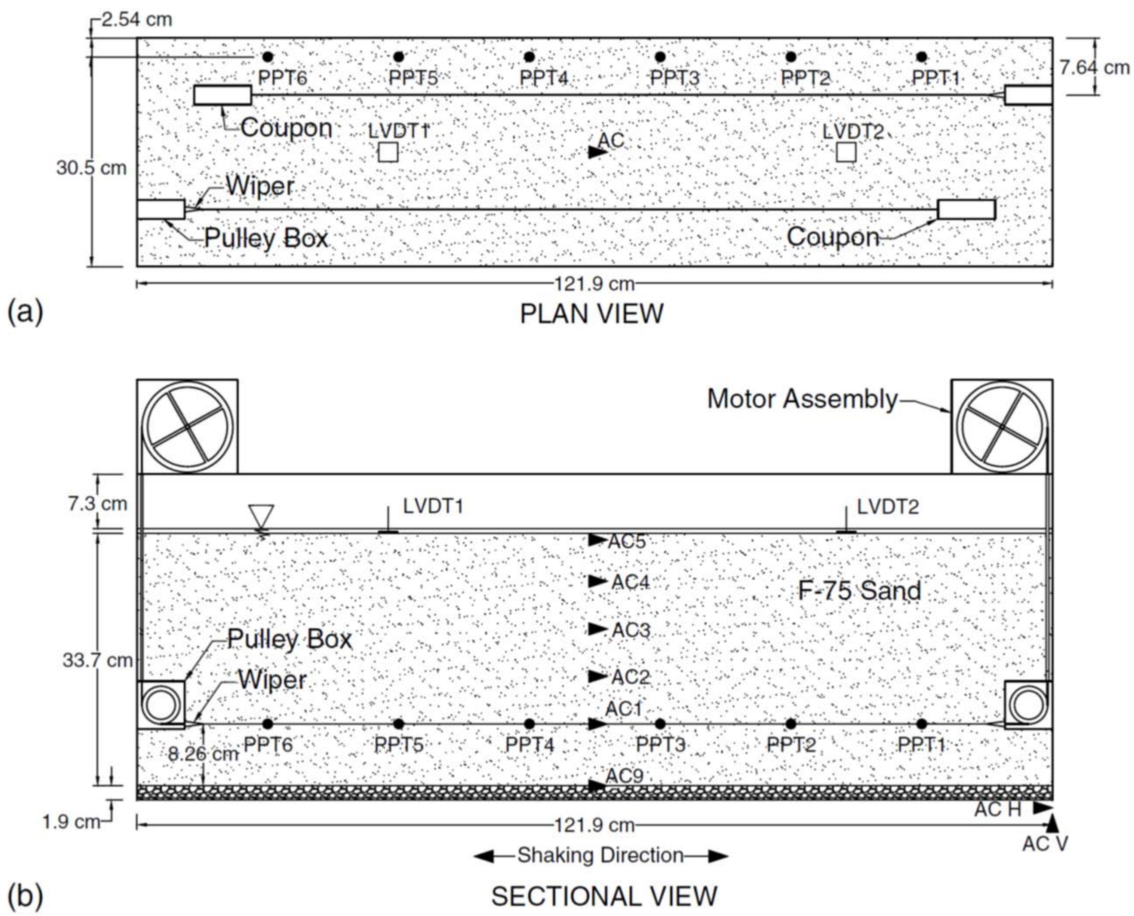
PF1. Better documented field case histories

- Complete pre- and post-failure geometries (remote imagery)
- Well-defined stratigraphy with Penetration resistance (SPT, CPT, BPT) and Vs
- Well-defined pre-failure phreatic surface
- Strengths for non-liquefied soils

- Development of instrumented field sites for flow slides?

PF2. Novel methods to measure s_u(liq)

- Field vane shear test (W. Charlie's piezovane?) in silty soils
- T-bar in blast-induced liquefied soils
- Coupon pull test in centrifuge (Dewoolkar et al. 2016)



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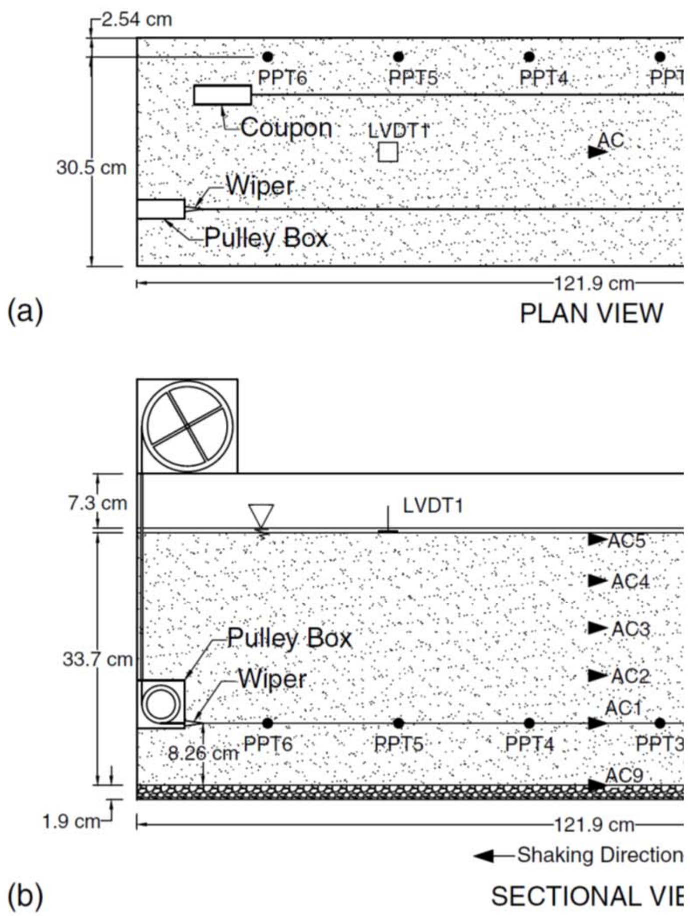

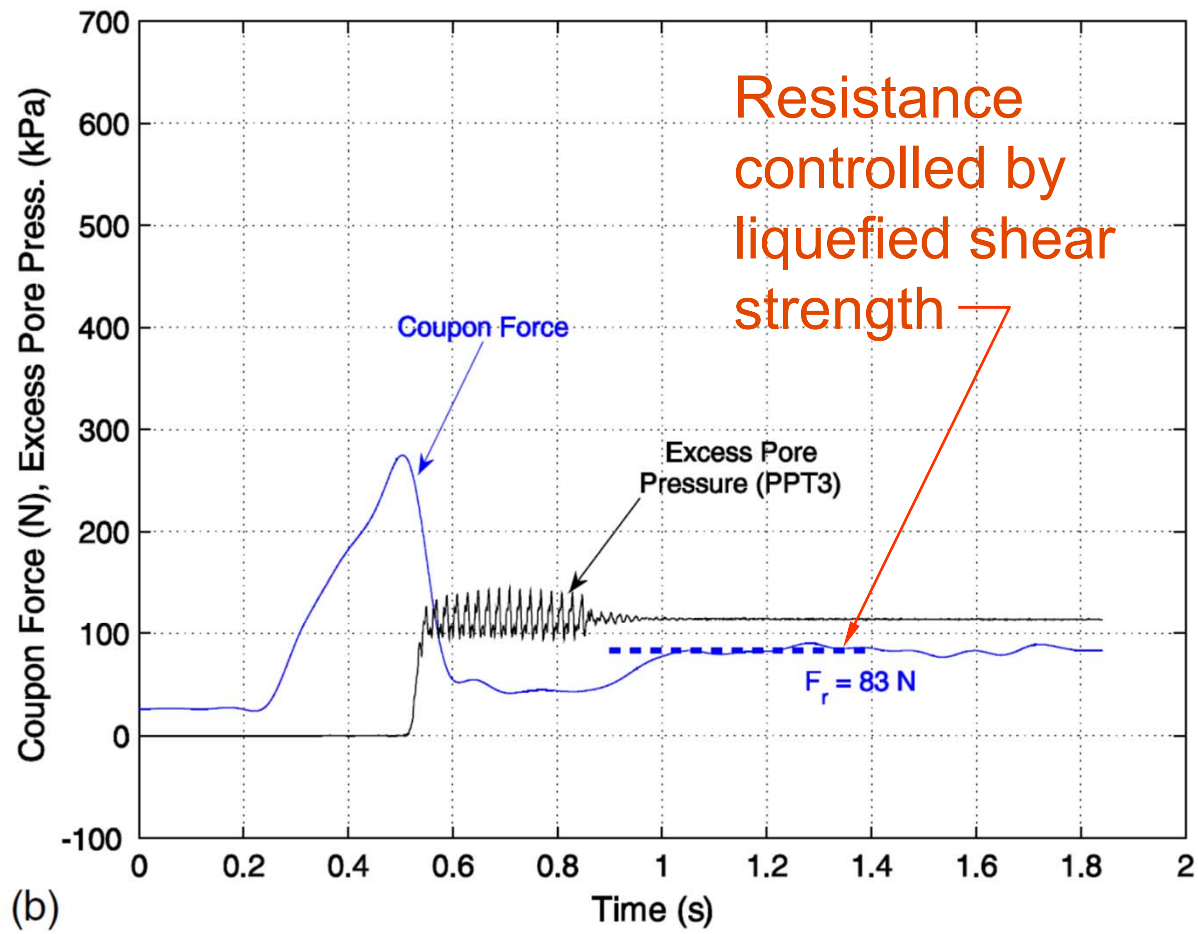

Thank you!

Questions?

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(G. Chiaro)

U.S.-New Zealand-Japan International Workshop

Liquefaction-induced Ground Movements Effects

UC Berkeley, California, 2-4 November 2016

Understanding the mechanics of earthquake-induced flow liquefaction: from observations to predictions

Gabriele Chiaro

Lecturer, Canterbury University, Christchurch, New Zealand

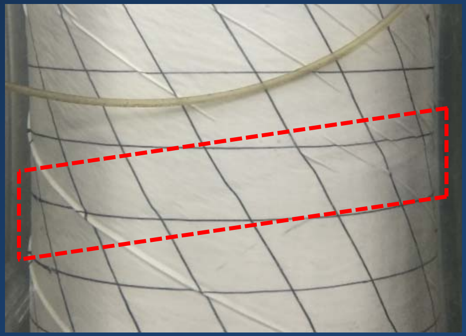
UC UNIVERSITY OF CANTERBURY

Liquefaction-induced failure of sloped ground

1964 Niigata Earthquake, Japan (a_{max}=0.16 g; M=7.5)

Laboratory observations
Failure for liquefiable soils in sloped ground*

(Chiaro et al., 2012)



FAILURE

Liquefaction

Δu=100%
γ_{DA}=5%

Shear failure
(accumulation strain)
No liquefaction

Flow-type
(abroupt development deformation)

Δu=100% and γ_{DA}>50% in just a few cycles

*** Loose fully-saturated sandy soils**

Key challenges

Understanding the failure mechanisms

Stresses

- **Initial static shear stress** (i.e. sloped ground)
- **Cyclic shear stress** (i.e. earthquake), **Number of cycles**
- **Confining pressure level**, **OCR**

Soil strength

- **Density state** (loose, dense), **soil structure** and **fabric**
- **Degree of saturation** (fully or partially saturated)
- **Soil type** (clean sand, gravelly sand or sand with fines)

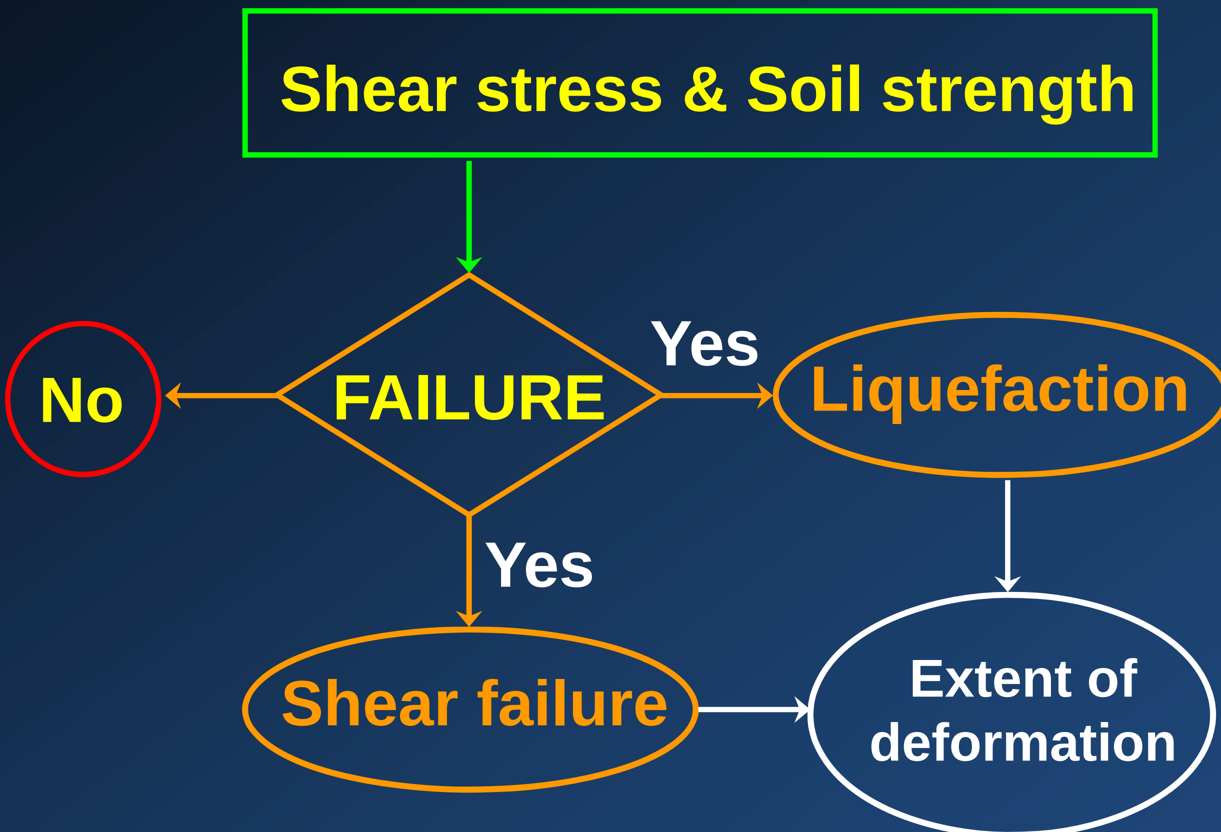
- **Testing conditions**
  - element tests (triaxial, simple shear, torsional shear)
  - model tests

Paths forward

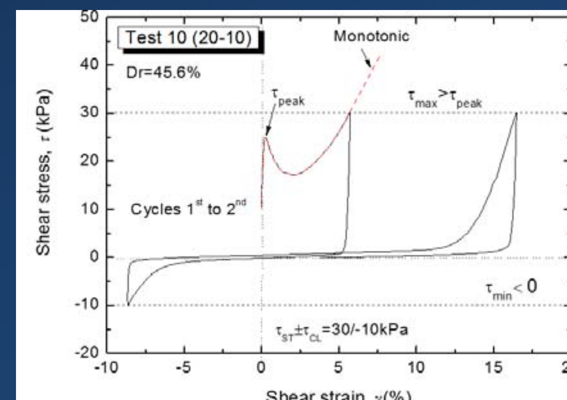
Predictive method including slope effects

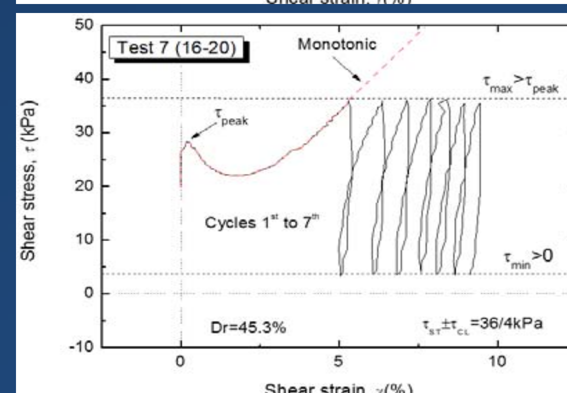
(not using Kα)

Laboratory ⟺ Field

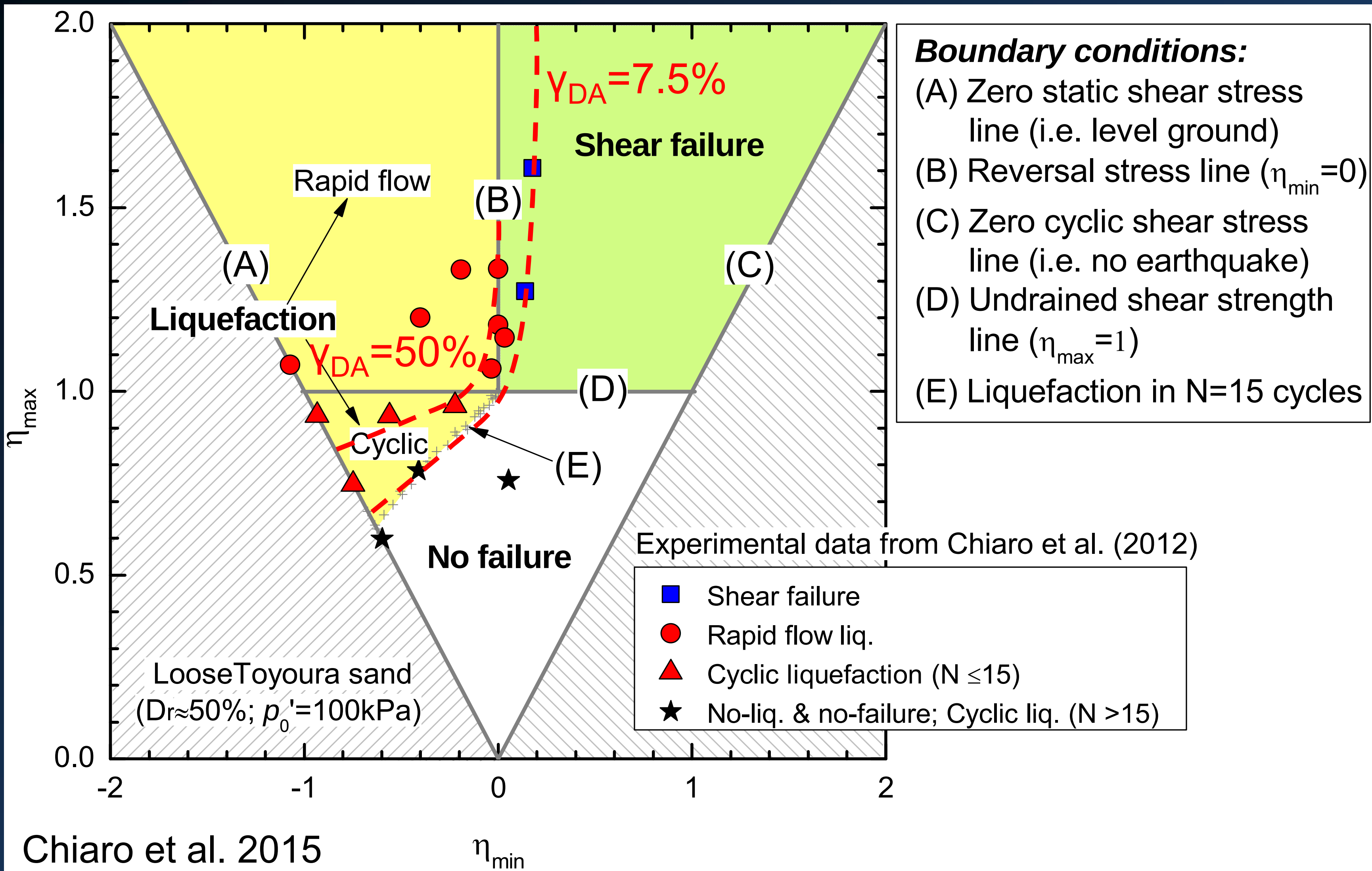


CONSEQUENCES





Lab observations (torsional simple shear tests)



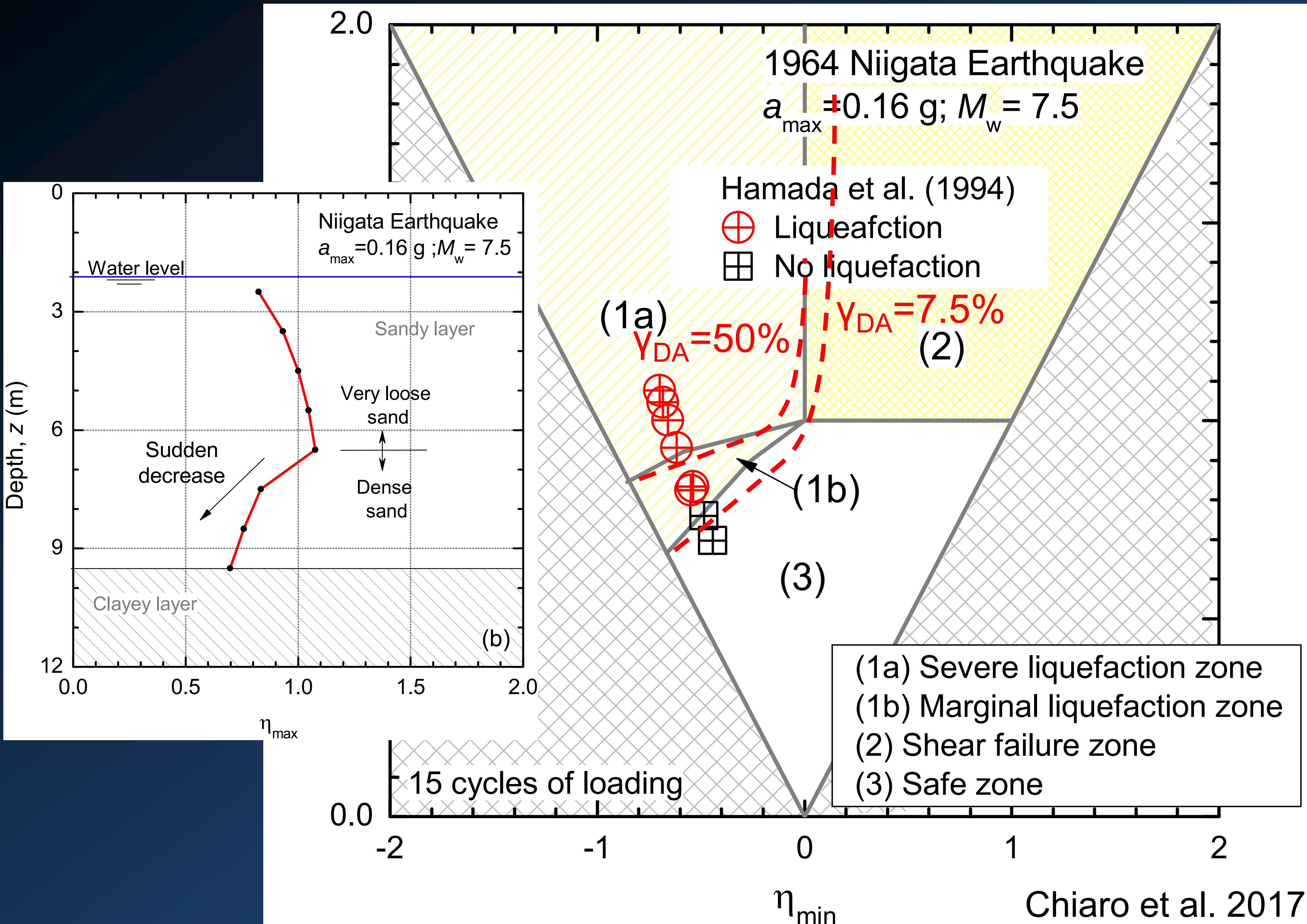
Triggers

$$\eta_{max} = \frac{SSR + CSR}{USS}$$

$$\eta_{min} = \frac{SSR - CSR}{USS}$$

Soil strength

Field observations



References

Chiaro G., Koseki J. & Sato T. (2012). Effects of initial static shear on liquefaction and large deformation properties of loose saturated Toyoura sand in undrained cyclic torsional shear tests. Soils and Foundations, 52(3): 498-510.

Chiaro G., Koseki J. & Kiyota T. (2015). New insights into the failure mechanisms of liquefiable sandy sloped ground during earthquakes. In: Proc. of the 6th International Conference on Earthquake Geotechnical Engineering, Nov. 1-4, Christchurch, New Zealand, CD-ROM, pp.8.

Chiaro G., Koseki J. & Kiyota T. (2017). An investigation on the liquefaction-induced sloped ground failure during the 1964 Niigata Earthquake. Geomechanics and Geoengineering: Geotechnical Hazards from Large Earthquakes and Heavy Rainfalls, Springer, 134-143.

Thank you for your kind attention !

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(Adda A-Z)

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Post-Liquefaction Response of Gravelly Soils

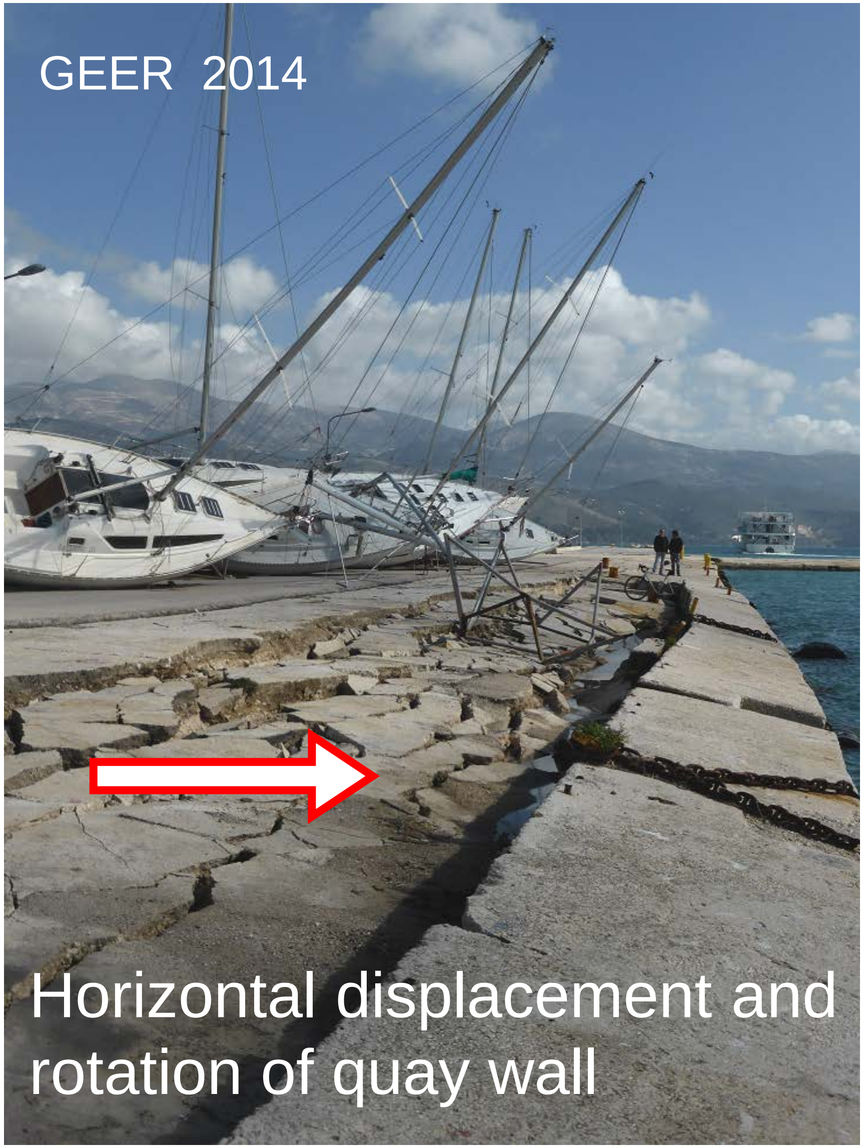
Adda Athanasopoulos-Zekkos
Associate Professor
Civil and Environmental Engineering
University of Michigan

US-NZ-Japan International Workshop on: “Liquefaction-Induced Ground Movements Effects”

Berkeley, CA 2-4 November 2016

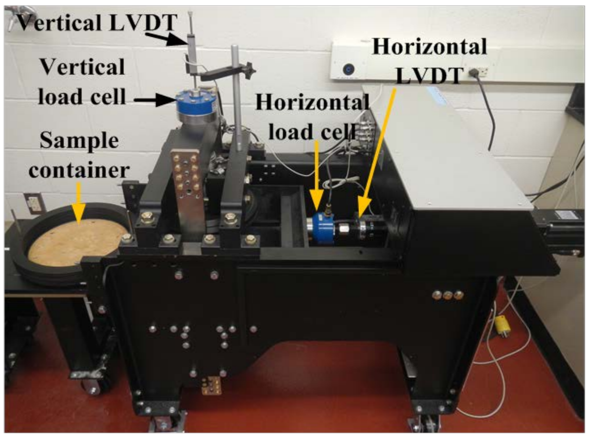
Motivation

- Response of gravelly soils during earthquakes still not fully understood
- Characterizing gravelly soils in a reliable, cost-effective manner is very challenging
- No reported back calculated residual (post-liquefaction) shear strengths from sites with liquefied gravels



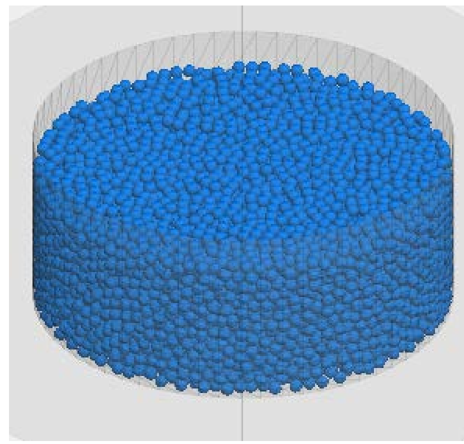
Integrated Approach

Laboratory Testing



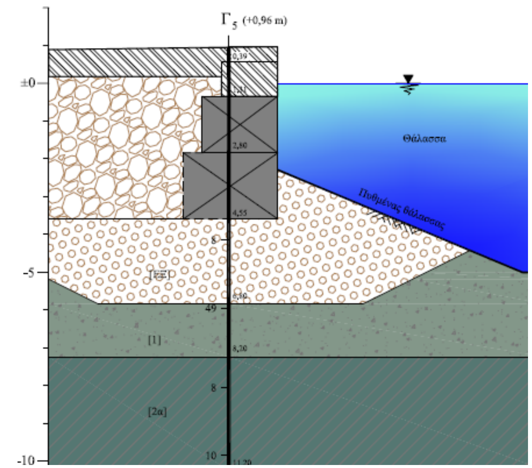
Large-scale CSS used for constant-volume monotonic, cyclic, and post-cyclic shear tests.

Numerical Modeling



3D DEM analyses

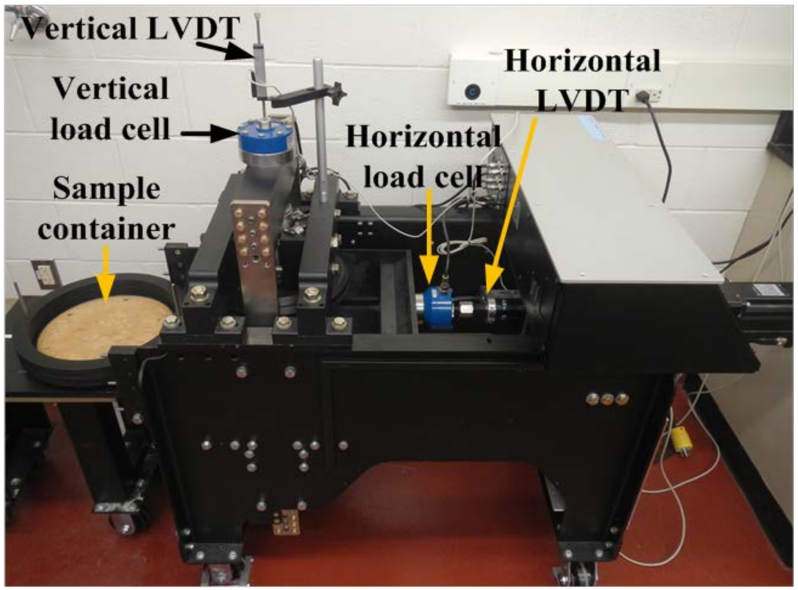
Field Response

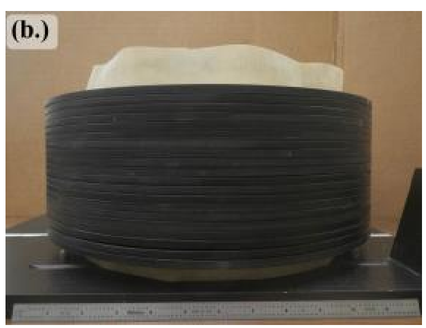


Vs and DPT measurements in the field.

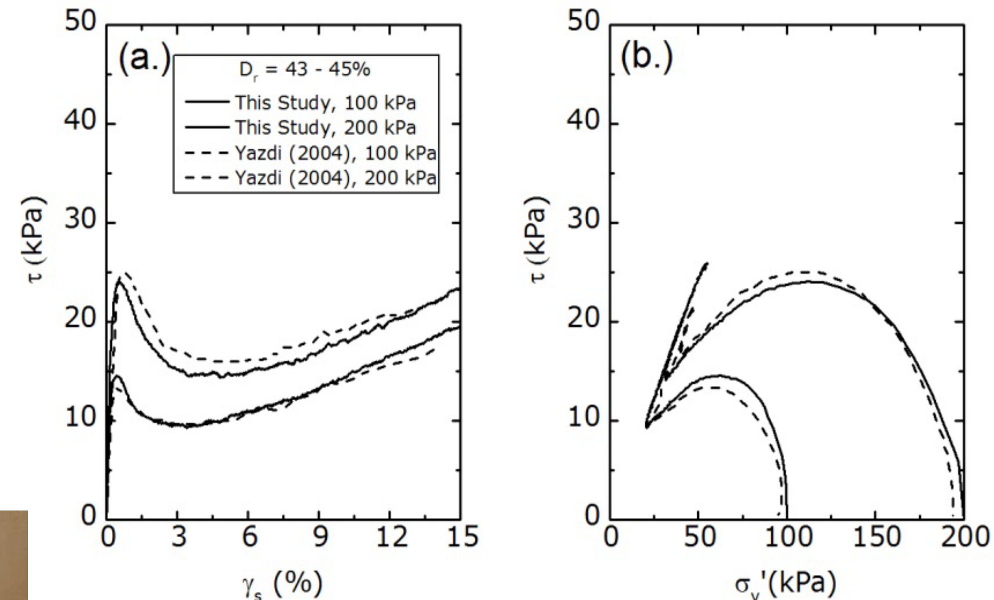
Back-analysis of case histories from the 2014 Cephalonia, Greece earthquake.

Laboratory Testing



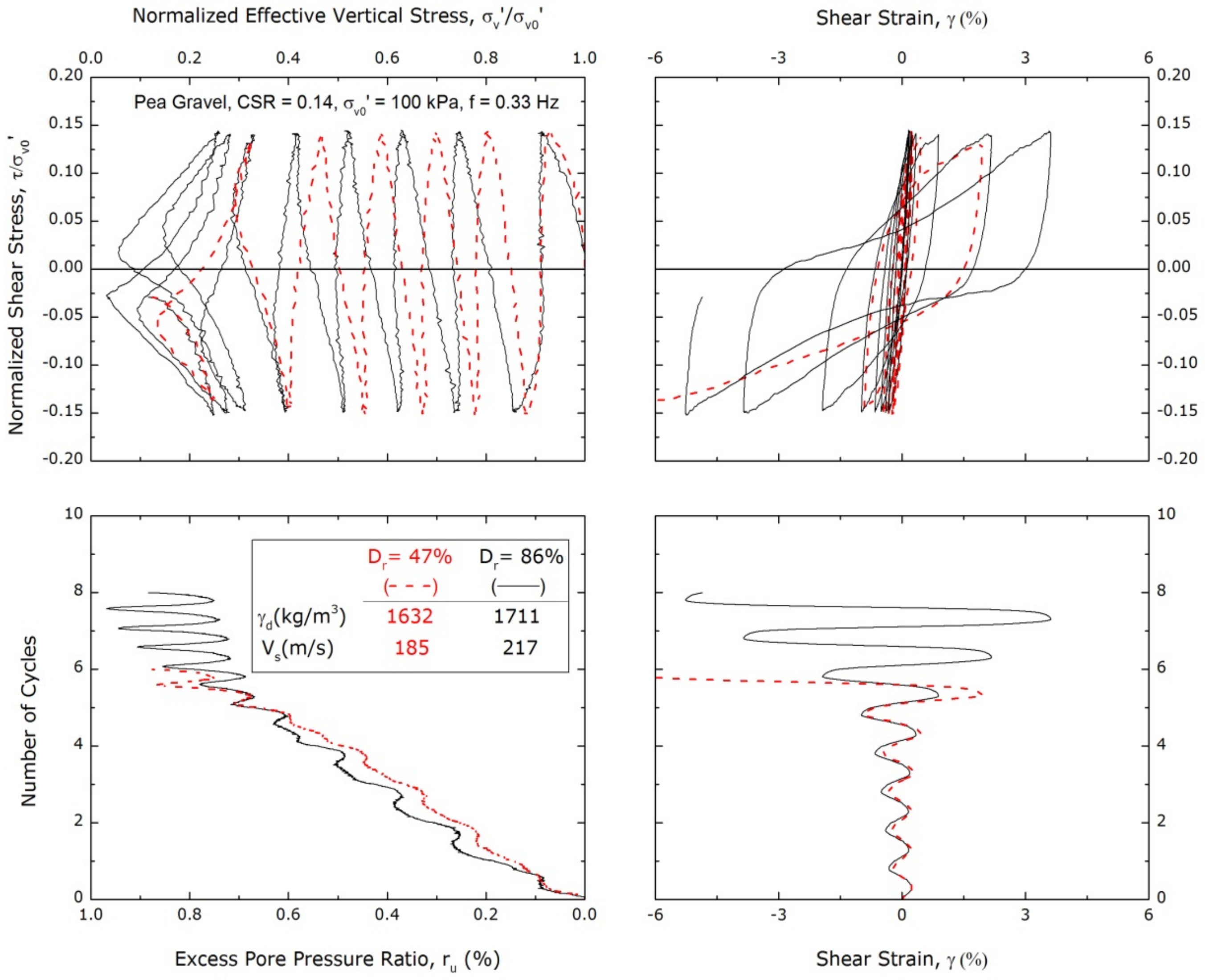


Cyclic Simple Shear: 12'' diameter constant volume/constant-load monotonic/cyclic



Bender Elements and Accelerometers for measuring Vs

Cyclic Simple Shear Test Results - Gravels



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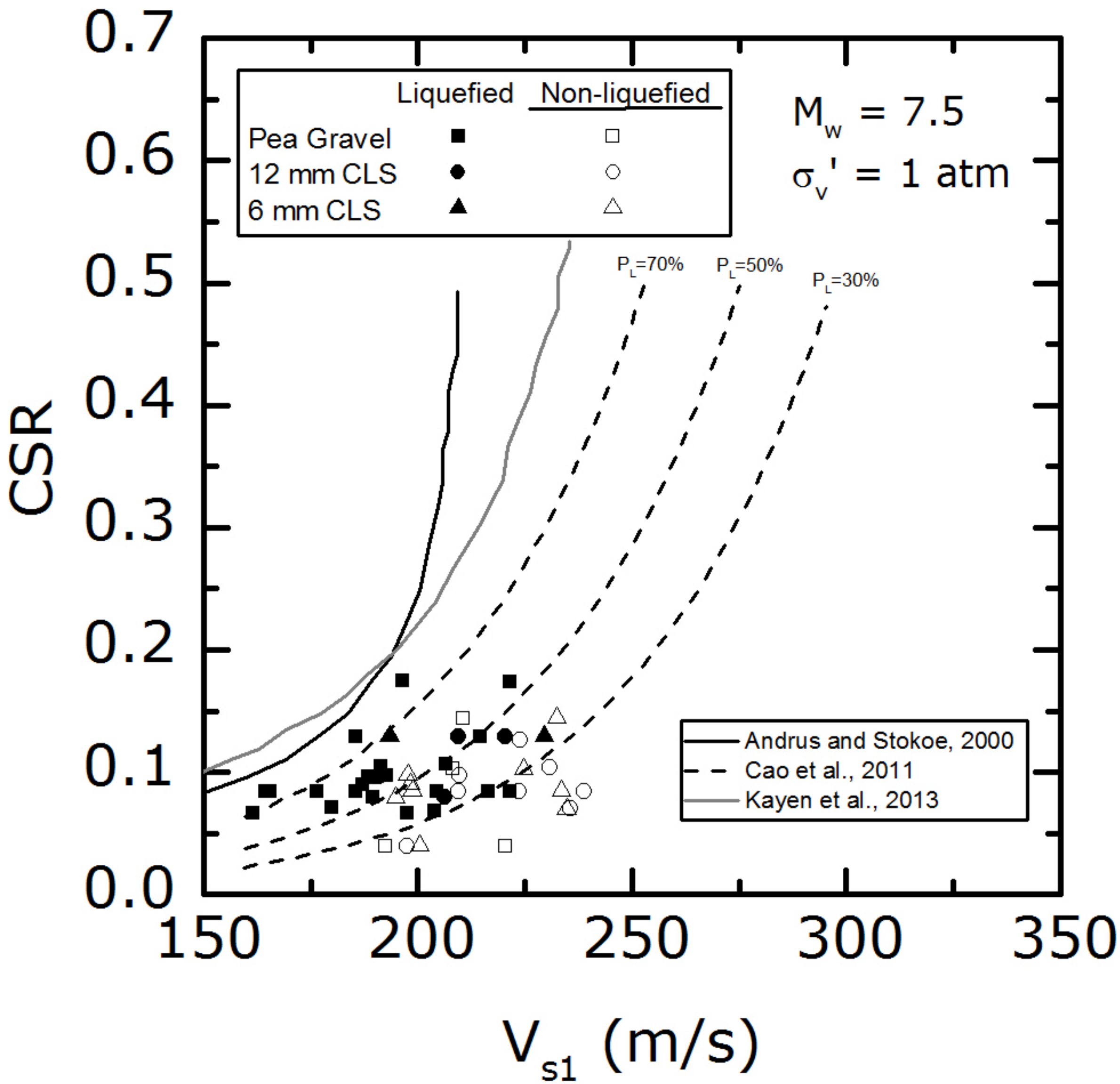
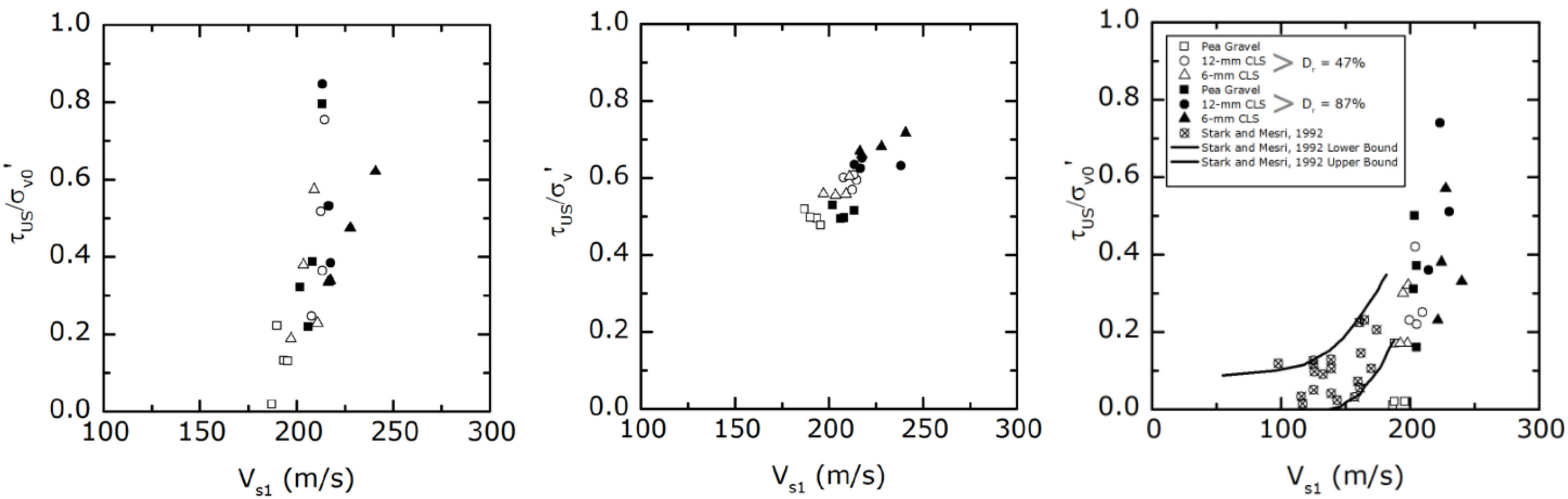
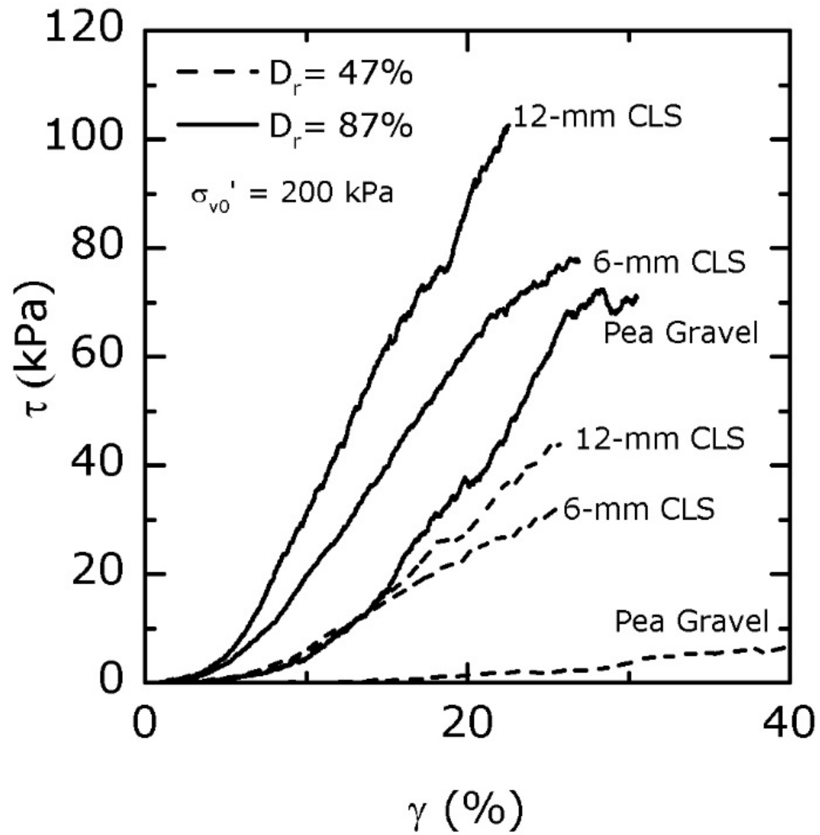
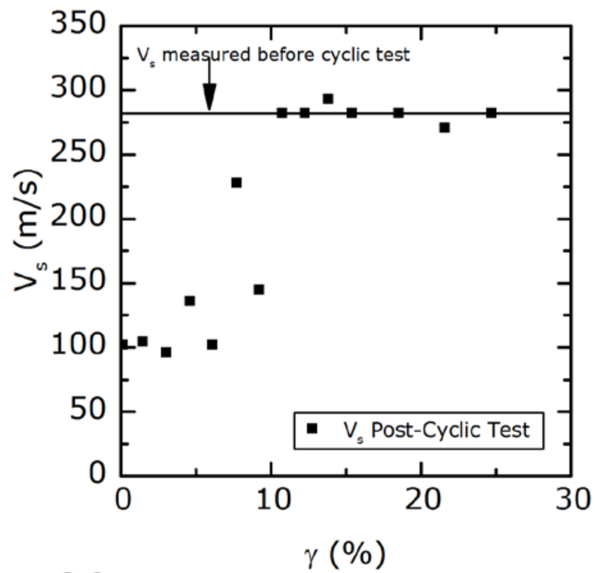


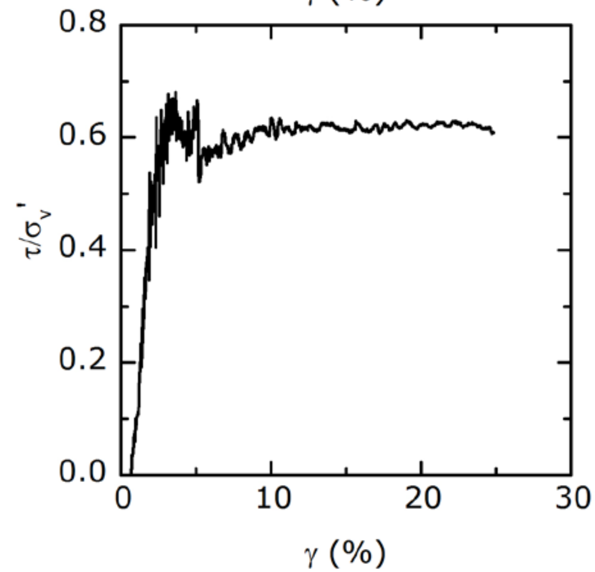
Ultimate State (US) Shear Strength



Post-Cyclic Monotonic Shear Response







<u>Volumetric strains:</u>

All Denser specimens: ~ 1%
Looser Pea Gravel 1.5%
Looser Crushed Limestone 1.0%

Looser Ottawa Sand ~2%

Field Testing and Case-Histories

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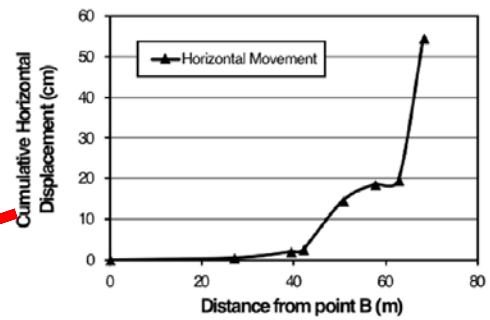
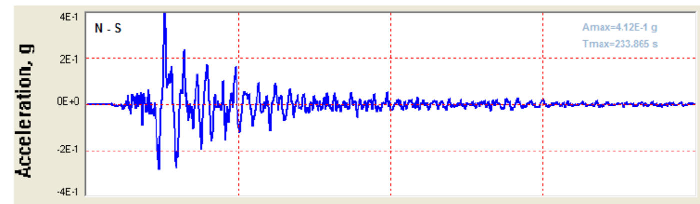
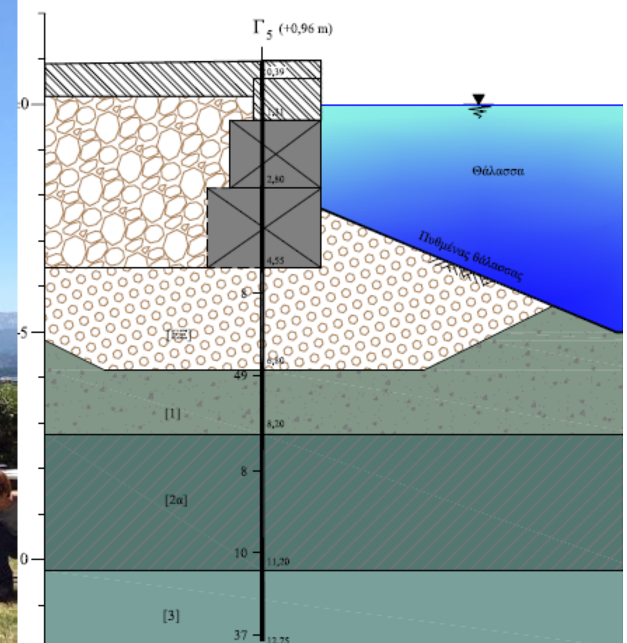
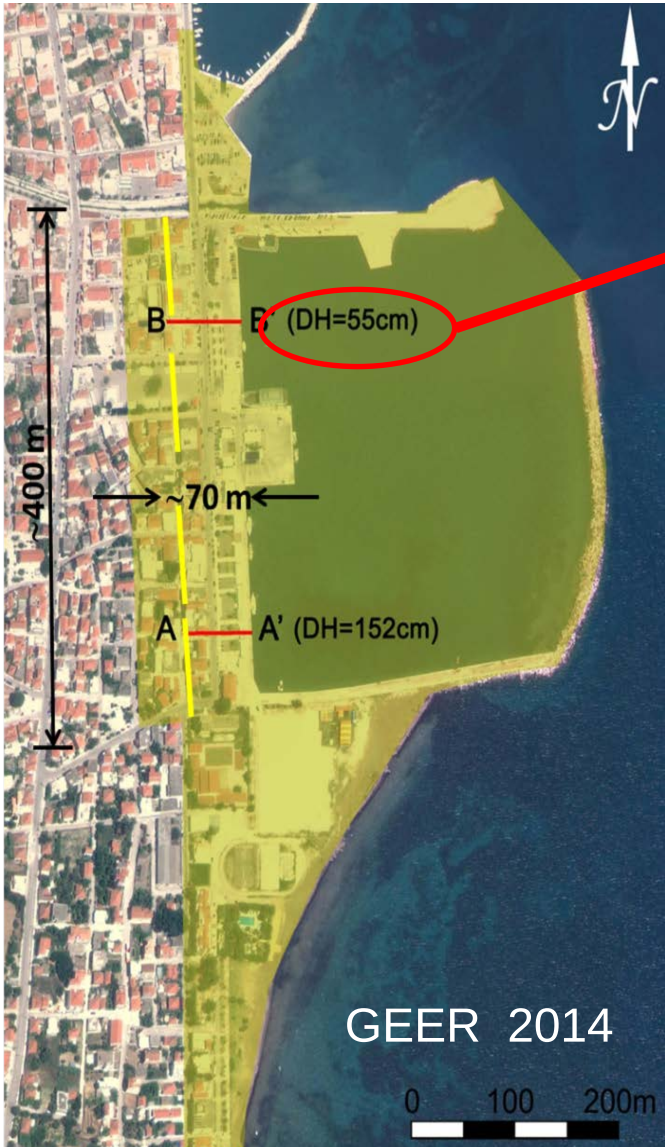


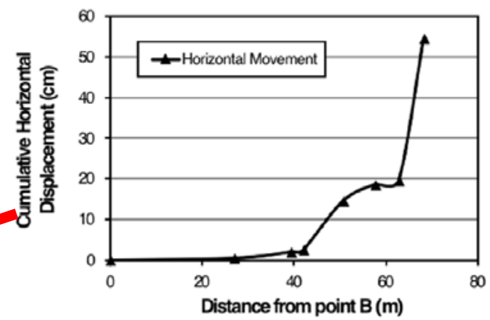


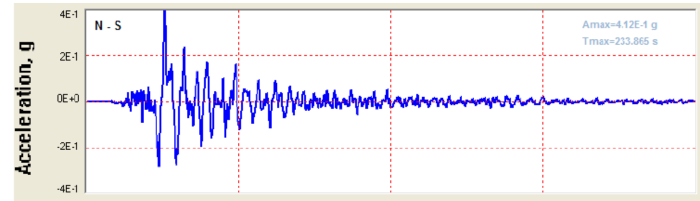


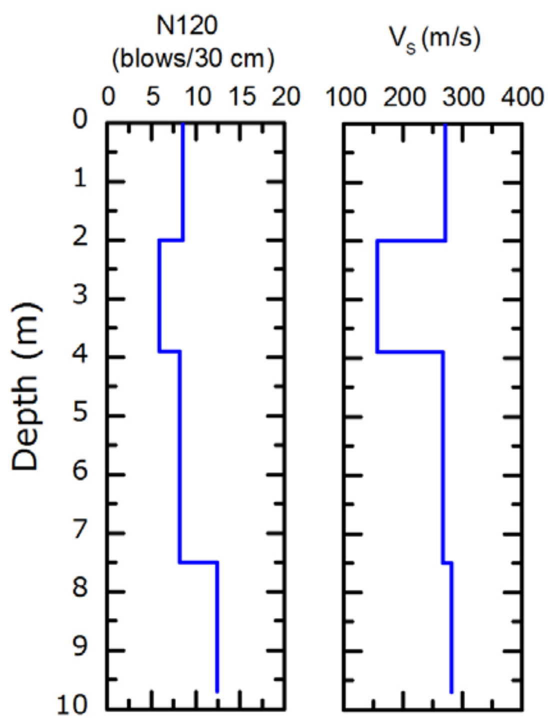
In collaboration with K. Rollins and D. Zekkos

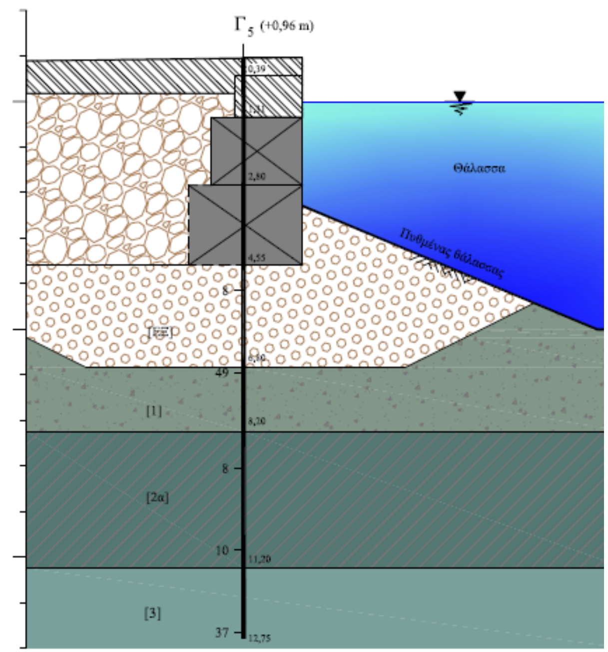
Field Testing and Case-Histories











In collaboration with K. Rollins and D. Zekkos

Thank you!!!

- Jon Hubler, PhD candidate
- Nina Zabihi, PhD candidate
- Prof. Kyle Rollins, BYU
- Prof. Dimitris Zekkos, UM

NSF

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Geocomp
Technologies to manage risk for infrastructure

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(A. Takahashi)

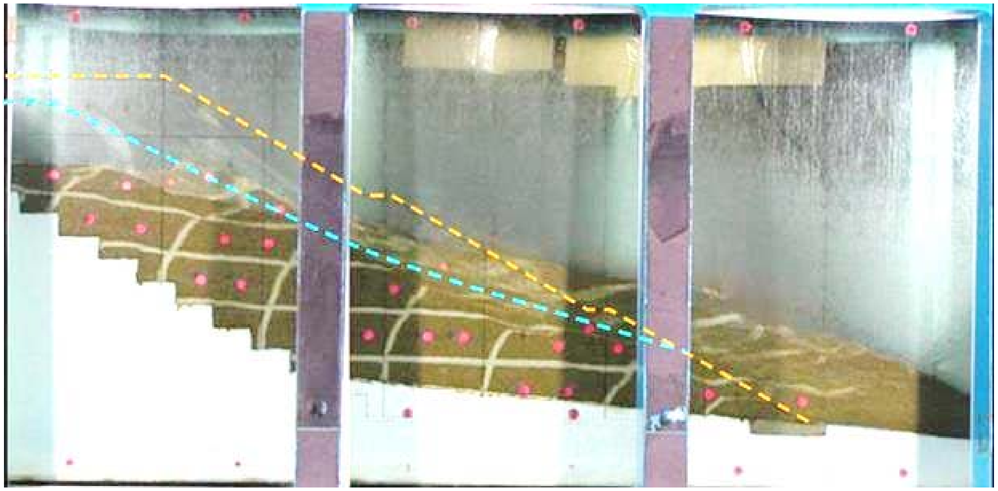
Seismic Performance of Geotechnical Structures in Consideration of Seepage-induced Deterioration

"Key Underlying Geologic Processes" and "Path Forward"

Tokyo Institute of Technology

Akihiro Takahashi

Flow slide of road embankment

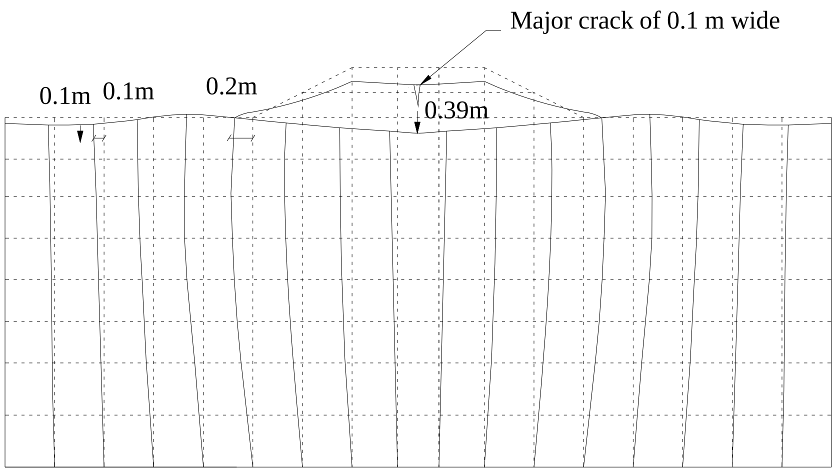


Enomoto and Sasaki (2015)

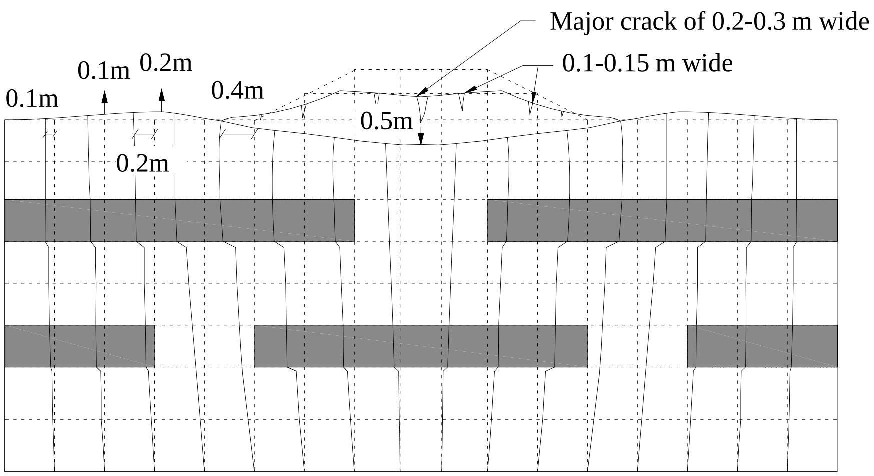
Centrifuge tests mimic the reality?

- Enomoto and Sasaki (2015) tried to mimic the flow slide of the high embankment in the 2007 Noto-Hanto Earthquake using the geotechnical centrifuge.
- They could reproduce the similar deformation pattern using sand with small fines content, which was different from the soil in the actual site.
  - ➢ Suggesting that the cause of flow slide was different from the actual one.

Levee on heterogeneous foundation



Uniform sandy foundation



Sandy foundation with
discontinuous less permeable silt layers

Maharjan and Takahashi (2014)

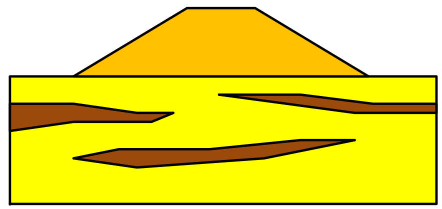
What centrifuge tests demonstrate?

- Maharjan and Takahashi (2014) demonstrated that accumulation of pore water beneath the less permeable layer causes large shear strain there.
- <u>Existence of less permeable layers leads to the larger lateral spreading and excessive settlement in non-homogeneous foundation.</u>
  - ➢ Suggesting that ignorance of thin layers can underestimate the consequence.

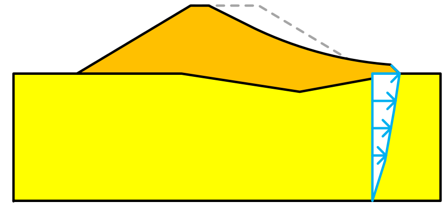
Assumption and reality

Simplification in liquefaction-induced deformation analysis

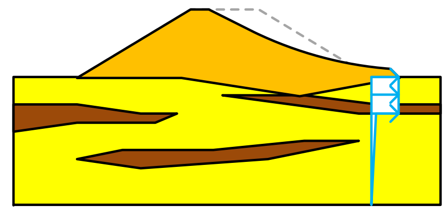
- Simplification in modelling of soil layer(s) can result in misunderstanding of cause of failure mechanism.
  - ➢ Overall response is the same, but actual cause can be different from reality.



Real foundation composition



Analysis with
simplified soil layer(s)



Actual deformation pattern

What is missing?

- Majority of the past liquefaction-induced severe damage of the actual earthwork seems to have occurred due to **<u>localised shear deformation</u>**.
  - ➢ Consideration of weak and/or less permeable layer in the ground.
  - ➢ Such weak zone can be formed due to **<u>deterioration of the soil in the long term</u>**?

How weak zone can be formed?

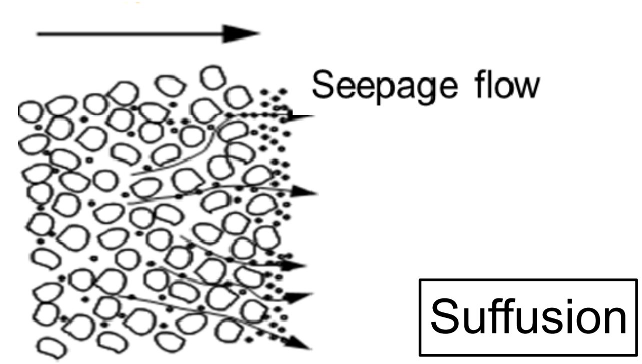
Seepage-induced internal erosion

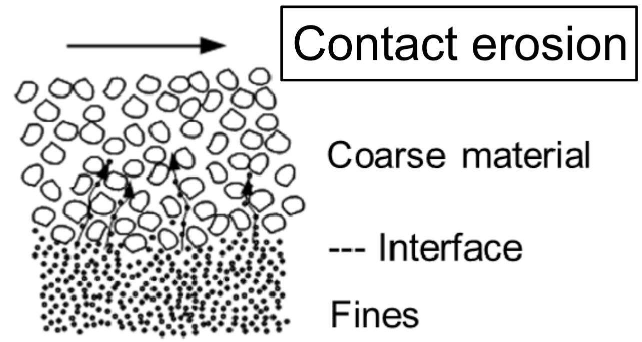
- Suffusion:

  Internal migration of fines in coarser soil
- Contact erosion:

  Migration of fines into coarse soil

➢ Internal erosion increases void size and creates loose state of soil.

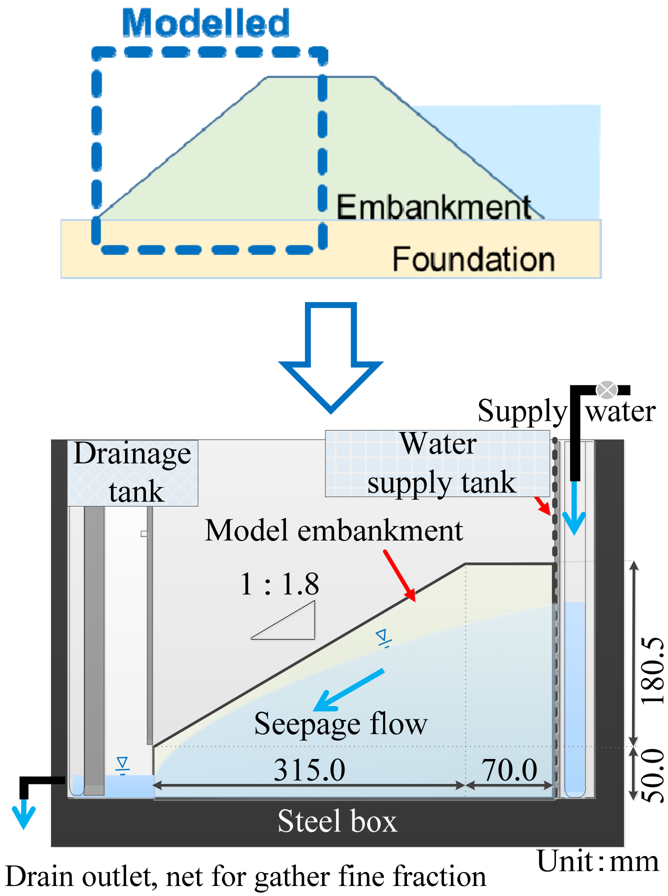


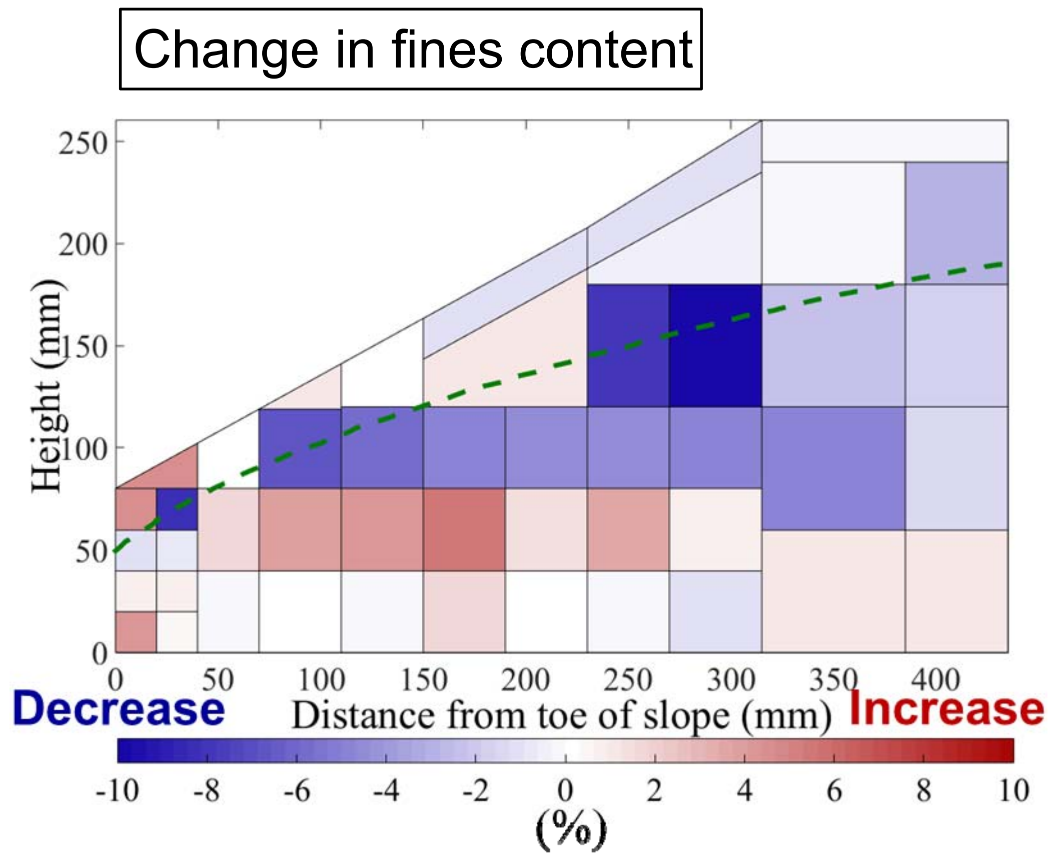


How and where internal erosion occurs?

- Internal erosion process should be examined.
- Horikoshi and Takahashi (2015) examined seepage-induced internal erosion process in an embankment during the phases of initiation and continuation of erosion through a series of physical model tests.

How weak zone can be formed? (cont'd)





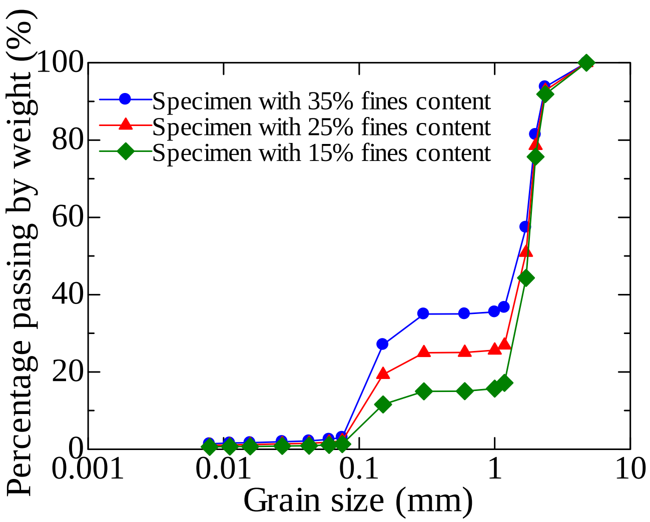
Horikoshi & Takahashi, 2015

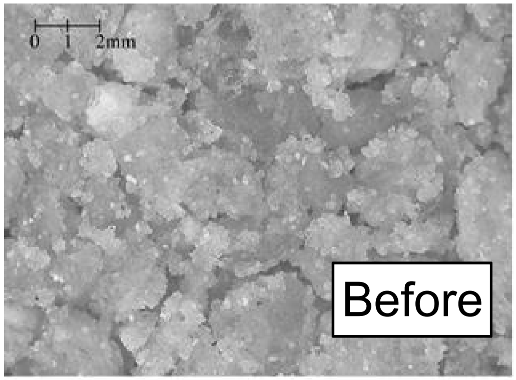
- Loss of fines develops backward along phreatic surface from downstream.
- <u>Internal erosion forms weak zone around phreatic surface</u>.

How internal erosion deteriorates soil?

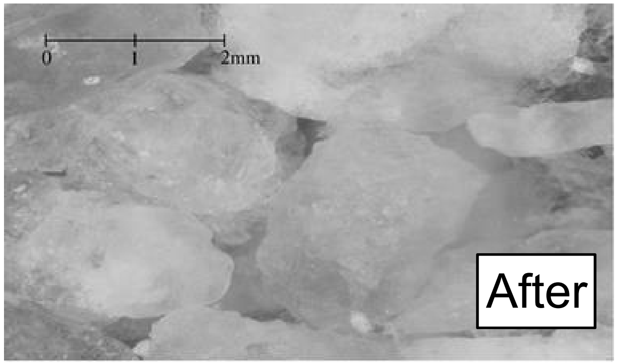
Impact on soil strength / stiffness?

- Mechanical behaviour of the internally eroded soils should be examined.
- Ke and Takahashi (2014) developed a triaxial internal erosion apparatus.
- Responses of the internally eroded soils in the triaxial compression were investigated under both drained condition (Ke & Takahashi, 2015) and undrained condition (Ouyang & Takahashi, 2016).





(c) Before erosion (25% initial fines content)

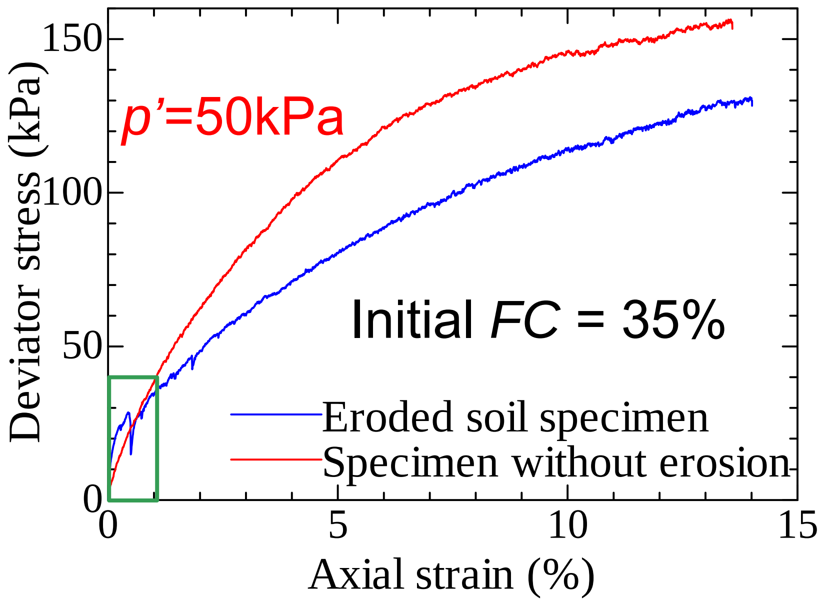


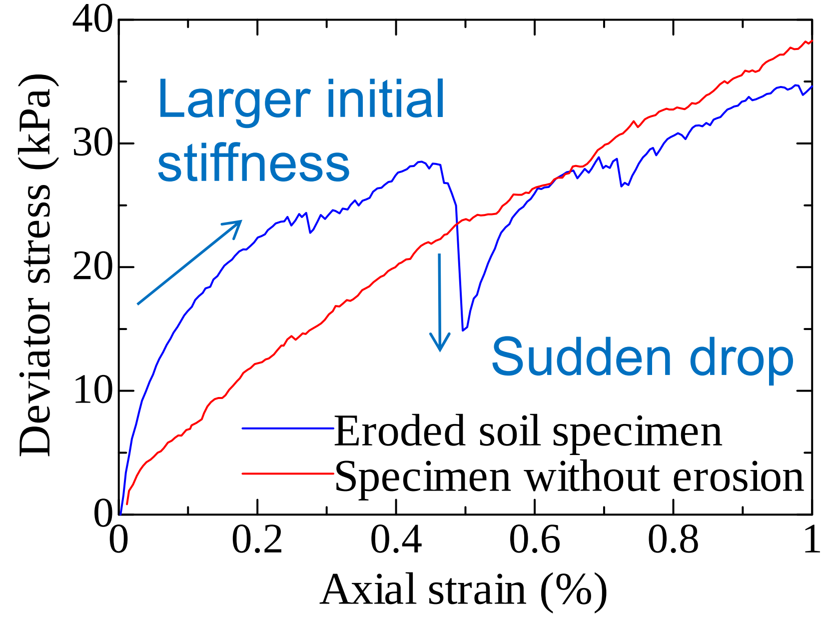
(b) After erosion (25% initial fines content)

Ouyang & Takahashi, 2016

- For the start, tests are conducted on gap-graded soils (initial *FC* = 25%).

How internal erosion deteriorates soil? (cont'd)





Ke and Takahashi (2015)

- Drained strength of eroded soil is obviously smaller than that without erosion.
- In the seepage (internal erosion) stage, fines got impeded and accumulated at the contacting points of coarse particles, which might form local reinforcement.
  - ➢ Stiffness of the eroded soil at the beginning of shearing is large.
  - ➢ Due to deterioration of the reinforcement with the progress of shearing, strength of the eroded soil becomes smaller than that without erosion.

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(Bruce Kutter)

“RESIDUAL SHEAR STRENGTH” CANNOT BE UNIQUELY CORRELATED TO PENETRATION RESISTANCE

- AND WE SHOULD STOP USING THE IDEA

Bruce L. Kutter
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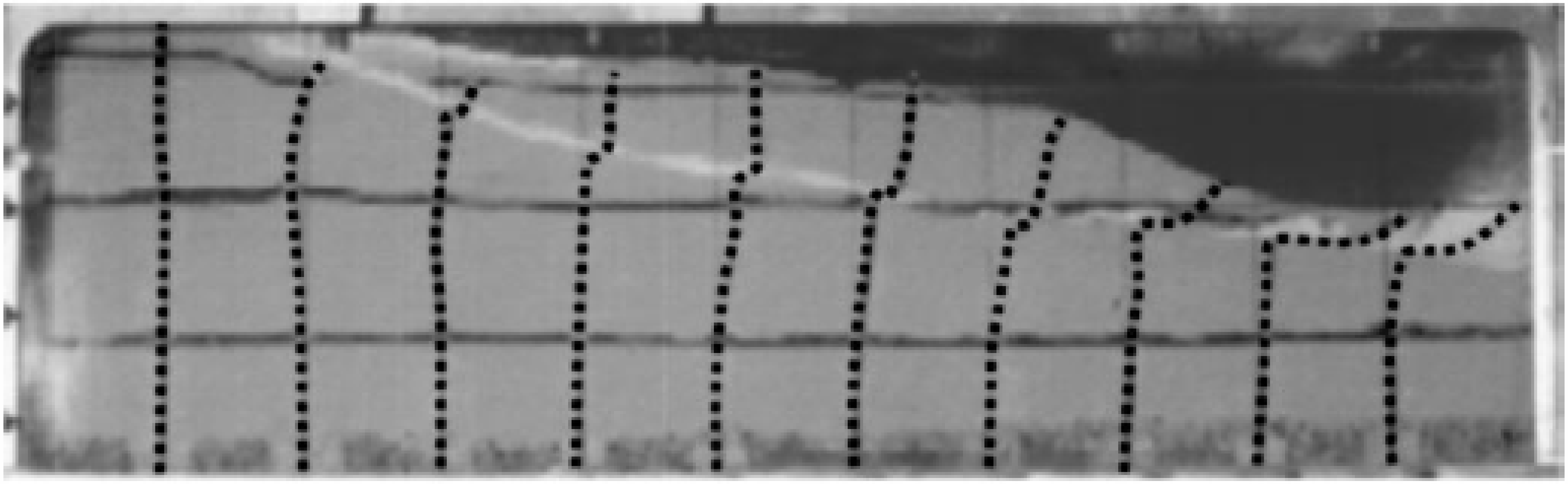
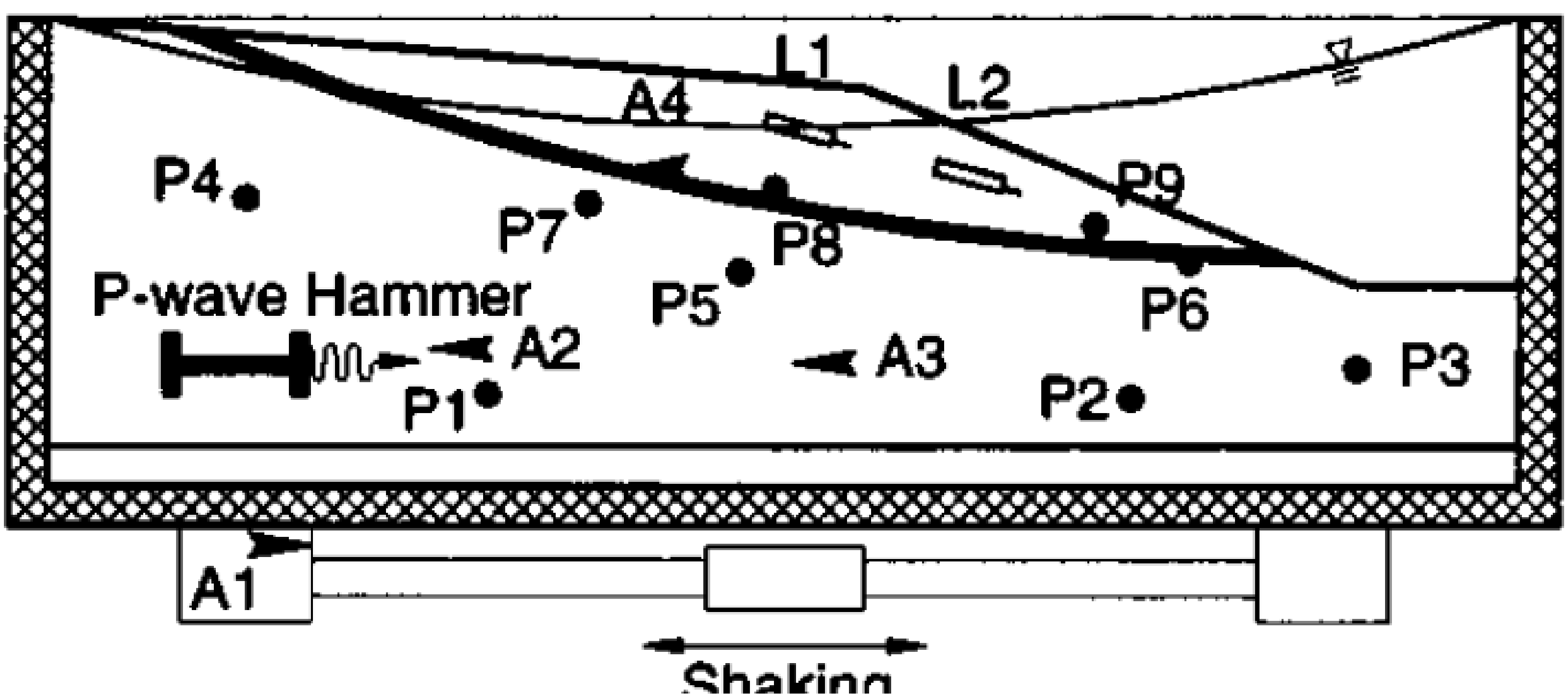
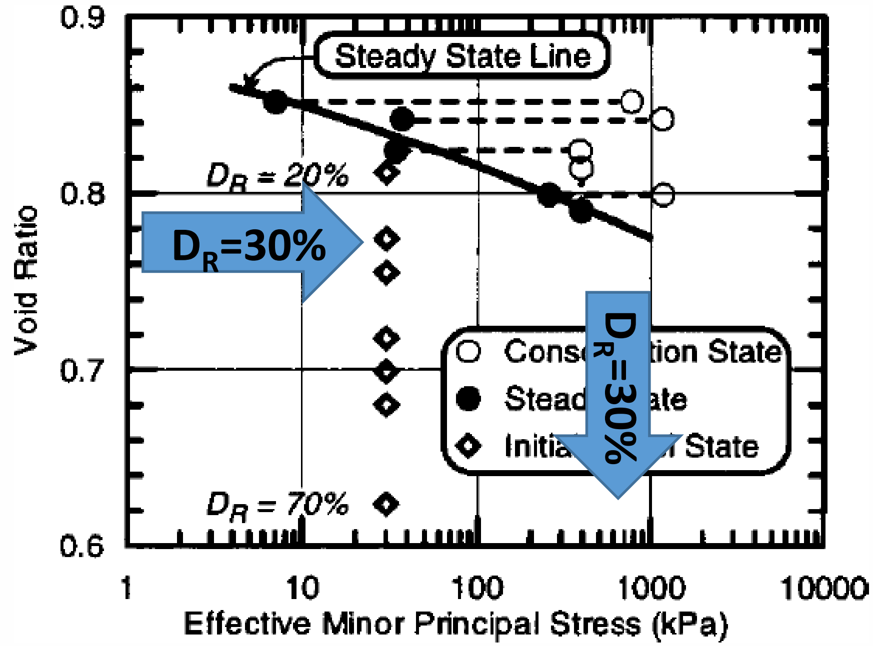
Mechanism of void redistribution (Kulasingam et al. 2004)

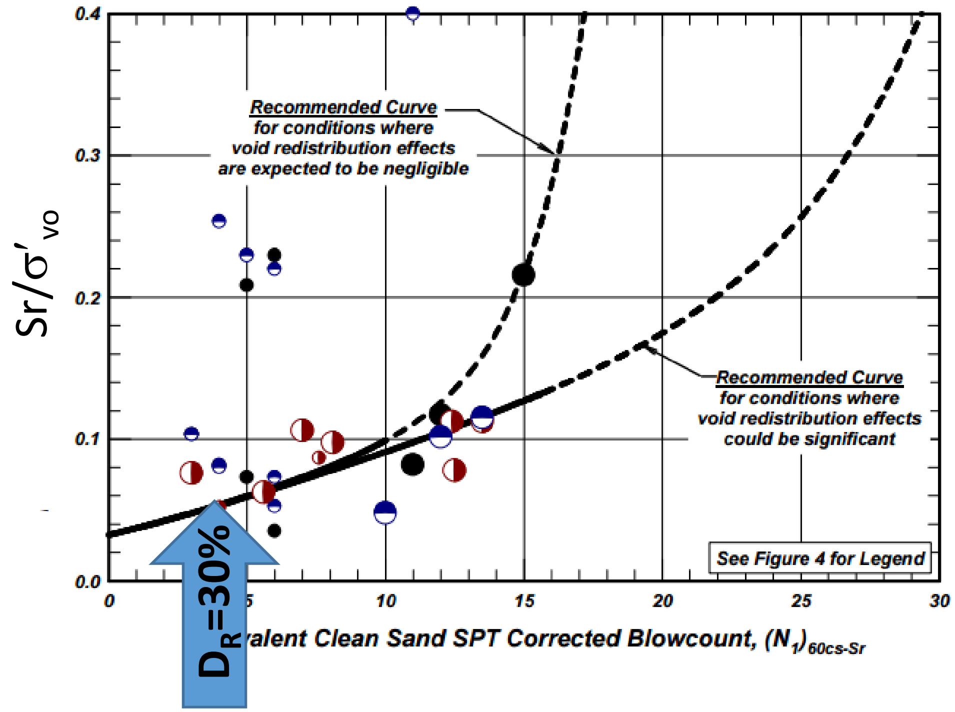




Kulasingam et al (2004)



Idriss and Boulanger (2007).



Dr = 30% → Sr/σ′$_{vo}$ ~ 0.05,

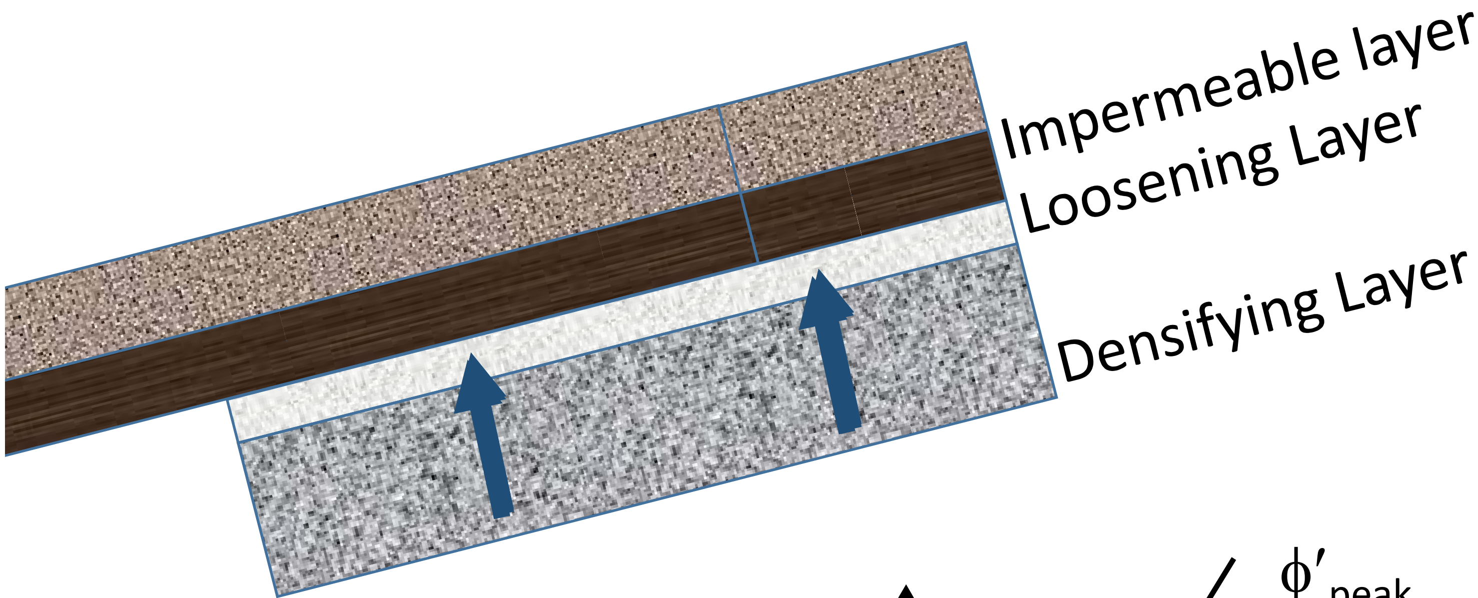
σ′$_{vo}$ = 100 kPa → Sr = 0.05*100 = 5 kPa

P’ at Steady State Line is ~ 500kPa

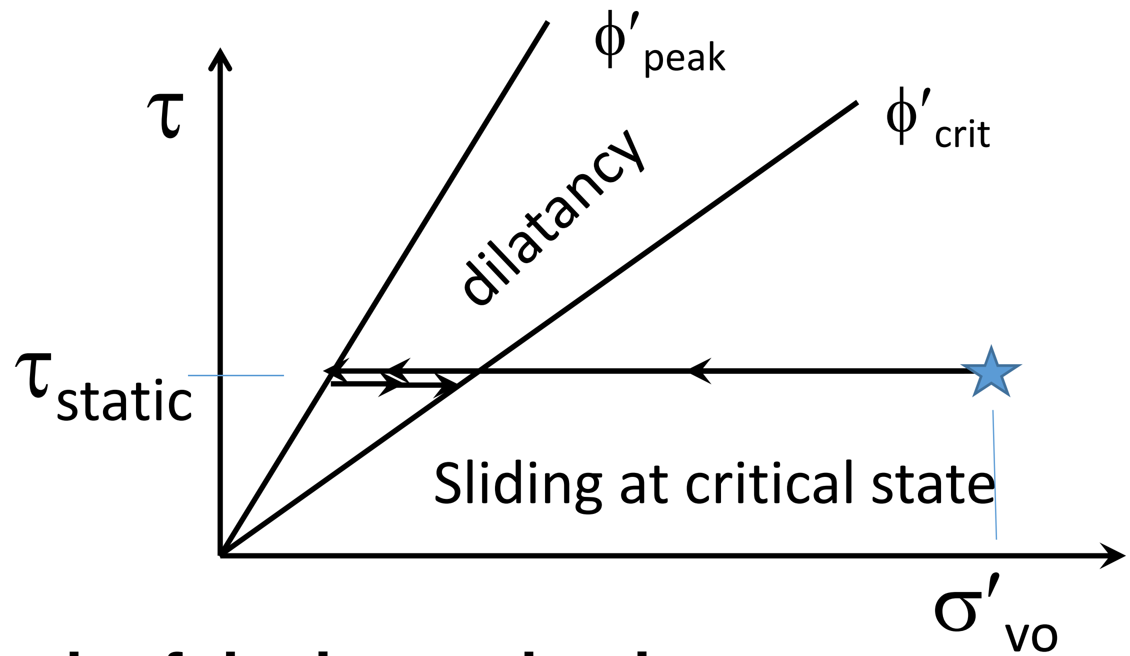
Q: Why is residual strength so much lower than steady/critical state strength?

A: Void redistribution (loosening), particle mixing.

Shaking causes pore pressure, then water flows upward



More shaking causes more softening. Failure occurs when dilatancy is exhausted.



Stress path of the loosening layer

IT IS WRONG TO USE RESIDUAL STRENGTH FOR DESIGN OF CRITICAL INFRASTRUCTURE

- It is wrong to deduce the fully softened residual strength by back-analysis of a failure.
  - Void redistribution may cause continual loss of shear resistance.
  - Failure occurs when the critical state sliding resistance drops to the sliding force; not when the sliding resistance is a minimum.
  - Flow failure, in general, will occur before the soil is fully softened.
- The term “undrained residual strength” should not be applied to flow failures
  - there is no justification to the assumption that material in a flow failure is undrained.
  - Sliding resistance at failure is a “system parameter”, not a “material strength”

PATH FORWARD

Instead of a strength-based assessment of stability, the approach to the critical state may be figured out by calculating how much water is being expelled by the zones of densification, and how much of this water contributes to loosening of the failure mechanism. We need:

- Realistic constitutive models
- Solution schemes that can predict strain softening, localization of shear strains, and large deformations,
- Multi-physics modeling capabilities are needed to predict void redistribution
  - Water escape through cracks and boils
  - Water accumulation in shear zones
  - Stochastic models of stratigraphy and Monte Carlo simulation

Continued reliance on erroneous residual strength delays true progress!

LEAP – Liquefaction Experiments and Analysis Projects

- International effort to evaluate the accuracy of existing models and calibration procedures used for simulation of the effects of liquefaction.
  - LEAP-UCD-2017 (tentatively December 2017) will involve a sufficient number of experiments performed on a variety of centrifuge facilities to demonstrate the uncertainty and median response of a liquefiable layer and to allow us to evaluate the sensitivity of the response to key input parameters.
  - Comparisons between numerical models and a group of centrifuge model tests.

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(Les Harder)

U.S. – N.Z. – Japan International Workshop on
"*Liquefaction–Induced Ground Movements Effects*,"
Berkeley, CA 2-4 November 2016

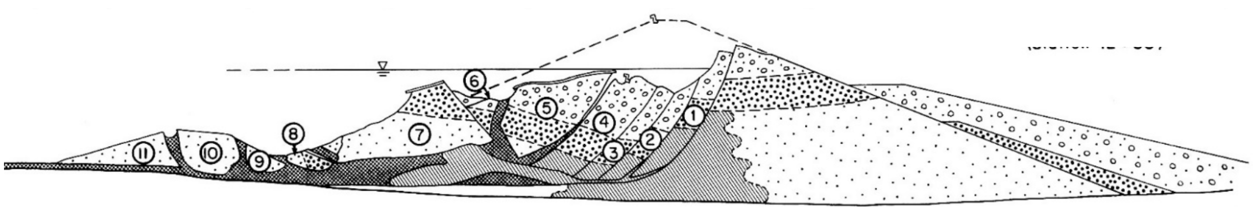
Development and effects of liquefaction-induced flow slides that are governed by the undrained residual shear strength of liquefied soil

Absence of Residual Shear Strength Case Histories for Medium Dense Soils

Presented by:

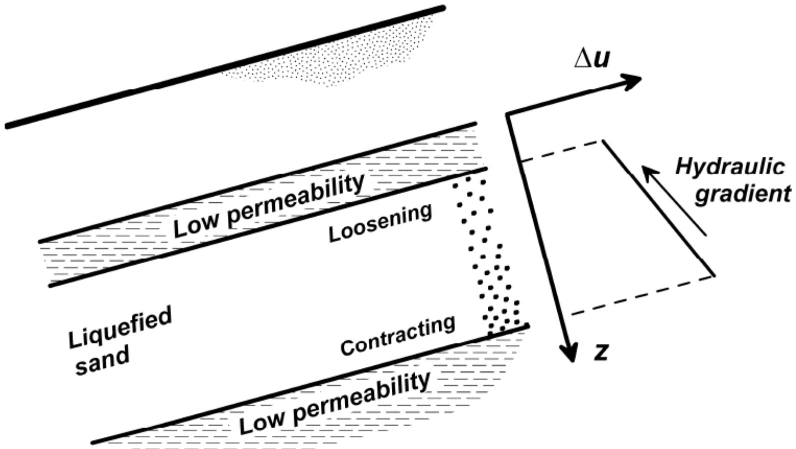
Leslie F. Harder, Jr.

HDR Engineering Inc.

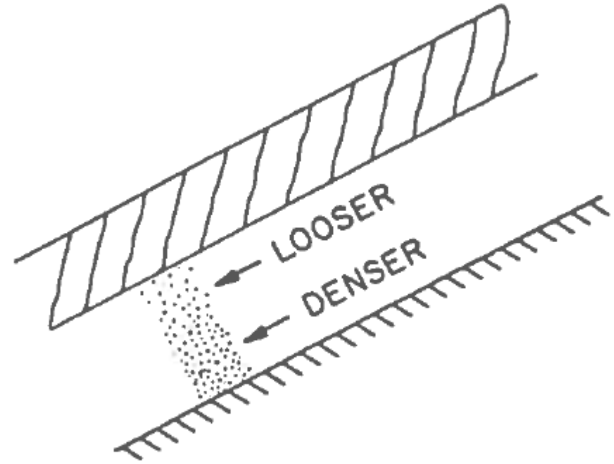


HDR

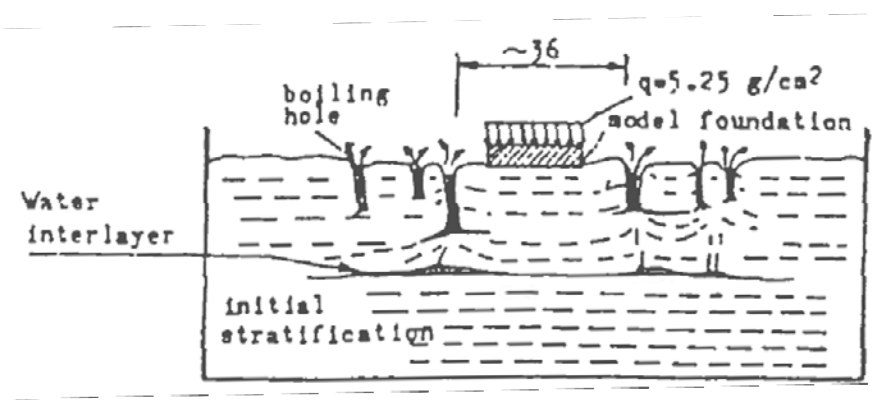
Looking Back: Recognition of Potential for Void Ratio Redistribution



from Whitman (1985)



from Seed (1987)



Results of Shaking Table Tests on Deposit of Stratified Sand (after Liu and Qiao, 1984; as discussed by Seed, 1987)

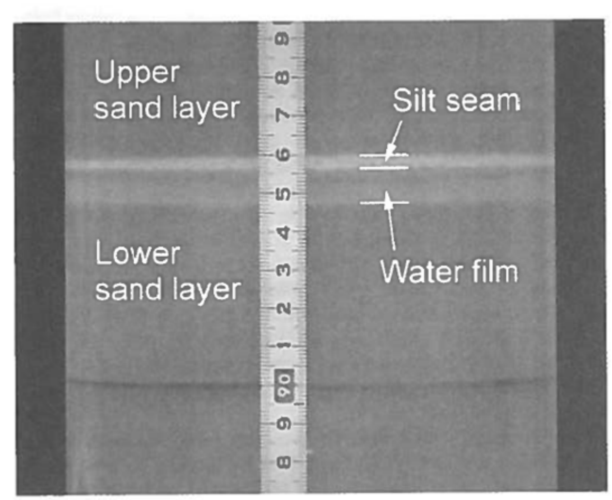
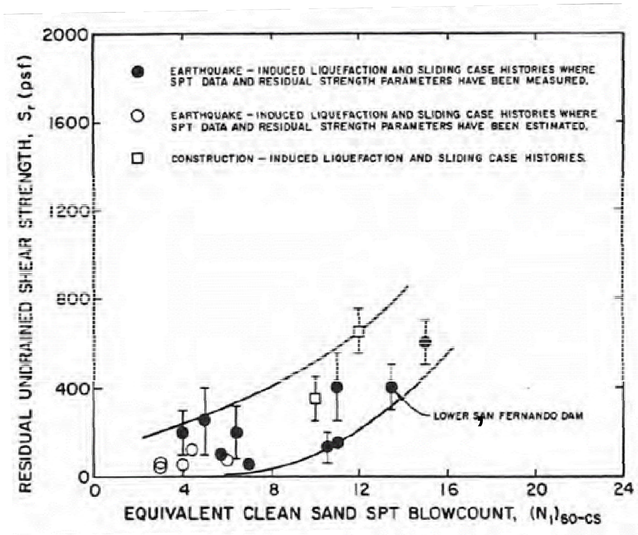


FIG. 5. Photograph of Water Film Consisting of Clear Water Formed beneath Silt Seam

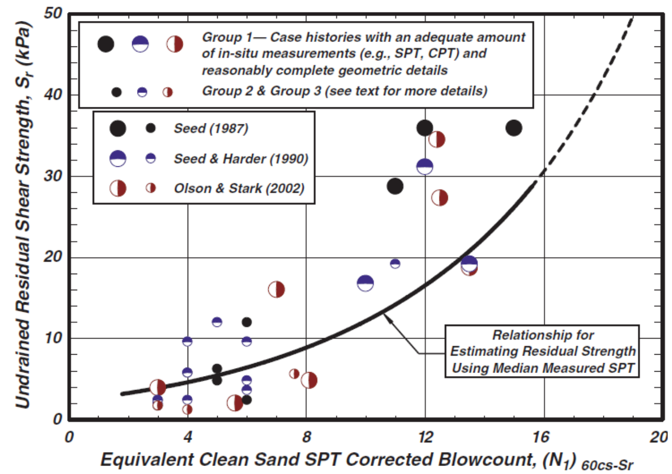
from Kokusho (1999)

Current State-of-the-Art: Back-Calculation of Residual Shear Strengths from Case Histories

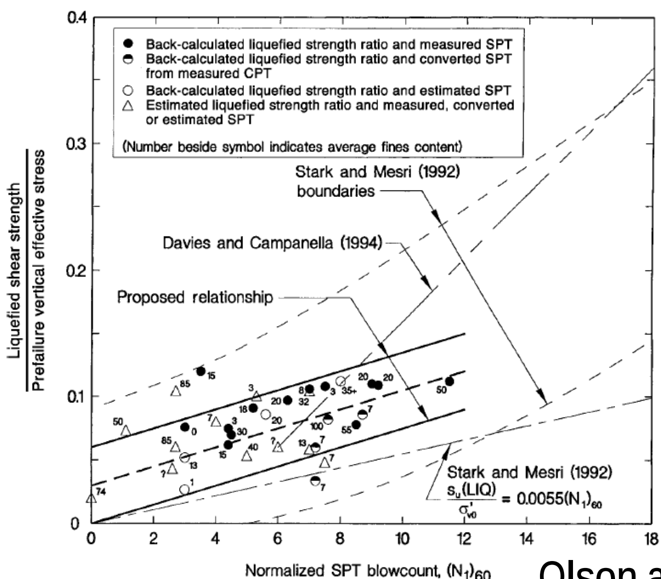
SPT Correlations of S_r or Normalized Values of S_r / σ_{vo} in Current Use



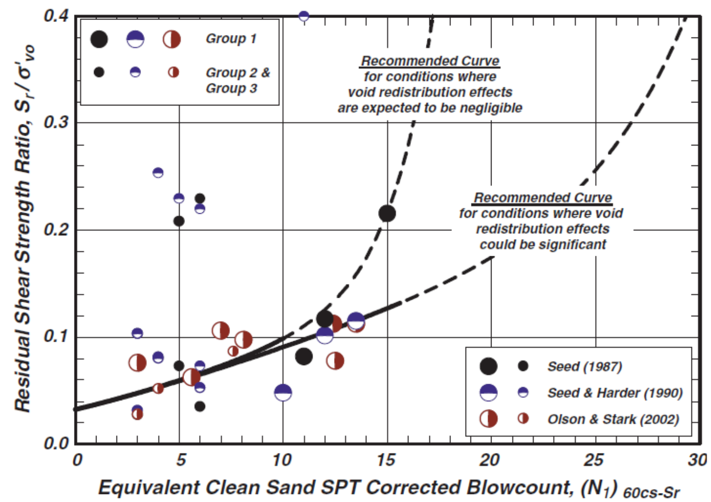
Seed and Harder (1990)



Idriss and Boulanger (2007)



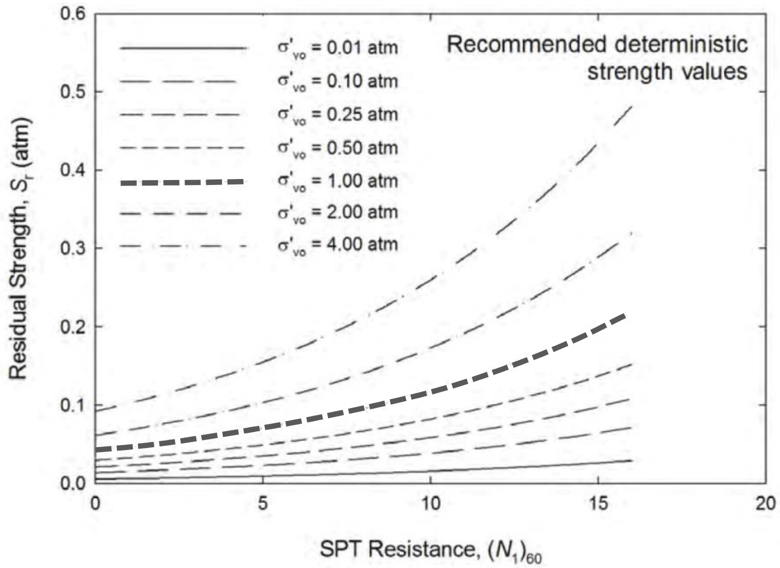
Olson and Stark (2002)



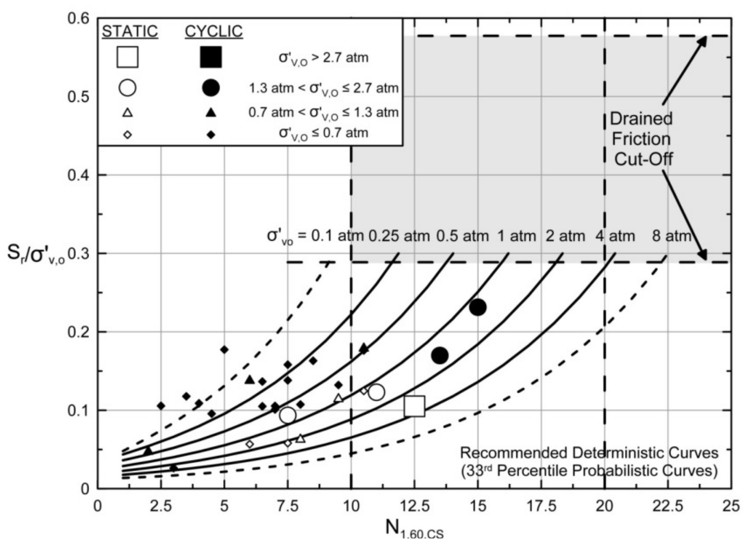
Idriss and Boulanger (2007)

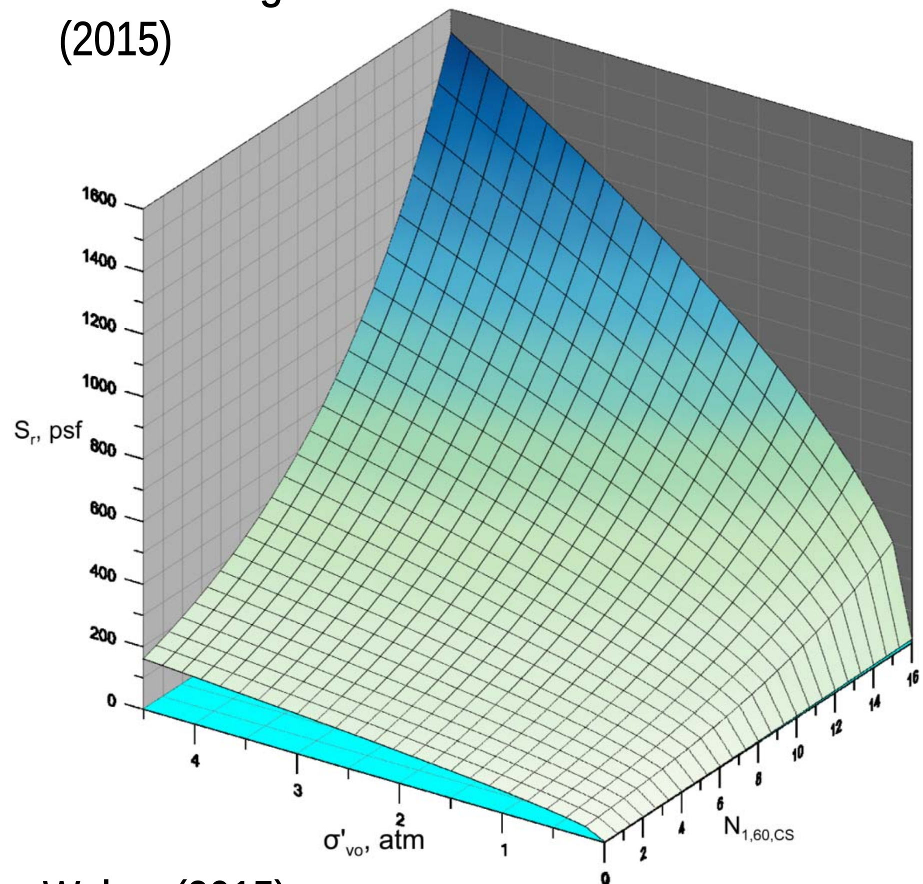
Current State-of-the-Art: Back-Calculation of Residual Shear Strengths from Case Histories

Hybrid Correlations between SPT Blowcount, S_r, and σ_{vo}'



Kramer and Wang (2015)





Weber (2015)

Challenges: Limited Number of Case Histories

Group	Case	Seed and Harder (1990)		Olson and Stark (2002)				Wang (2003) + Kramer (2008)				This Study			
		S_r (psf)	$N_{1,60,CS}$	S_u(Liq) (psf) (1)	$S_u(Liq)/\sigma'_{vo}$	σ'_{vo} (psf)	$N_{1,60}$ (2)	$\bar{S}_r$ (psf) (3)	$\bar{S}_r/\bar{\sigma}'_{vo}$	$\bar{\sigma}'_{vo}$ (psf) (4)	$\bar{N}_{1,60,CS}$	$\bar{S}_r$ (psf)	$\bar{S}_r/\bar{\sigma}'_{vo}$	$\bar{\sigma}'_{vo}$ (psf)	$\bar{N}_{1,60,CS}$
	Wachusett Dam - North Dike			334	0.106	3158	7	348	0.136	2559	7.3	294	0.094	3142	7.5
	Fort Peck Dam	350	10	570	0.078	7341	8.5	671.6	0.091	7380	15.8	762	0.105	7258	12.5
	Uetsu Railway Embankment	40	3	36	0.027	1280	3	43.7	0.048	910	2.9	38	0.026	1448	3
	Lower San Fernando Dam - U/S Slope	400	13.5	390	0.120	3482	11.5	484.7	0.133	3644	14.5	539	0.170	3174	13.5
	Hachiro-Gata Road Embankment			42	0.062	670	4.4	65	0.164	396	5.7	68	0.101	673	7
	La Marquesa Dam - U/S Slope	200	6	[104]	0.114	911	4.5	(185.1)	0.110	1683	6.5	103	0.105	981	6.5
A	La Marquesa Dam - D/S Slope	400	11	[152]	0.152	1000	9	(343.5)	0.186	1847	9.9	214	0.176	1215	10.5
	La Palma Dam	200	4	[125]	0.158	789	3.5	(193.3)	0.123	1572	4.2	136	0.177	767	5
	Lake Ackerman Highway Embankment			82	0.076	1076	3	98	0.114	860	4.8	107	0.118	909	3.5
	Chonan Middle School			[142]	0.127	1119	5.2	(178.7)	0.091	1964	6.4	141	0.137	1032	6.5
	Soviet Tajik - May 1 Slide			[334]	0.154	2170	7.6	(334.3)	0.082	4077	8.9	341	0.179	1907	10.5
	Shibecha-Cho Embankement			117	0.086	1351	5.6	208.9	0.200	1045	5.6	224	0.158	1416	7.5
	Route 272 at Higashiarekinai			100	0.097	1030	6.3	130.5	0.125	1044	8.5	138	0.107	1285	8
	Zeeland - Vlietepolder			[180]	0.075	2396	7.5	(226.0)	0.048	4708	8.5	156	0.063	2488	8
	Sheffield Dam	75	6	[159]	0.111	1429	5	(100.0)	0.072	1389	8.2	138	0.106	1308	7
	Helsinki Harbor			[44]	0.084	522	6	(53.2)	0.060	887	5.9	48	0.057	846	6
	Solfatara Canal Dike	50	4	[71]	0.114	624	4	(77.1)	0.063	1224	4.9	64	0.096	669	5
	Lake Merced Bank	100	6	[205]	0.149	1372	7.5	(139.5)	0.106	1316	5.9	136	0.163	834	8.5
	El Cobre Tailings Dam			<40>	0.020	1946	0	(195.2)	0.020	9760	6.8	95	0.046	2075	2
	Metoki Road Embankment			[90]	0.103	875	2.6	(116.8)	0.044	2655	2	92	0.106	871	2.5
	Hokkaido Tailings Dam			[138]	0.100	1376	1.1	(250.6)	0.074	3386	5.1	131	0.109	1203	4
	Upper San Fernando Dam - D/S Slope	600	15									726	0.231	3138	15
B	Tar Island Dyke			[401]	0.093	4300	7	(364.2)	0.058	6279	8.9	516	0.123	4197	11
	Mochi-Koshi Tailings Dam, Dikes 1 and 2	250	5	[207]	0.165	1251	2.7	(158.9)	0.091	1746	8.9	211	0.138	1532	6
				[180]	0.165	1090	2.7	(233.6)	0.081	2884	10				
	Nerlerk Embankment, Slides 1 ,2 and 3			[44]	0.071	616	8.7	(178.5)	0.1239	1440	11.4	68	0.058	1171	7.5
				[50]	0.077	650	7.2								
				[52]	0.056	925	7.2								
	Asele Road Embankment			[192]	0.153	1251	7	(163.6)	0.104	1573	11	137	0.132	1037	9.5
	Nalband Railway Embankment			[121]	0.110	1101	9.2	(139.9)	0.109	1283	6.3	167	0.138	1209	7.5
	Sullivan Tailings											277	0.114	2422	9.5
	Jamuna Bridge											175	0.125	1404	10.5
C	Calaveras Dam	650	12	721	0.112	6422	8	636.9	0.099	6433	10.5	749	0.106	7097	15

Notes : (1) Where noted in brackets, S_u(Liq) and $S_u(Liq)/\sigma'_{vo}$ for Olson (2001) reinterpreted using reported values of Su Yield and S_u Residual in Olson (2001) and the equation $S_u(Liq) = 0.8$ (S_u Yield + S_u Residual)/2. Reinterpretation of S_u(Liq) performed for cases not calculated using the Kinetic procedure in Olson (2001). Where noted in triangular brackets, no S_u Yield value reported in Olson (2001).

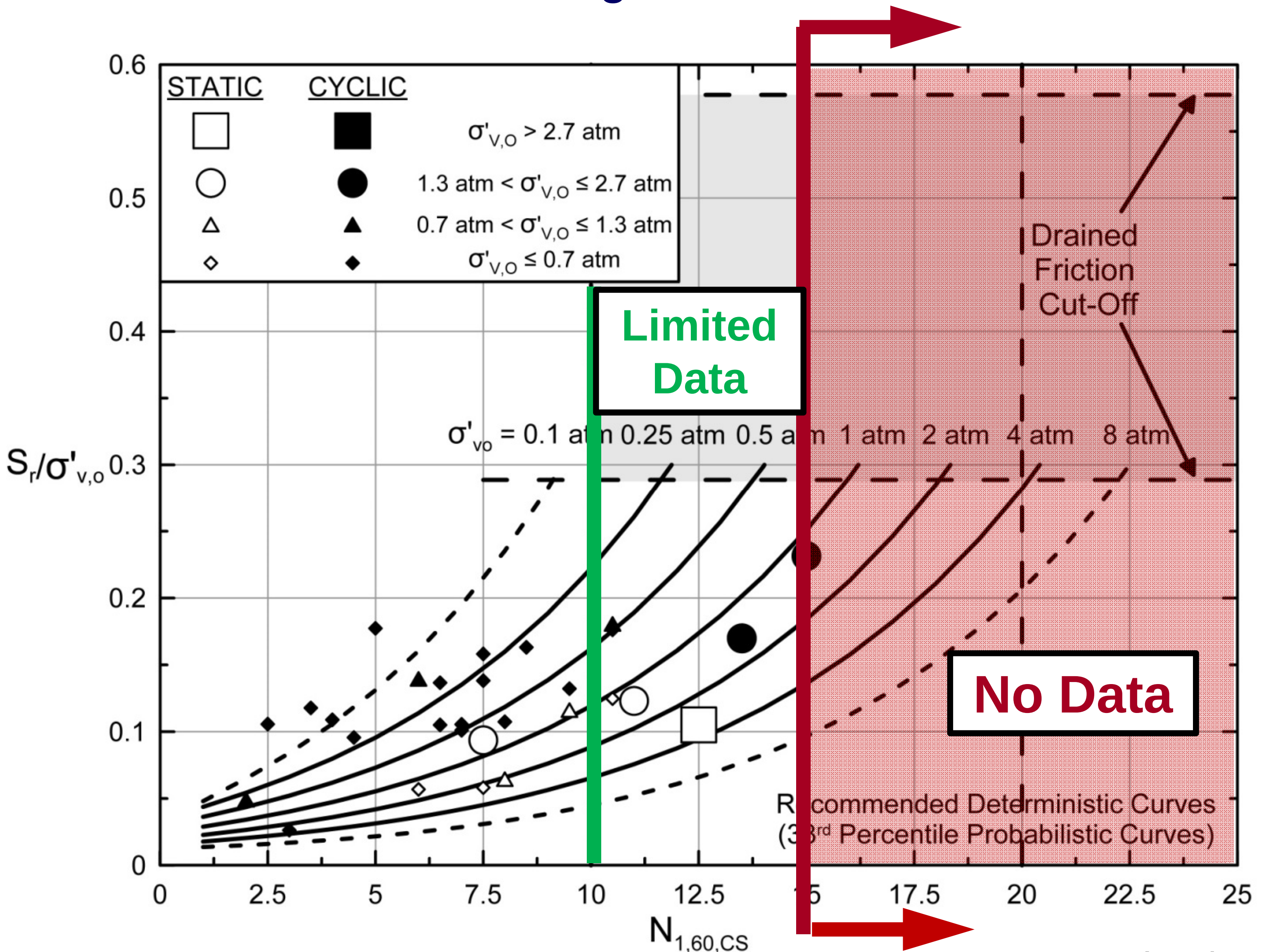
(2) No fines content correction utilized in Olson and Stark (2002).

(3) Where noted in parentheses, $S_{u,r}$ values are for secondary cases in Wang (2003) and were not fully reanalyzed.

(4) σ'_{vo} not explicitly reported in Wang (2003) or Kramer (2008). Values shown were back calculated from reported S_r and S_r/σ'_{vo}.

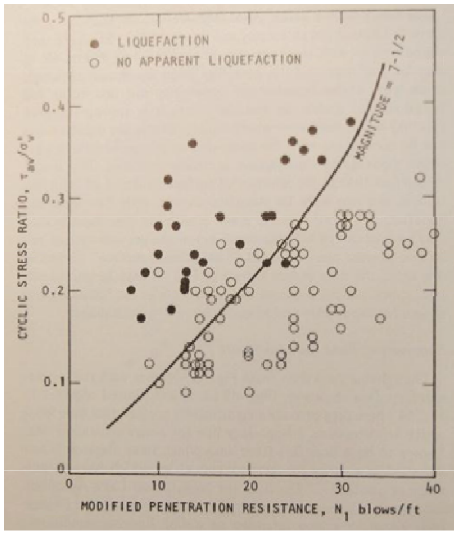
from Weber (2015)

Current State-of-the-Art: Back-Calculation of Residual Shear Strengths from Case Histories

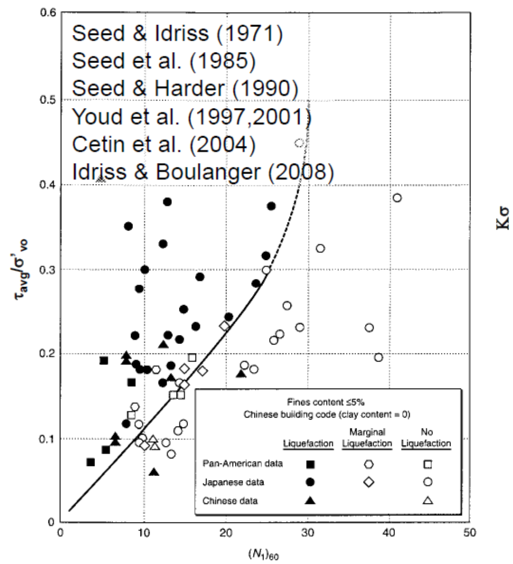


Weber (2015)

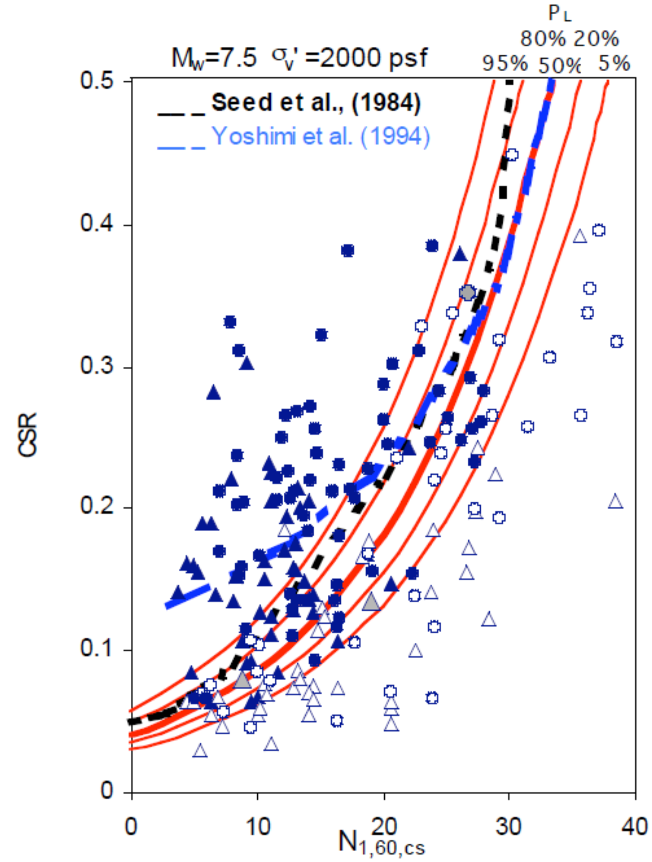
Liquefaction Triggering Relationships Looked at Both Liquefied and Non-Liquefied Case Historires



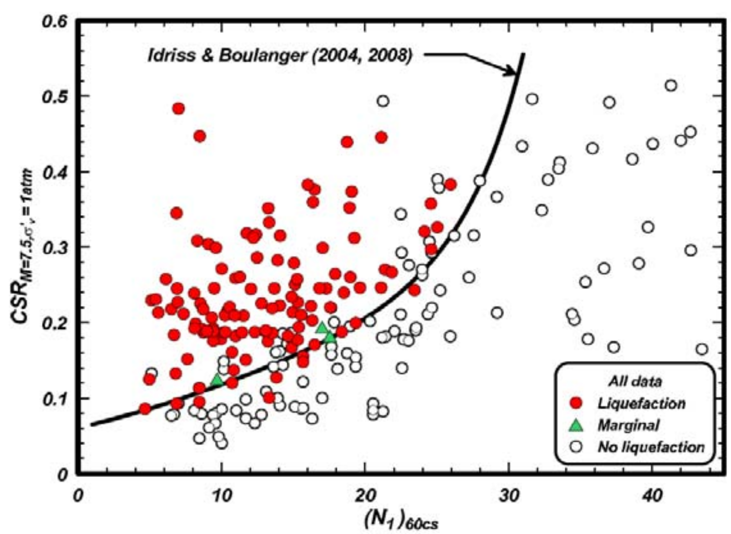
Seed and Idriss (1982)



Seed et al., (1984) and Youd et al. (2001)



Cetin et al. (2004)



Idriss and Boulanger (2010)

Paths Forward

1. **Recognize Bias in current State-of-the-Art.**
2. **Investigate medium dense soils under sloping ground conditions following strong seismic events.**
3. **Current liquefaction triggering correlations indicate that sandy soils with SPT blowcounts between 15 and 25 can be triggered to liquefy if the ground shaking is strong enough.**
4. **Look for such soils/sloping ground conditions, <u>especially if the slopes performed well</u>.**
5. *Learn from what worked, not from what did not.*


4
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Thank you