



# Pile Foundations in Liquefying Sands According to SNI 1726 and SNI 8460

**GEO – Talk VIII**

**31 October 2018**

**JIEXPO – Kemayoran, Jakarta**

**F. X. Toha**

# Bandara Adisucipto, Yogya 2006 Earthquake



# Teluk Bayur Port



# URL of liquefaction videos

- <https://www.youtube.com/watch?v=o4r7aLdRoEE>
- <https://www.youtube.com/watch?v=fEqGktY5aj0>

# Is liquefaction a real threat to us??

- PGA SNI 1726 2012:
  - Jakarta (Utara): 0.350g
  - Bandung (Selatan): 0.497g
  - Surabaya (Utara): 0.322g
  - Denpasar (Barat & Selatan): 0.440g
- Average  $\sigma_v$  and  $\sigma_v'$  @ 6 m depth: 108/48 kPa
- $PGA_M = F_{PGA} PGA$
- $\tau_{cyc} = 0.65 PGA_M/g \sigma_v r_d$

# $F_{PGA}$ (SNI 1726 2012)

Kelas Situs	PGA ≤ 0,1	PGA = 0,2	PGA = 0,3	PGA = 0,4	PGA ≥ 0,5
SA	0,8	0,8	0,8	0,8	0,8
SB	1,0	1,0	1,0	1,0	1,0
SC	1,2	1,2	1,1	1,0	1,0
SD	1,6	1,4	1,2	1,1	1,0
SE	2,5	1,7	1,2	0,9	0,9
SF	Lihat 6.9				

**CATATAN** Gunakan interpolasi linier untuk mendapatkan nilai PGA antara.

Kelas Situs	PGA ≤ 0,1	PGA = 0,2	PGA = 0,3	PGA = 0,4	PGA = 0,5	PGA ≥ 0,6
SA	0,8	0,8	0,8	0,8	0,8	0,8
SB	0,9	0,9	0,9	0,9	0,9	0,9
SC	1,3	1,2	1,2	1,2	1,2	1,2
SD	1,6	1,4	1,3	1,2	1,1	1,1
SE	2,4	1,9	1,6	1,4	1,2	1,1
SF	SS <sup>(a)</sup>					

# CSR<sub>demand</sub> (Cyclic Shear Stress Ratio)

Location	PGA (g) <sup>1</sup>	$F_{PGA}$ <sup>2</sup>	PGA <sub>M</sub> (g)	$\tau_{cyc}/\sigma_v$ <sup>3</sup>
Jakarta (North)	0.350	1.05	0.37	<b>0.51</b>
Bandung (South)	0.497	0.90	0.45	<b>0.62</b>
Surabaya (North)	0.328	1.13	0.37	<b>0.52</b>
Denpasar (South West)	0.440	0.90	0.40	<b>0.55</b>

<sup>1</sup> From [http://puskim.pu.go.id/Aplikasi/desain\\_spektra\\_indonesia\\_2011/](http://puskim.pu.go.id/Aplikasi/desain_spektra_indonesia_2011/)

<sup>2</sup> From SNI 1726 2012 for *SE* site class: Table 8, page 27 of 138

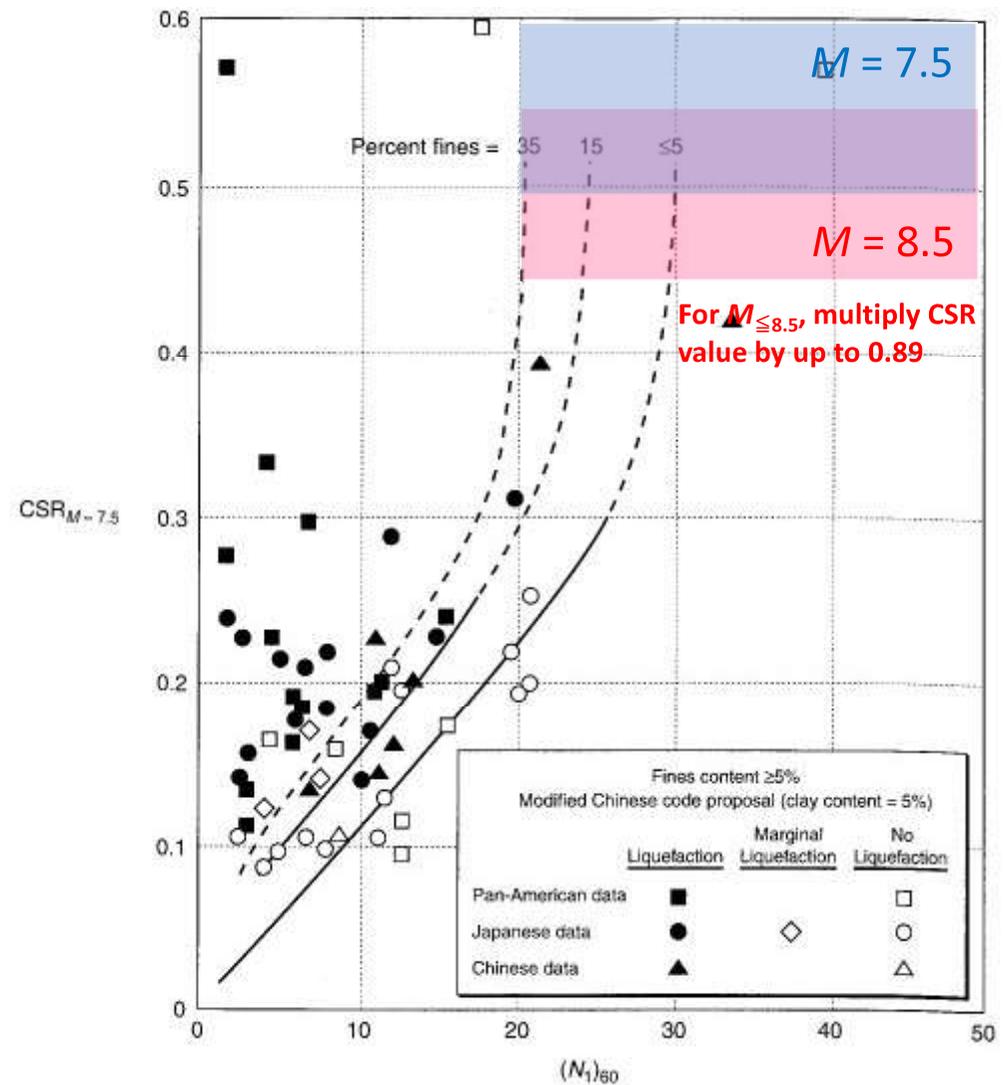
<sup>3</sup> at 6 m depth  $r_d = 0.95$  and  $\sigma_v/\sigma_v' = 2.25$

Earthquake Magnitude,  $M$ , = 7.5, for up to  $M = 8.5$  multiply by up to 0.89

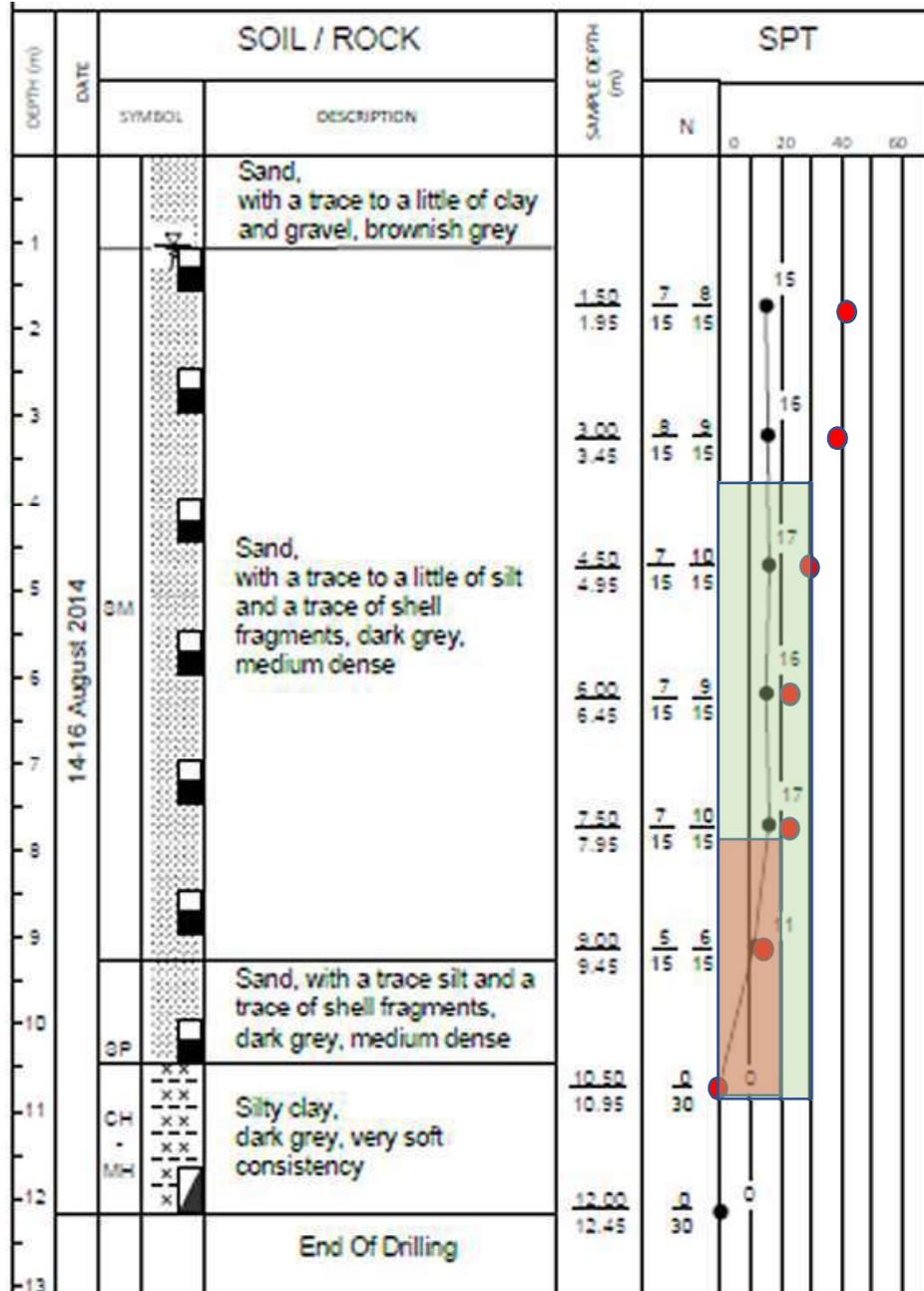
For CSR 0.5 – 0.6,

we need

$$(N_1)_{60} > 20 - 30$$



**Figure 9.31** Relationship between cyclic stress ratios causing liquefaction and  $(N_1)_{60}$  values for silty sands in  $M = 7.5$  earthquakes. (After Seed et al. (1975). Influence of SPT procedures in soil liquefaction resistance evaluations, *Journal of Geotechnical Engineering*, Vol. 111, No. 12. Reprinted by permission of ASCE.)



- $(N_1)_{60} = \sqrt{(1/\sigma'_v)} (N)_{60}$

Liao and Whitman (1986)

Fine < 5%

Fine = 35%

# How likely is a pile foundation to fail in liquefying soils?

- No clear % damage records.
- Only stated that once liquefaction occurred, the damage is extensive, even more so when there is lateral spreading.

Tokimatsu, K. and Asaka, Y. (1998), "Effects of liquefaction-induced ground displacements on pile performance in the 1995 Hyogoken-Nambu earthquake," *Special Issue of Soils and Foundations*, Japanese Geotechnical Society, Sept. 1998.

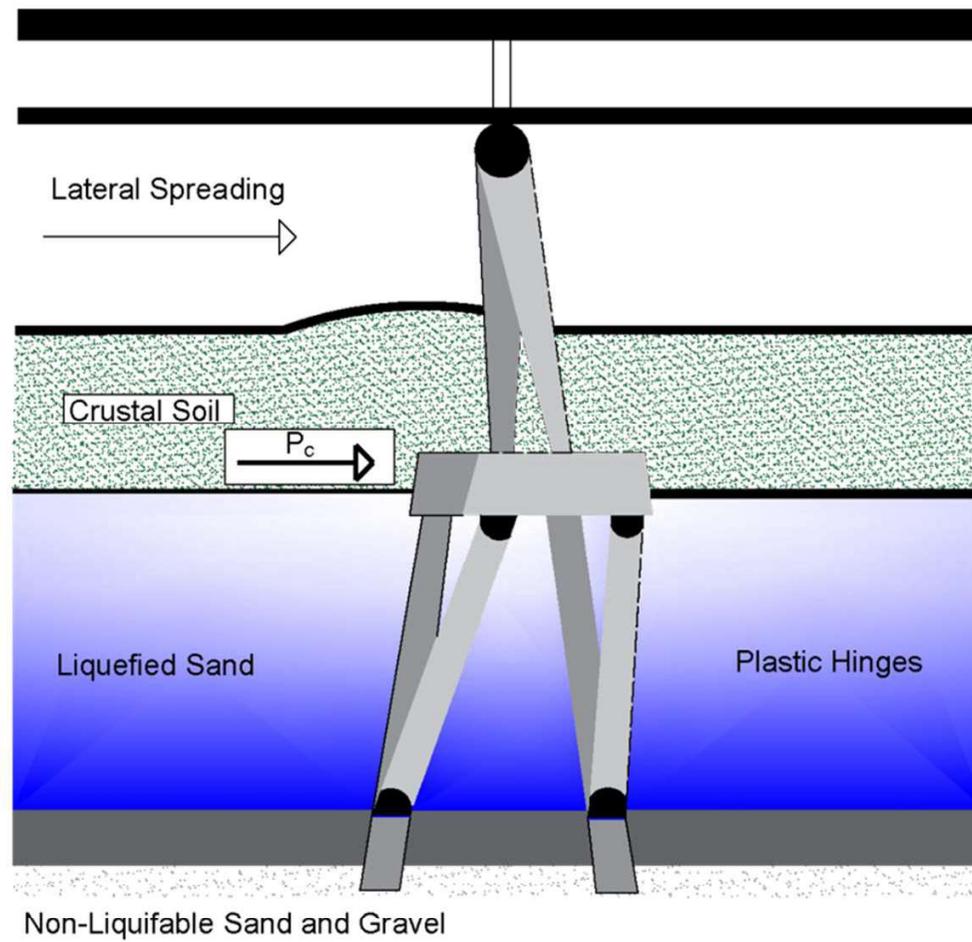
Cubrinovski, C. and Ishihara, K. (2003), "Liquefaction-induced ground deformation and damage to piles in the 1995 Kobe earthquake," *Proc. Int. Conf. on Earthquake Engineering to Mark 40 years from the Catastrophic 1963 Skopje Earthquake and Successful City Construction*, Skopje, Macedonia.

# Case Studies

Summary of case histories on pile foundation performance in past earthquakes.

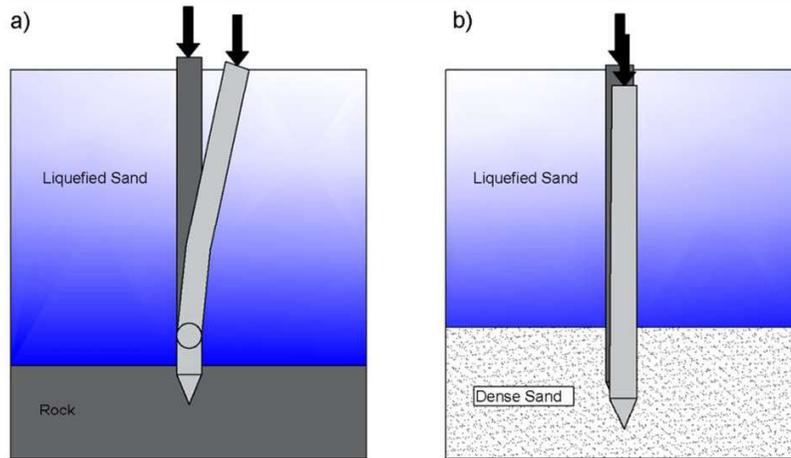
No	Case History	Earthquake Event	Pile Material	Pile diameter (m)	Pile length (m)	Lateral spreading observed?	Pile performance	Reference
1	10 storey-Hokuriku building	Niigata earthquake 1964	RCC	0.4	12	Yes	Good	Hamada (1992a,b)
2	Showa Bridge	Niigata earthquake 1964	Steel tubular	0.6	25	Yes	Poor	Hamada (1992a,b)
3	Landing bridge	Edgecumbe earthquake, 1987	PSC (square)	0.4	9	Yes	Good	Berrill <i>et al.</i> (2001)
4	14 storey building in American park	Kobe earthquake, 1995	RCC	2.5	33	Yes	Good	Tokimatsu <i>et al.</i> (1996)
5	Hanshin expressway pier	Kobe earthquake, 1995	RCC	1.5	41	Yes	Good	Ishihara (1997)
6	LPG tank 101	Kobe earthquake, 1995	RCC	1.1	27	Yes	Good	Ishihara (1997)
7	Kobe Shimim hospital	Kobe earthquake, 1995	Steel tubular	0.66	30	Yes	Good	Soga (1997)
8	NHK Building	Niigata earthquake 1964	RCC	0.35	12	Yes	Poor	Hamada (1992a)
9	NFCH building	Niigata earthquake 1964	RCC hollow	0.35	9	Yes	Poor	Hamada (1992a)
10	Yachiyo Bridge	Niigata earthquake 1964	RCC	0.3	11	Yes	Poor	Hamada (1992a,b)
11	Gaiko Ware House	Chubu earthquake, 1983	PSC hollow	0.6	18	Yes	Poor	Hamada (1992b)
12	4 storey fire house	Kobe earthquake, 1995	PSC	0.4	30	Yes	Poor	Tokimatsu <i>et al.</i> (1996)
13	3 storied building, Fukae at Kobe	Kobe earthquake, 1995	PSC	0.4	20	Yes	Poor	Tokimatsu <i>et al.</i> (1998)
14	Elevated port liner railway	Kobe earthquake, 1995	RCC	0.6	12	Yes	Poor	Soga (1997)
15	LPG tank-106,107	Kobe earthquake, 1995	RCC hollow	0.3	20	No	Poor	Ishihara (1997)
16	Harbour Master's building, Kandla port	Bhuj earthquake, India, 2001	RCC	0.4	25	Yes	Poor	Madabhushi <i>et al.</i> (2005)

RCC – Reinforced concrete  
PSC – Presetressed concrete

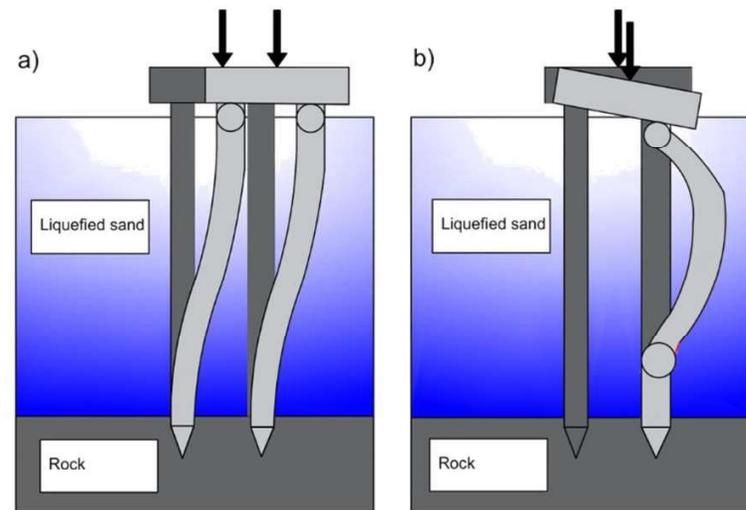


Failure mechanism by formation of plastic hinges in the piles and the pier

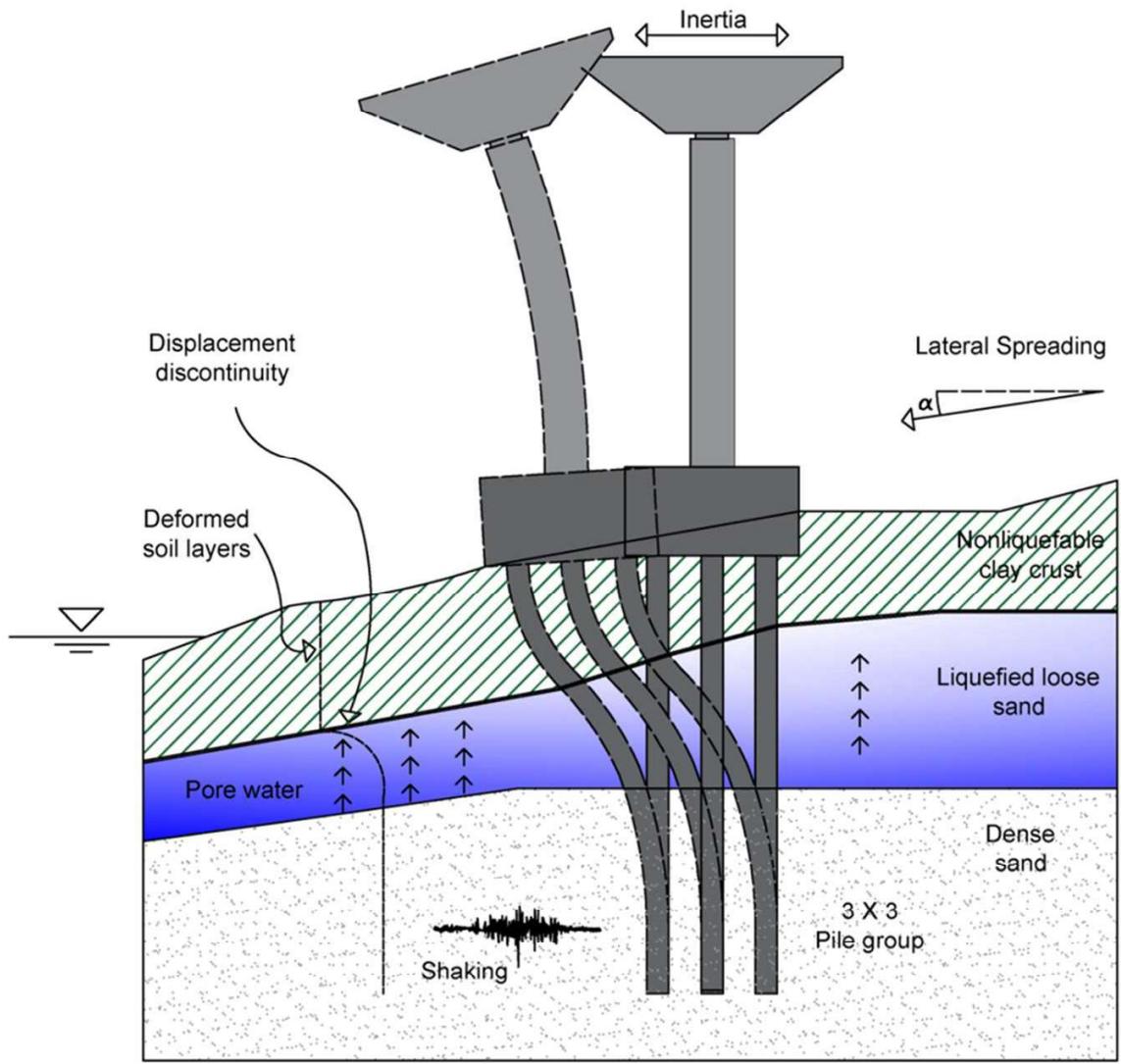
# Failure Mechanisms

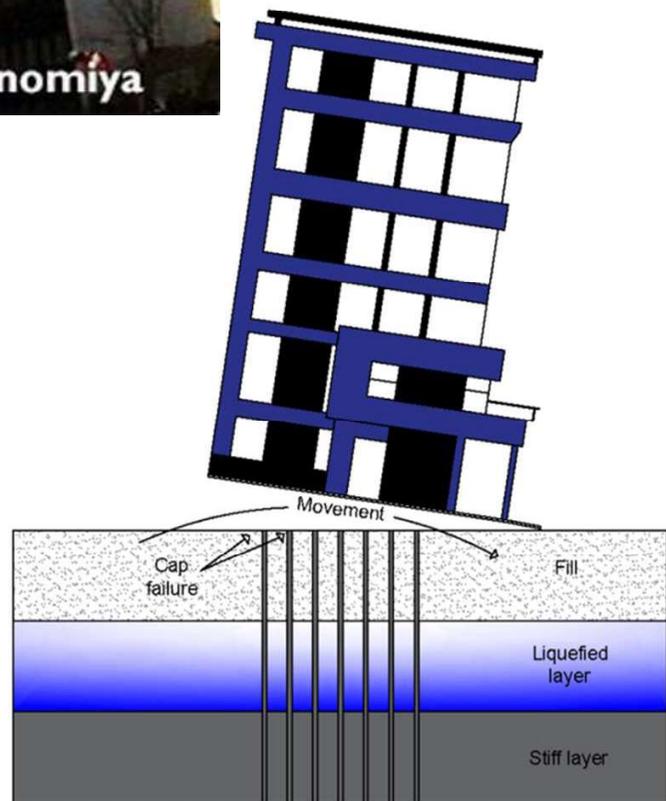
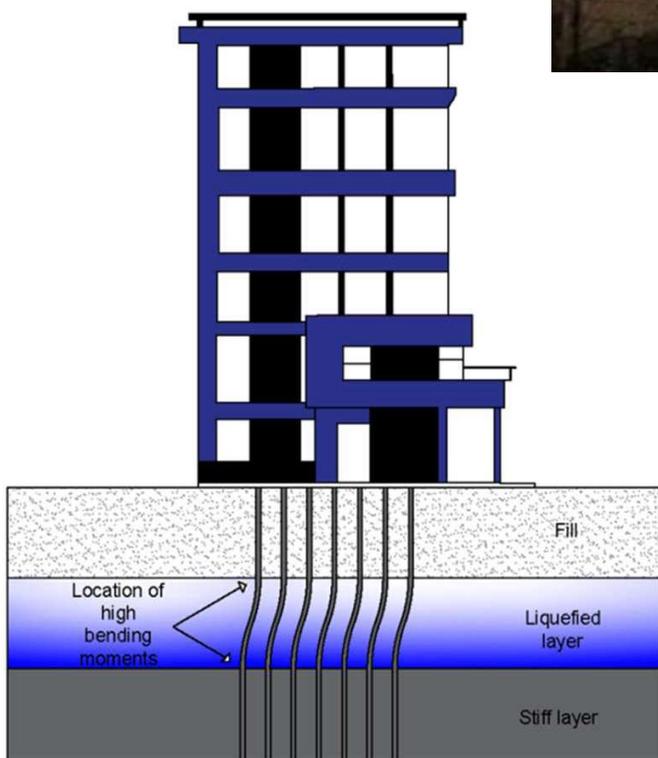


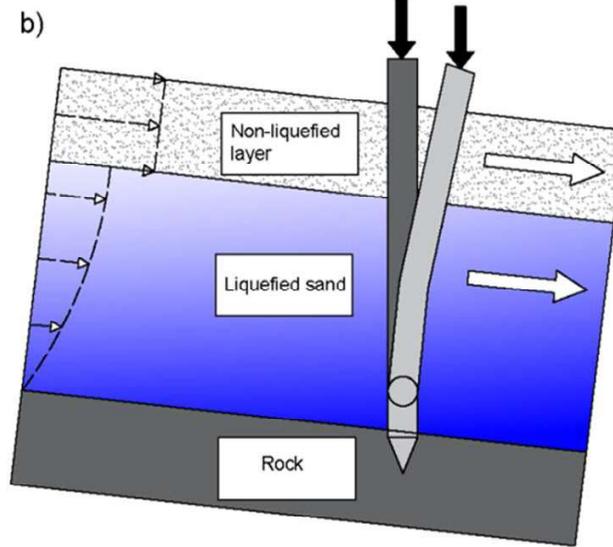
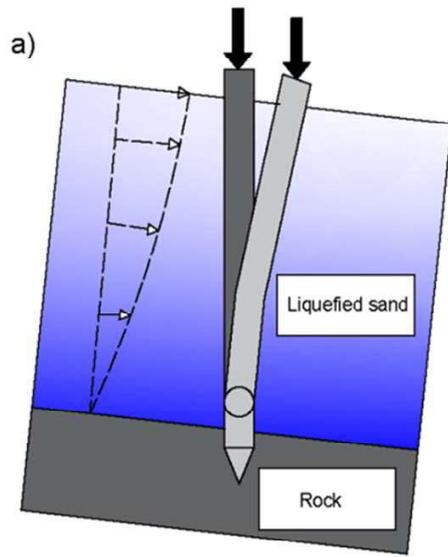
Models of collapse for single piles in liquefiable soil (a) buckling instability (b) bearing failure.



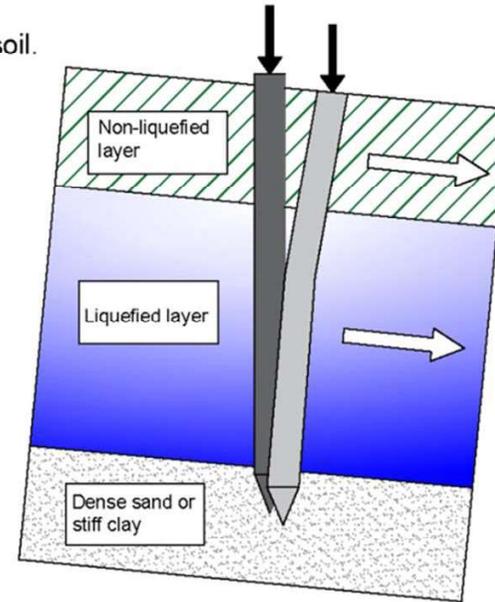
Pile group instability in level ground (a) four hinge mechanism (b) three hinge mechanism



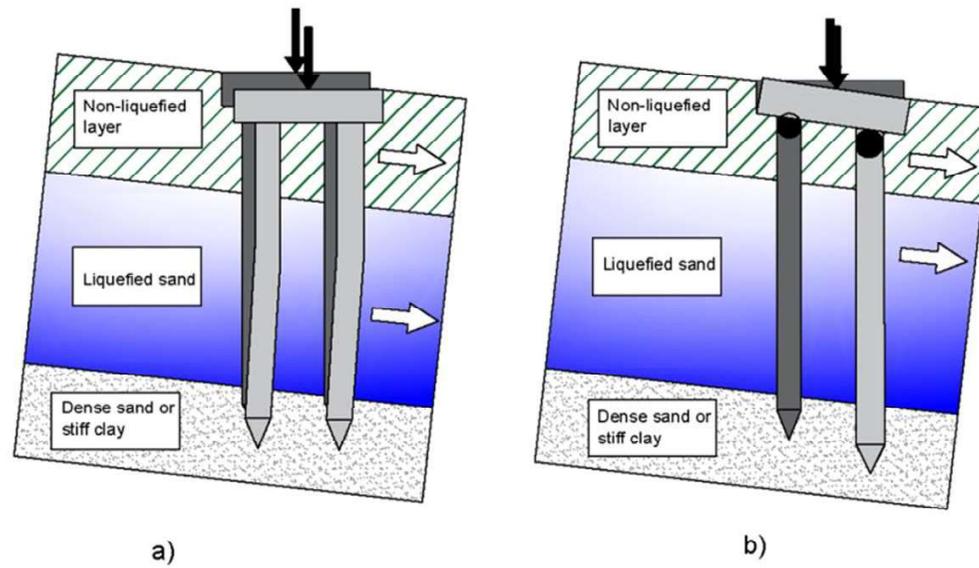




Failure of piles under combined lateral and axial loads in laterally spreading soil.  
 (a) liquefiable sand only (b) with non-liquefied crustal layer



Combined bending and settlement failure of a pile in laterally spreading ground



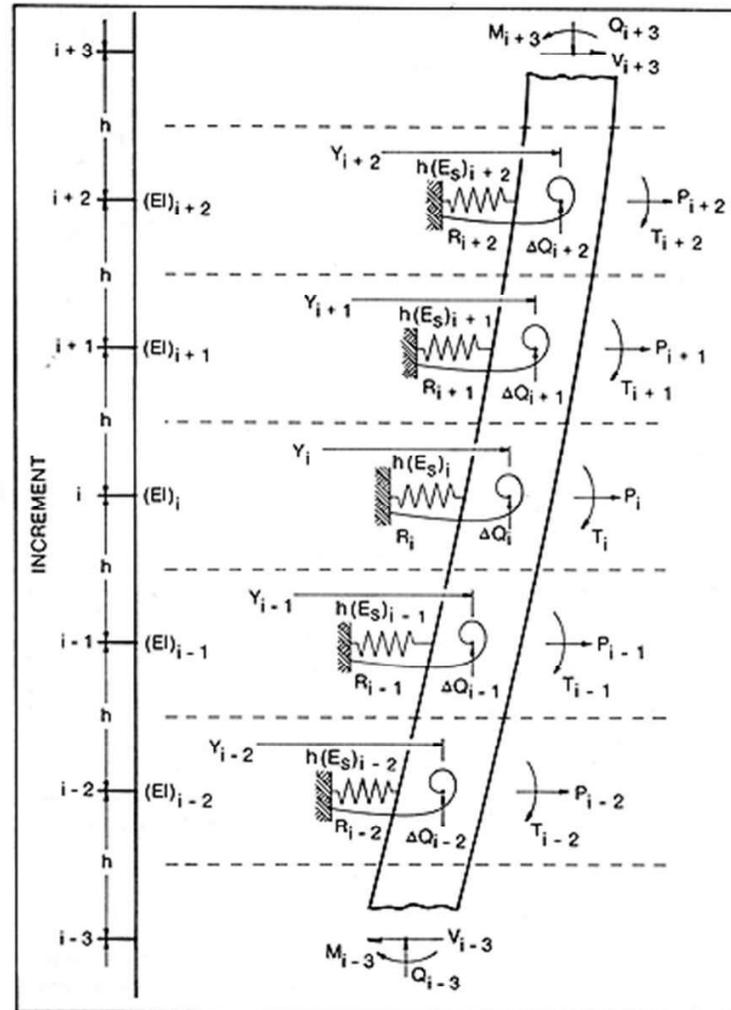
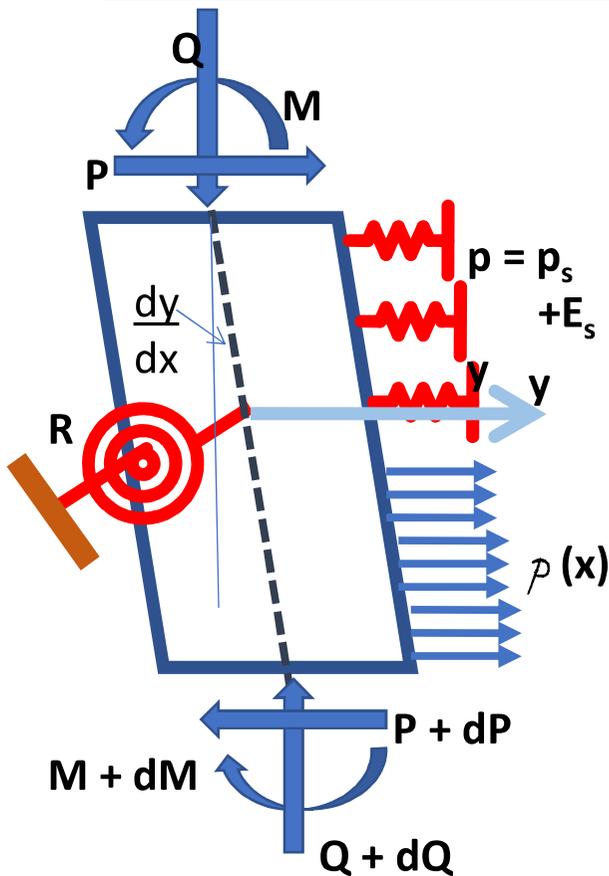
Bearing failure of pile groups in laterally spreading ground (a) bearing failure alone (b) combination of local bearing failure and plastic hinging

# SNI 1726 requirements (& AASHTO)

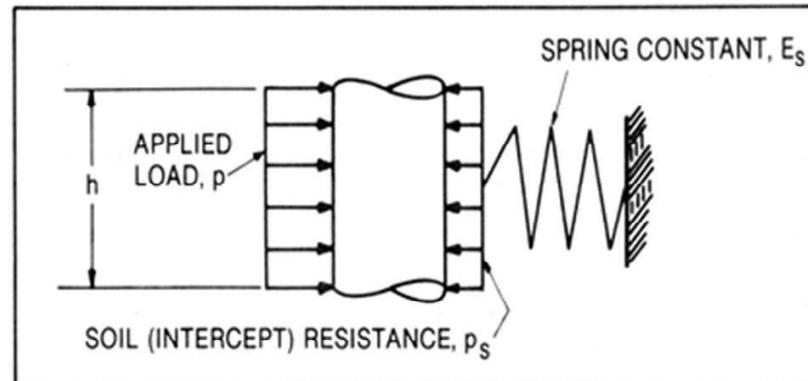
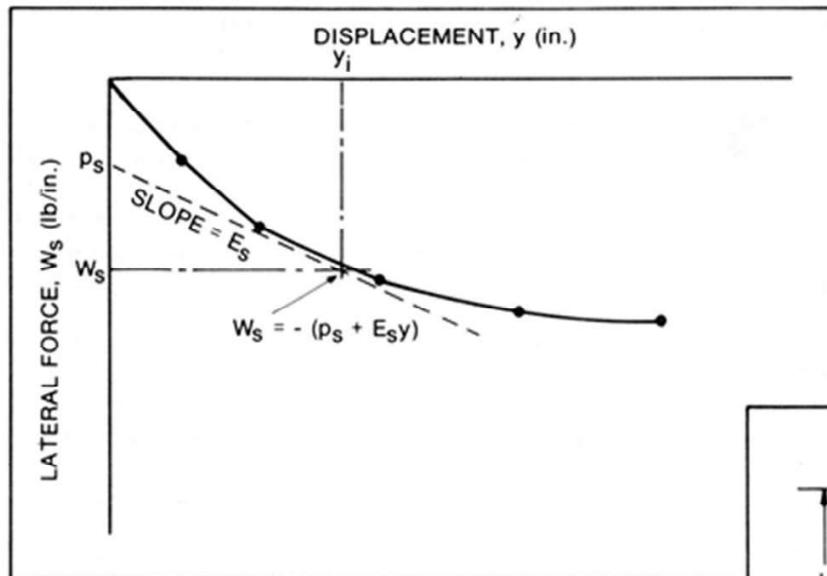
- Analyze using BOTH *SE* and *SF* site class.
- **Case 1. No liquefaction spectra, 100 % inertial and kinematic (not lateral spread) forces.**
- **Case 2. Soil Liquefied, 100% kinematic (lateral spread) + 50% inertial.**
- Case 3. No liquefaction, using upper structure plastic demand.
- Case 4. With liquefaction, upper structure plastic demand.

# Pile model in liquefied sand

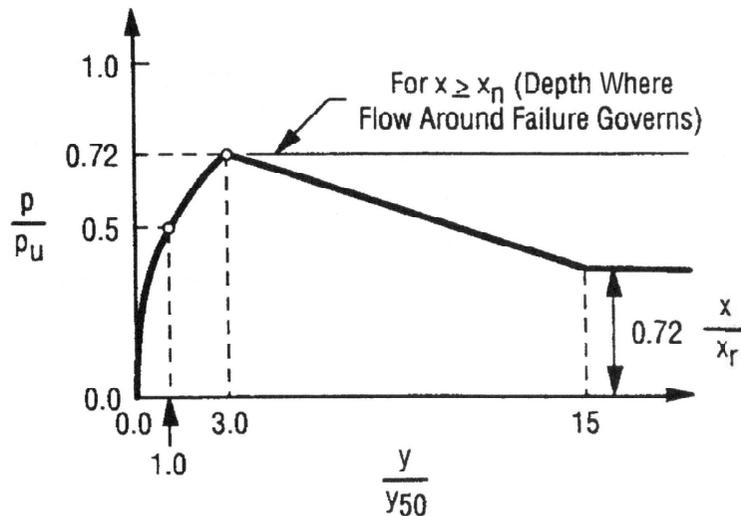
- Beam Column Model



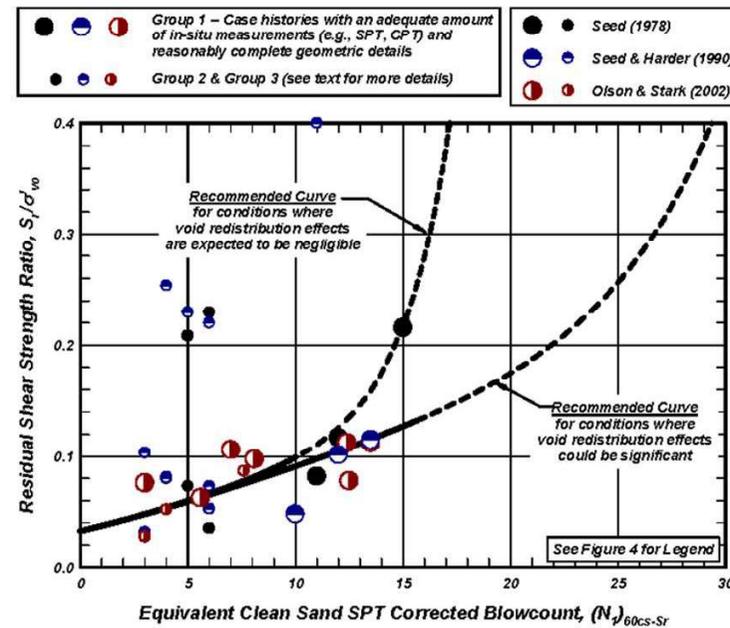
# $p$ - $y$ curve



# $p$ - $y$ curve for liquefied sand (residual strength approach)



$p$ - $y$  curve for liquefied sand  
(=soft clay, saturated, cyclic load)



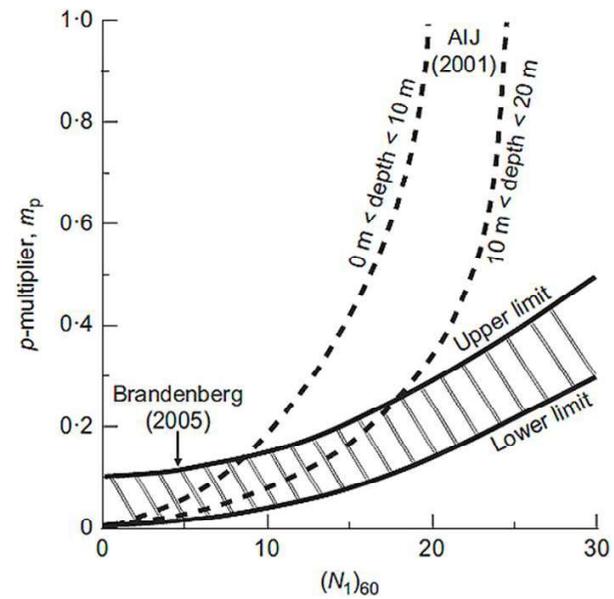
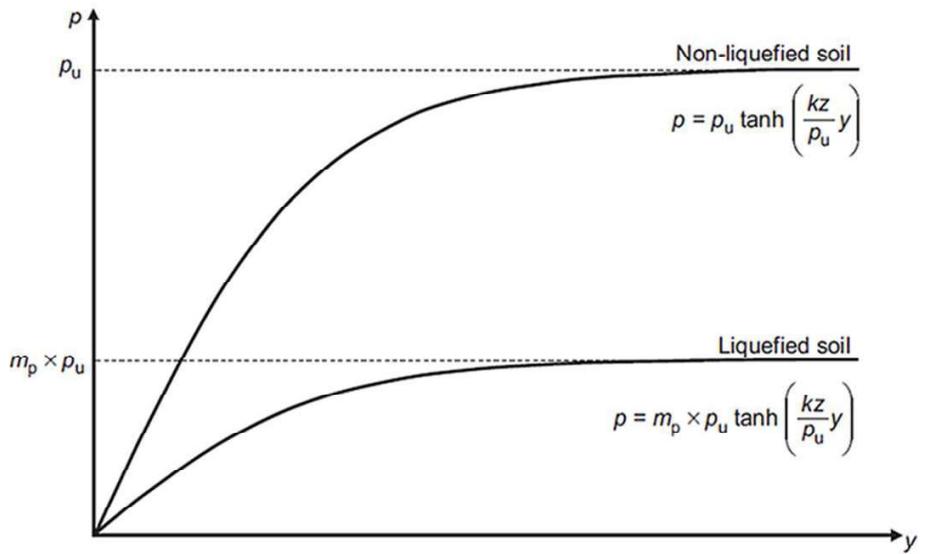
Fines Content, FC (% passing No. 200 sieve)	$\Delta(N_1)_{60-Sr}$
10	1
25	2
50	4
75	5

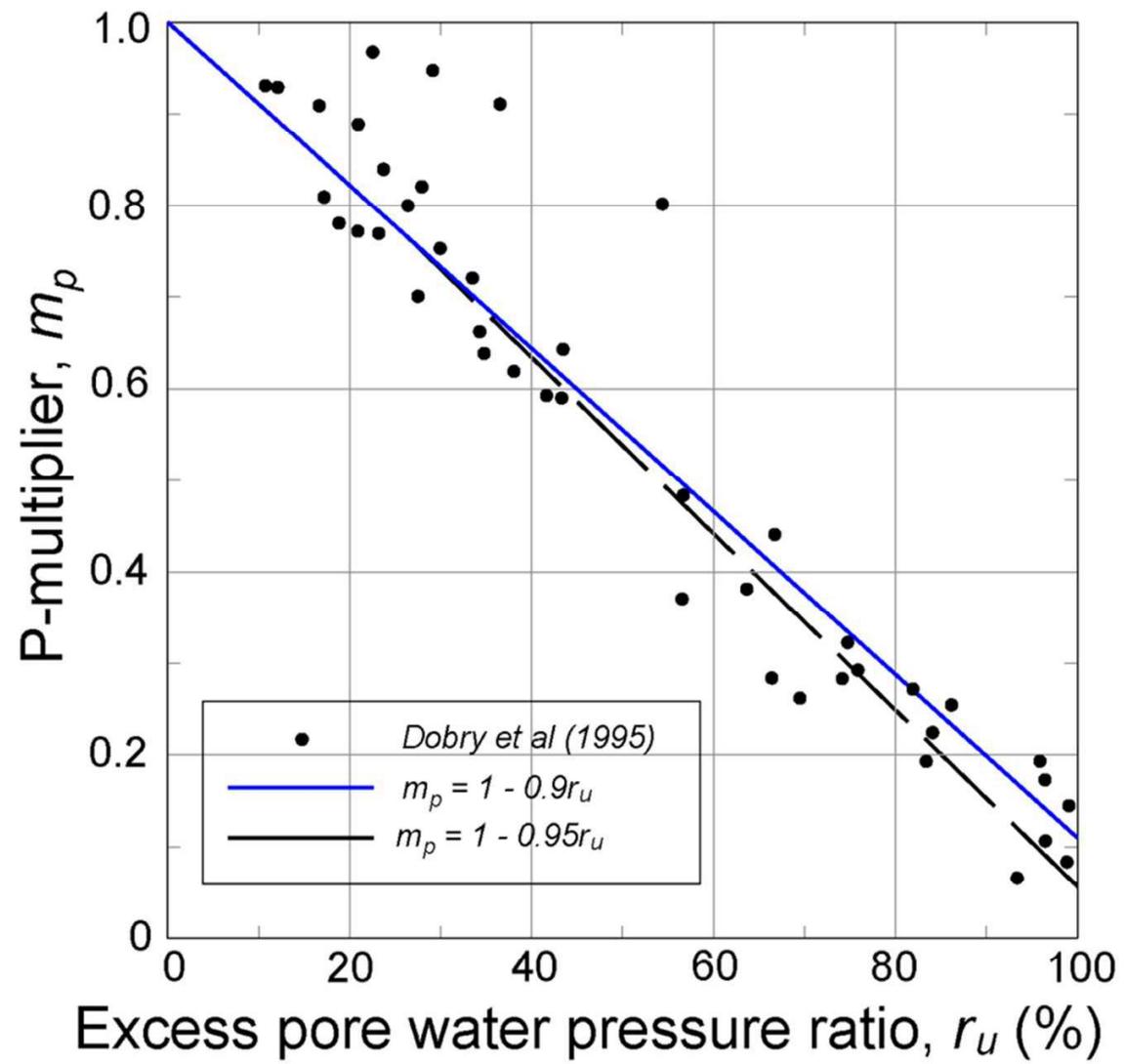
## **Published case histories of liquefaction flow failures**

Case History		$(N_1)_{exc-S}$ ①/②/③ *	FC **	Residual Strength, $S_r$ (kPa) published by			$\sigma'_v$ (kPa)***
Number	Structure			Seed (1987) ①	Seed & Harder (1990) ②	Olson & Stark (2002) ③	
1	Wachusett Dam – North Dike	--/--/7	5–10 / 5	--	--	16.0	151.2
2	Lower San Fernando Dam	15/13.5/11.5	5–90 / 25	35.9	19.2	18.7	166.7
3	Fort Peck Dam	11/10/8.5	55 / 50	28.7	16.8	27.3	351.5
4	Calaveras Dam	12/12/8	10–60 / 60	35.9	31.1	34.5	307.5
5	Hachiro-Gata Road Embankment	--/--/4.4	10–20 / 15	--	--	2.0	32.1
6	Lake Ackerman Highway Embankment†	--/--/3	0–5 / 0	--	--	3.9	51.5
7	Route 272 at Higashiarekinai	--/--/6.3	20 / 20	--	--	4.8	49.3
8	River Bank, Lake Merced	5/6/7.5	1–4 / 3	4.8	4.8	6.9	65.7
9	Kawagishi-cho	4/4/4.4	0–<5 / 3	5.7	5.7	5.3	70.6
10	Mochi-koshi Tailings Dam – Dike 2	6/5/2.7	>60–85 / 85	12.0	12.0	5.4	52.2
11	La Marquesa Dam – U/S Slope ††	--/6/4.5	30 / 30	--	9.6	3.1	43.6
12	La Marquesa Dam – D/S Slope †††	--/11/9	20 / 20	--	19.2	5.3	47.9
13	La Palma Dam ††††	--/4/3.5	15 / 15	--	9.6	4.8	37.8
14	Uetsu Railway Embankment	3/3/3	0–2 / 2	1.7	1.9	1.7	61.3
15	Solfataro Canal Dike	5/4/4	<5–8 / 7	6.2	2.4	2.4	29.9
16	Koda Numa Railway Embankment	3/3/3	13 / 13	2.4	2.4	1.2	23.2
17	Shibecha-cho Embankment	--/--/5.6	12–35 / 25	--	--	5.6	64.7
18	Sheffield Dam	6/6/5	25–48 / 40	2.4	3.6	3.6	68.4

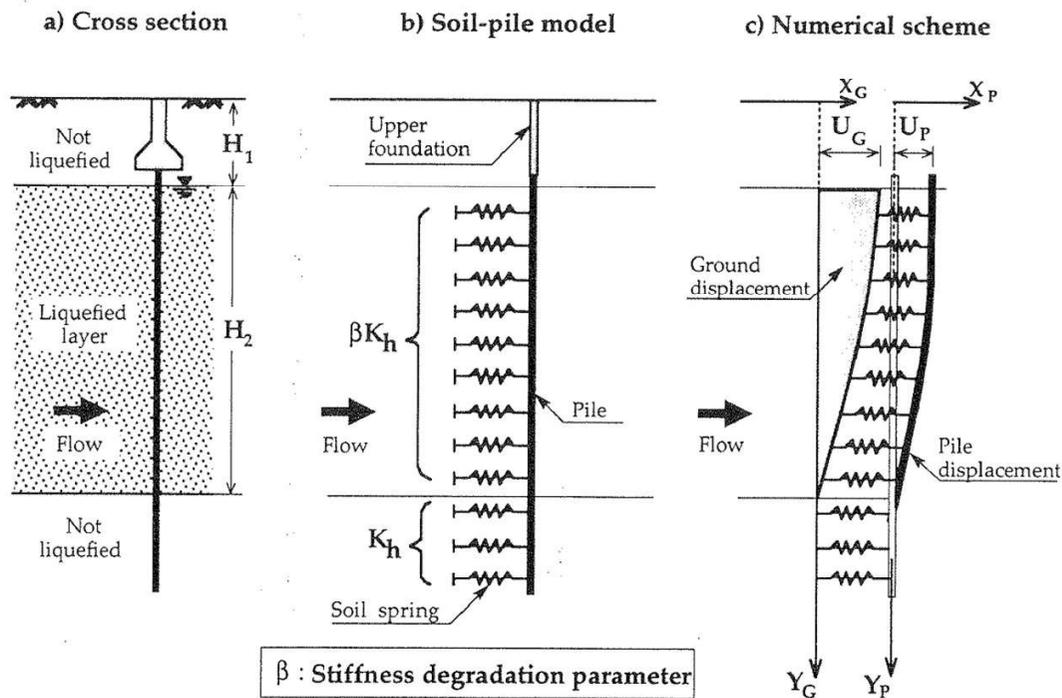
Cases 1 – 7 fit into Group 1, which consists of Case Histories with SPT and/or CPT measurements and reasonably complete geometric details.  
 Cases 8 – 13 fit into Group 2, which consists of Case Histories with SPT and/or CPT measurements, but geometric details are incomplete.  
 Cases 14 – 18 fit into Group 3, which consists of Case Histories with estimated SPT and/or CPT, but geometric details are reasonably complete.

# $p$ - $y$ curve for liquefied sand (degradation factor approach)





# Lateral spread soil pressure in liquefied sands

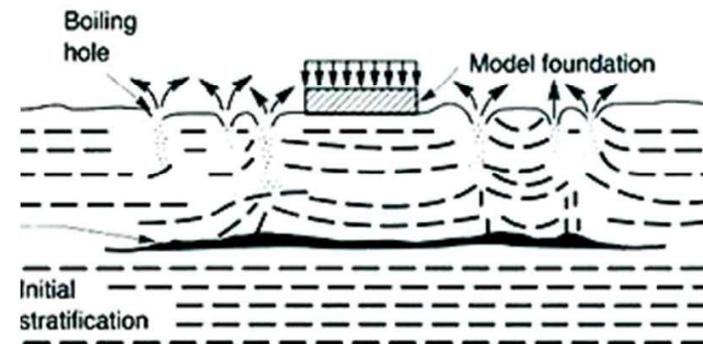
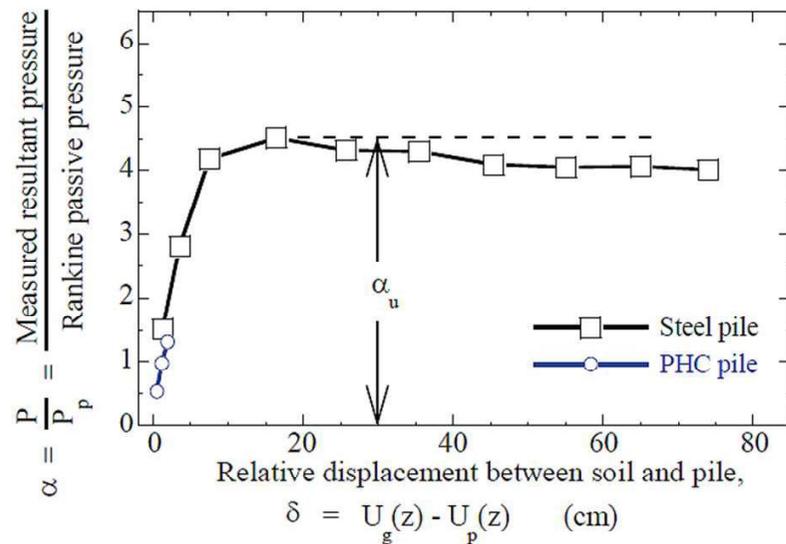


$$F = \beta k d (U_G - U_P)$$

$$\beta = 2 \times 10^{-4} \text{ to } 2 \times 10^{-2}$$

Ishihara, K., Cubrinovski, M., (1998), "Performance of large diameter piles subject to lateral spreading of liquefied deposits," *Proc. 13<sup>th</sup> Southeast Asian Geotechnical Conference, Taipei.*

# Lateral pressure in non liquefied crust



Adjust  $\alpha_u$  when pore pressure is generated in crust and surface manifestation occurred.

Lateral pressure from the crust layer on a single pile (full-size test on piles)

Cubrinovski, M., Kokusho, T., Ishihara, K., (2006). "Interpretation from large scale shake table tests on piles undergoing lateral spreading in liquefied soils," *Soil Dynamics and Earthquake Engineering*, Vol. 26.

# Lateral spread displacements

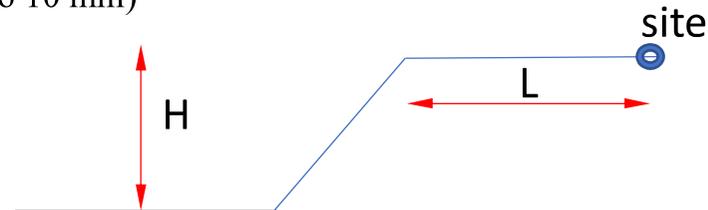
Youd, T.L, Hansen, C.M., Bartlett, S., (2002) "Revised multilinear regression equations for prediction of lateral spread displacement," *Jour. Geotech. & Geoenv. Eng.*, ASCE, Dec.

For free face condition:

$$\begin{aligned}\log D_H = & -16.713 + 1.532M - 1.406 \log R^* - 0.012R \\ & + 0.592 \log W + 0.540 \log T_{15} + 3.413 \log(100 - F_{15}) \\ & - 0.795 \log(D50_{15} + 0.1 \text{ mm})\end{aligned}$$

where:

$D_H$  = lateral displacement (m),  $M$  = magnitude (6 to 8 range),  $R$  = closest horizontal distance to surface projection of seismic source ( $> 0.5$  km),  $R^* = R + R_o$ ,  $R_o = 10^{(0.89M-5.64)}$ ,  $W = 100 H/L$  (range 1 to 20%),  $T_{15}$  = cumulative thickness (m) of soils with  $(N_1)_{60} < 15$  (range 1 to 15 m),  $F_{15}$  = average fine content within  $T_{15}$ ,  $D50_{15}$  = average value of  $D50$  within  $T_{15}$  (range 0.1 to 10 mm)



# Lateral spread displacements

Youd, T.L, Hansen, C.M., Bartlett, S., (2002) "Revised multilinear regression equations for prediction of lateral spread displacement," *Jour. Geotech. & Geoenv. Eng.*, ASCE, Dec.

For sloping ground  $S$  (in %, ranging from 1 to 6 %):

$$\begin{aligned}\log D_H = & -16.213 + 1.532M - 1.406 \log R^* - 0.012R \\ & + 0.338 \log S + 0.540 \log T_{15} + 3.413 \log(100 - F_{15}) \\ & - 0.795 \log(D50_{15} + 0.1 \text{ mm})\end{aligned}$$

where:

$D_H$  = lateral displacement (m),  $M$  = magnitude (6 to 8 range),  $R$  = closest horizontal distance to surface projection of seismic source ( $> 0.5$  km),  $R^* = R + R_o$ ,  $R_o = 10^{(0.89M-5.64)}$ ,  $T_{15}$  = cumulative thickness (m) of soils with  $(N_1)_{60} < 15$  (range 1 to 15 m),  $F_{15}$  = average fine content within  $T_{15}$ ,  $D50_{15}$  = average value of  $D50$  within  $T_{15}$  (range 0.1 to 10 mm)

# Can we prevent liquefaction?

- **YES!!**

- Densify the sand
- Allow pore pressure distribution
- Reduce cyclic shear stress occurrence in sand by reinforcing the layer
- Make the pore water compressible
- Make the sand “cohesive” (bio-chemical)

# Densification

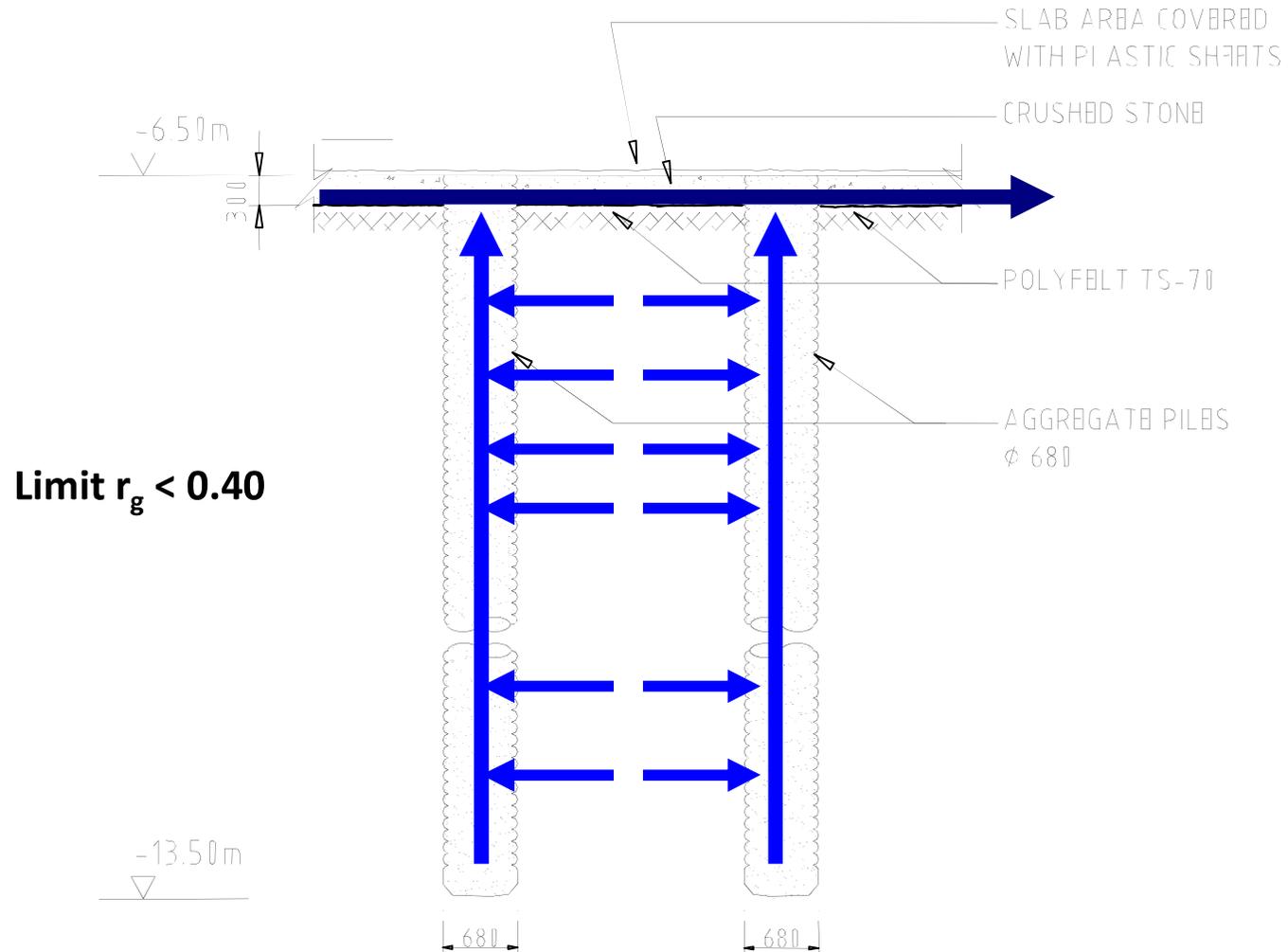
- Dynamic compaction
- Vibro techniques (Vibroflotation)
- Compaction Grouting or other Large Displacement Insertion
- Blasting

# Pore Pressure Dissipation

- Synthetic Seismic Drains
- Stone Columns

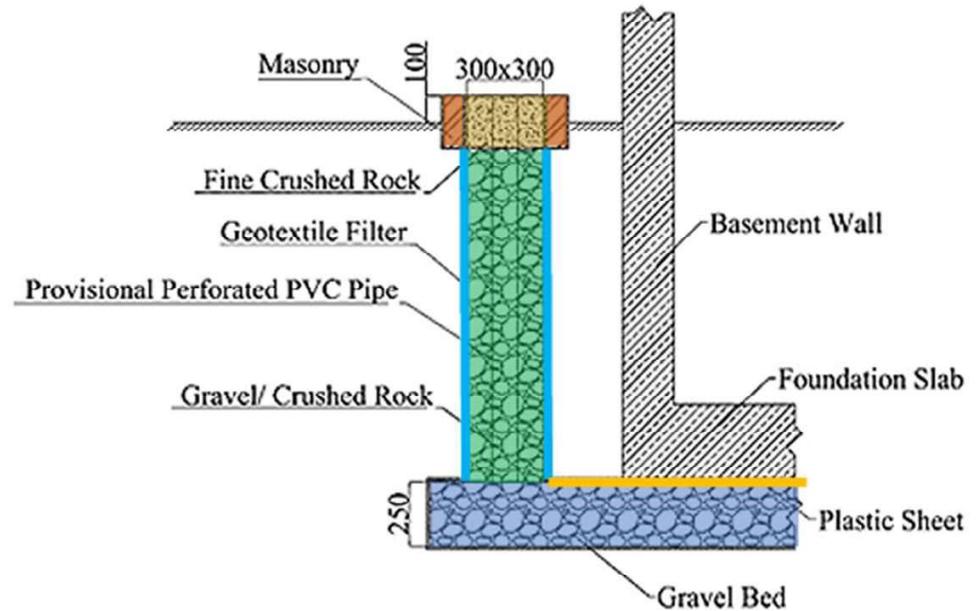
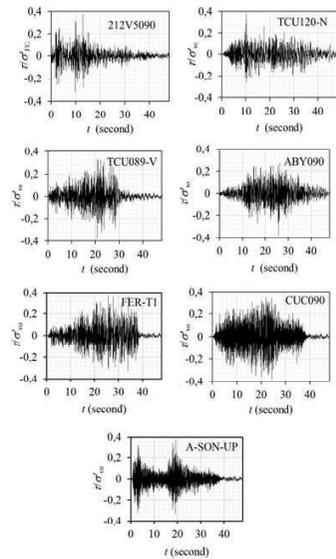
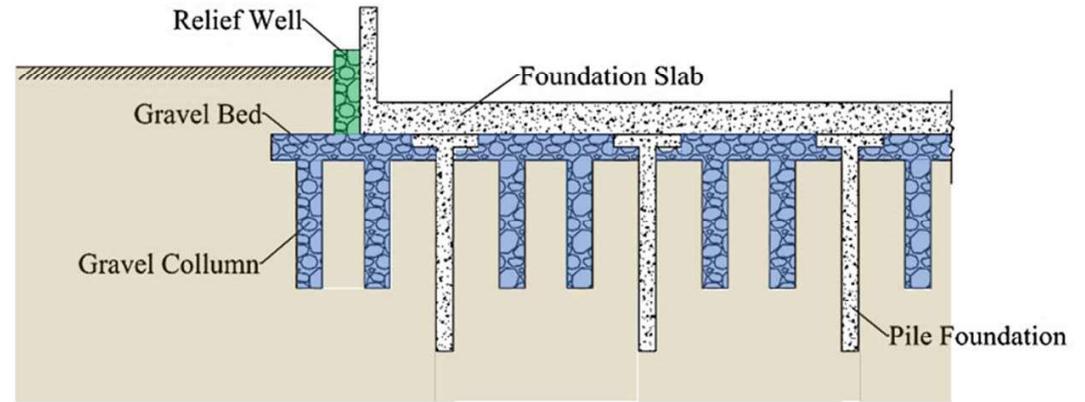
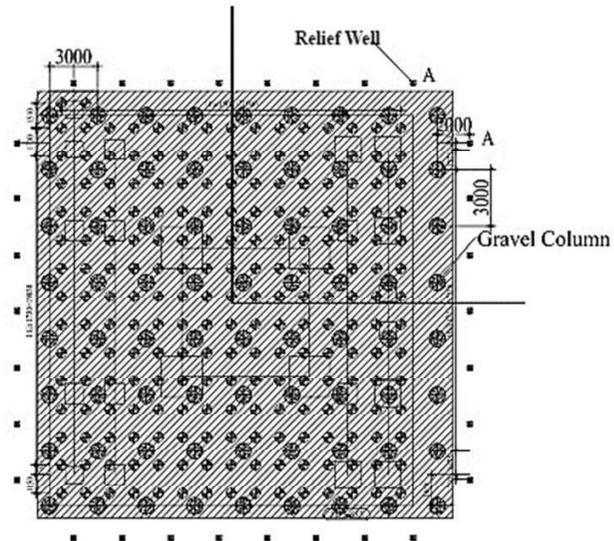


# Pore Pressure Dissipation by Aggregate Columns





# Drain System



# Sand Reinforcement

Rigid inclusions:

- Stone Columns
- DCM or Jet Grouting
- Piles
- Shear Beam Theory (Baez)
- Bending Stiffness as well (Goughnour & Pestana)

# Stone Column



# Reinforcing effect of rigid inclusions

- Baez & Martin (1994):

$$K_G = \frac{\tau_{sand}}{\tau} = \frac{1}{1 + \frac{A_{SC}}{A} \left( \frac{G_{SC}}{G_S} - 1 \right)}$$

- Goughnour & Pestana (1998):

$$K_G = \frac{1 + \frac{A_{SC}}{A} \left( \frac{\sigma'_{v,SC}}{\sigma'_{v,S}} - 1 \right)}{1 + \frac{A_{SC}}{A} \left( \frac{G_{SC}}{G_S} - 1 \right)}$$

See ben salem JGGE Oct 2017

# CAP Reinforcement effect

- Seismic Tests, 2 sets, 1 set for Process and 1 set for Non Process Area each. Measured average  $v_s$ , shall be  $> 175$  m/sec.
- CAP Axial Load Test

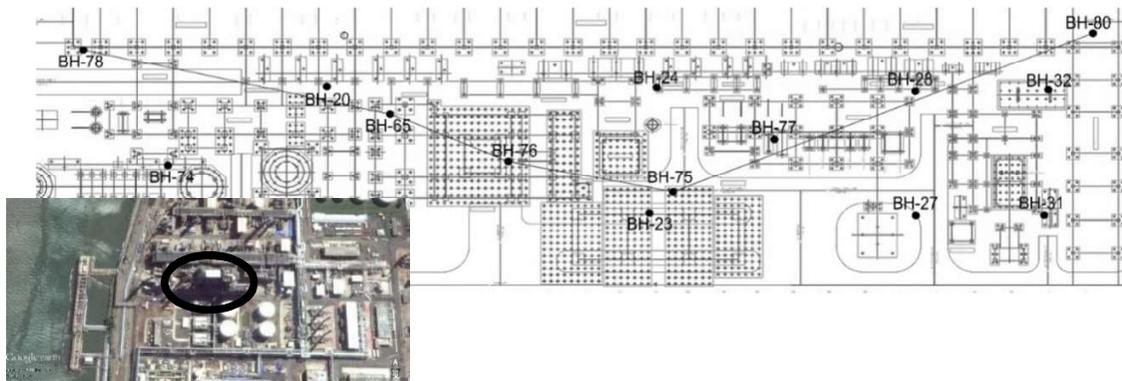
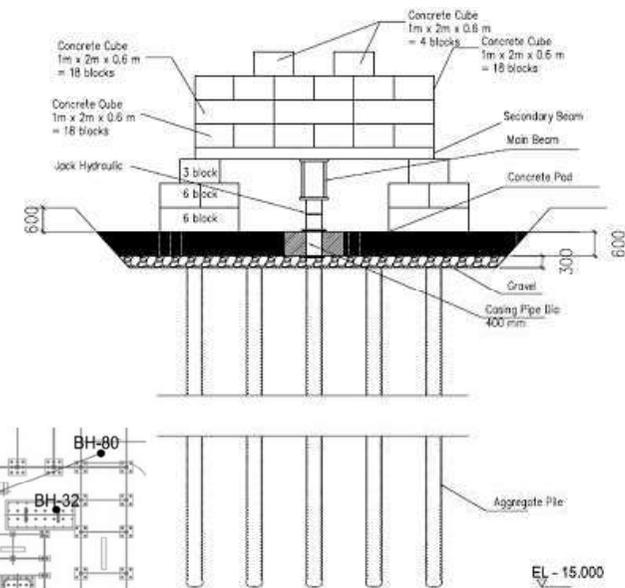


Figure 21. CAP load test set up.

# Stone Column Performance

Quimby, M.J. (2009),  
Liquefaction Mitigation in  
Silty Sands using Stone  
Columns with Wick Drains,  
MSc Thesis, BYU, Utah

No.	Location	Project Type	Site Characteristics (Soil Profile)	Initial Soil Properties	Reference
1	Treasure Island San Francisco, CA	Medical / Dental Bldg. (40% complete, 2-story steel-frame structure)	6 m loose to med. dense hydraulic sand fill over 6 m very loose silty sand fill over Bay Mud, GWT @ 2.3 m depth	$(N_1)_{60} = 4-62$ avg. $(N_1)_{60} = 27$ , <10% fines in upper sand layer	Mitchell and Wentz (1991) Bolt (1990)
2	Treasure Island San Francisco, CA	Office building No. 450 (3-story steel-frame with concrete walls and floors)	9 m loose to med. dense hydraulic sand fill over 2.5 m med. dense sand over Bay Mud	$(N_1)_{60} = 3-54$ avg. $(N_1)_{60} = 19$ , <10% fines	Mitchell and Wentz (1991) Bolt (1990)
3	Treasure Island San Francisco, CA	Facilities 487, 488, 489 (3-story concrete bldgs.)	6 m very loose to med. dense hydraulic sand fill over 5 m med. dense silty sand fill over Bay Mud, GWT @ 3.0 m depth	<12% fines in upper sand layer	Mitchell and Wentz (1991) Bolt (1990)
4	Treasure Island San Francisco, CA	Approach Area Pier 1	1.3 m loose to med. dense hydraulic sand fill over Bay Mud, GWT @ 3.0 m depth	<10% fines	Mitchell and Wentz (1991) Bolt (1990)
5	Treasure Island San Francisco, CA	Building No. 453 (4-story concrete bldg.)	8 m loose to med. dense hydraulic sand fill over 5.5 m loose silty sand fill over Bay Mud, GWT @ 2.8 m depth	$(N_1)_{60} = 3-46$ avg. $(N_1)_{60} = 14$ , <12% fines in upper sand layer	Mitchell and Wentz (1991) Bolt (1990)
6	Richmond, CA	Marina Bay Esplanade, Buttress Against Lateral Spreading	4 m med. dense to dense sandy and gravelly artificial fill over 3.5 m loose silty sand hydraulic fill over Bay Mud, GWT @ 1.4 m depth	$(N_1)_{60} = 11-22$ avg. $(N_1)_{60} = 15$ , avg. $q_{c1} = 45 \text{ kg/cm}^2$ , <55% fines in silty sand layer	Mitchell and Wentz (1991) Bolt (1990)
7	Emeryville, CA	East Bay Park Condominiums	3-6 m med. dense hydraulic sand fill over Bay Mud, GWT @ 1.5 m depth	avg. $(N_1)_{60} = 18$ , <5% fines in sand	Mitchell and Wentz (1991) Bolt (1990)
8	Alameda, CA	Perimeter Sand Dike, Harbor Bay Business Park	Loose to dense silty sand hydraulic fill overlying dense sand with pockets of soft to med. stiff silty clay with peat, GWT @ 3 m depth	Fill: $N_{60} = 2 \text{ to } \geq 25$ $q_c = 10 \text{ to } \geq 80 \text{ tsf}$ , <11% fines in hydraulic sand fill	Mitchell and Wentz (1991) Bolt (1990)
9	Union City, CA	Hanover Properties, 5 tilt-up panel bldgs. cover an area of 18,580 m <sup>2</sup>	0.6-0.9 m hard clayey silt fill over 0.6 m of alternating layers of loose sand and firm silt over Bay mud. GWT @ 2.1 m depth	n/a	Mitchell and Wentz (1991) Bolt (1990)
10	South San Francisco, CA	Kaiser Hospital Addition	2.4 m of unconsolidated fill over 8 m of loose to med. dense hydraulic sand fill	avg. $(N_1)_{60} = 19$	Mitchell and Wentz (1991) Bolt (1990)
11	Santa Cruz, CA	Riverside Avenue Bridge	1.5 m of sat. loose to med. dense sandy gravel over 3.4 m dense gravelly sand; soils are submerged	<5% fines	Mitchell and Wentz (1991) Bolt (1990)
12	Santa Cruz, CA	Adult Detention Facility	1.2-3.6 m of firm to very stiff clays and silts and medium to very dense sands and gravels over 6-21 m of soft to stiff sandy silts and loose to med. dense silty sands over siltstone bedrock, GWT @ 4.5 m depth	$(N_1)_{60} = 4-27$ avg. $(N_1)_{60} = 13$	Mitchell and Wentz (1991) Bolt (1990)

Recent centrifugal  
model tests @  
Univ. of Colorado  
Boulder

Isolated drain limited reliability

$A_r$  drainage capability and frequency, critical parameters for liquefaction mitigation.

$A_r < 10\%$  may not be effective in sand layer shear reduction

Granular columns presence may increase the seismic demand

Soils with higher fine contents may require higher  $A_r$ , as drainage capacity is reduced.

Badanagki, M., Dashti, S., Kirkwood, P., (2018), "Influence of dense granular columns on the performance of level and gently sloping liquefiable sites," *J. Geotech. Geoenviron. Eng.*, 144(9), ASCE.

# Size matters....!!!

