Earthquake and Tsunami Disaster Mitigation Lecture Session III

Liquefaction of Soils during Earthquakes

by
Assist.Prof. Dr. E. Ece Bayat
Istanbul Technical University

20 October 2017

OUTLINE

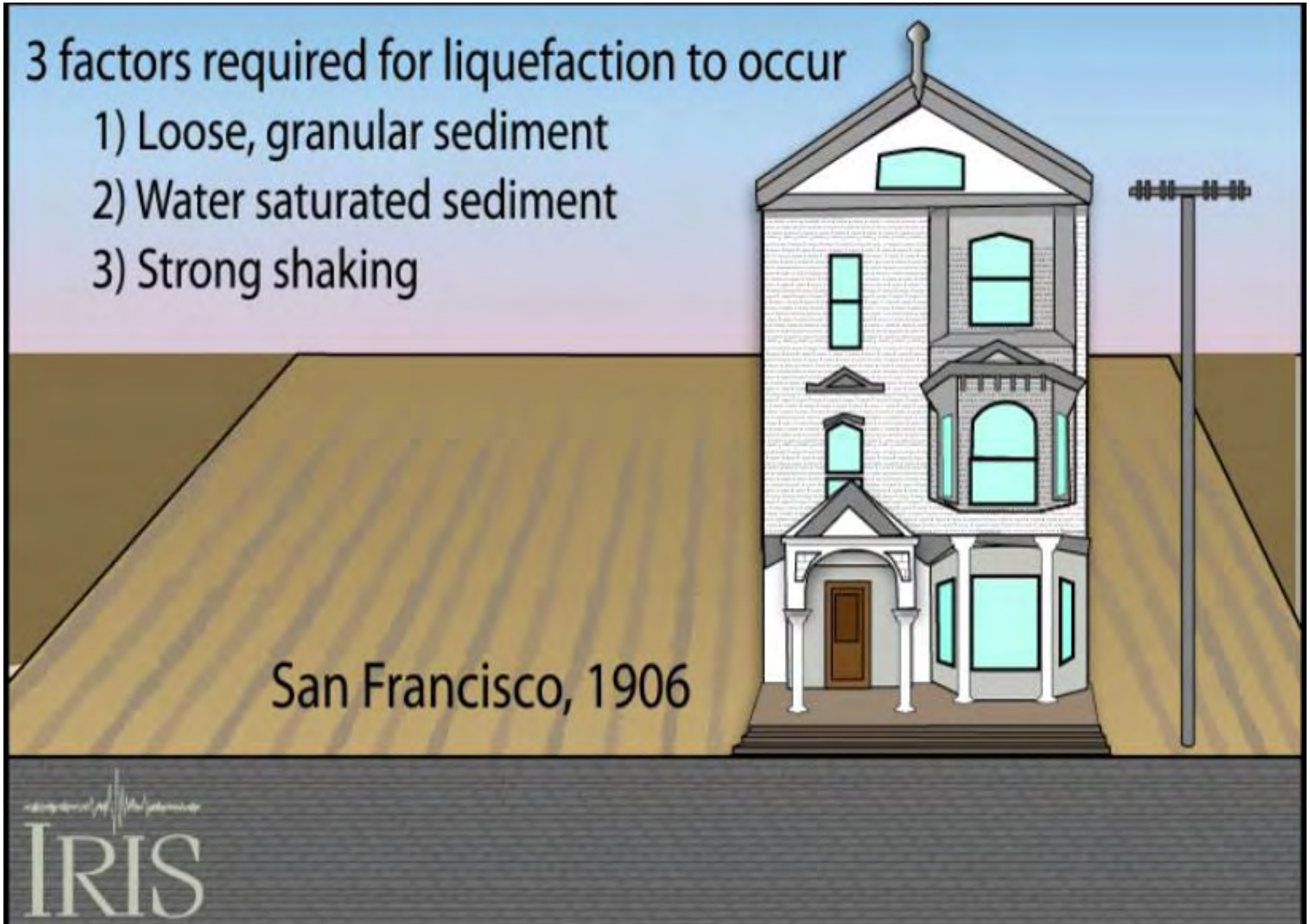
1. **Undrained Response of Sands under Dynamic Loads**
2. **Initial Liquefaction (Level ground liquefaction)**
3. **Evaluation of Initiation of Liquefaction**
4. **Consequences of Soil Liquefaction**
5. **Liquefaction Mitigation Methods**

1. Undrained Response of Sands under Dynamic Loads

Dynamic: Foundation failure by liquefaction after the 1964 Niigata Earthquake. (USGS)

Sand Boils, June 16 Niigata Earthquake M 7.5

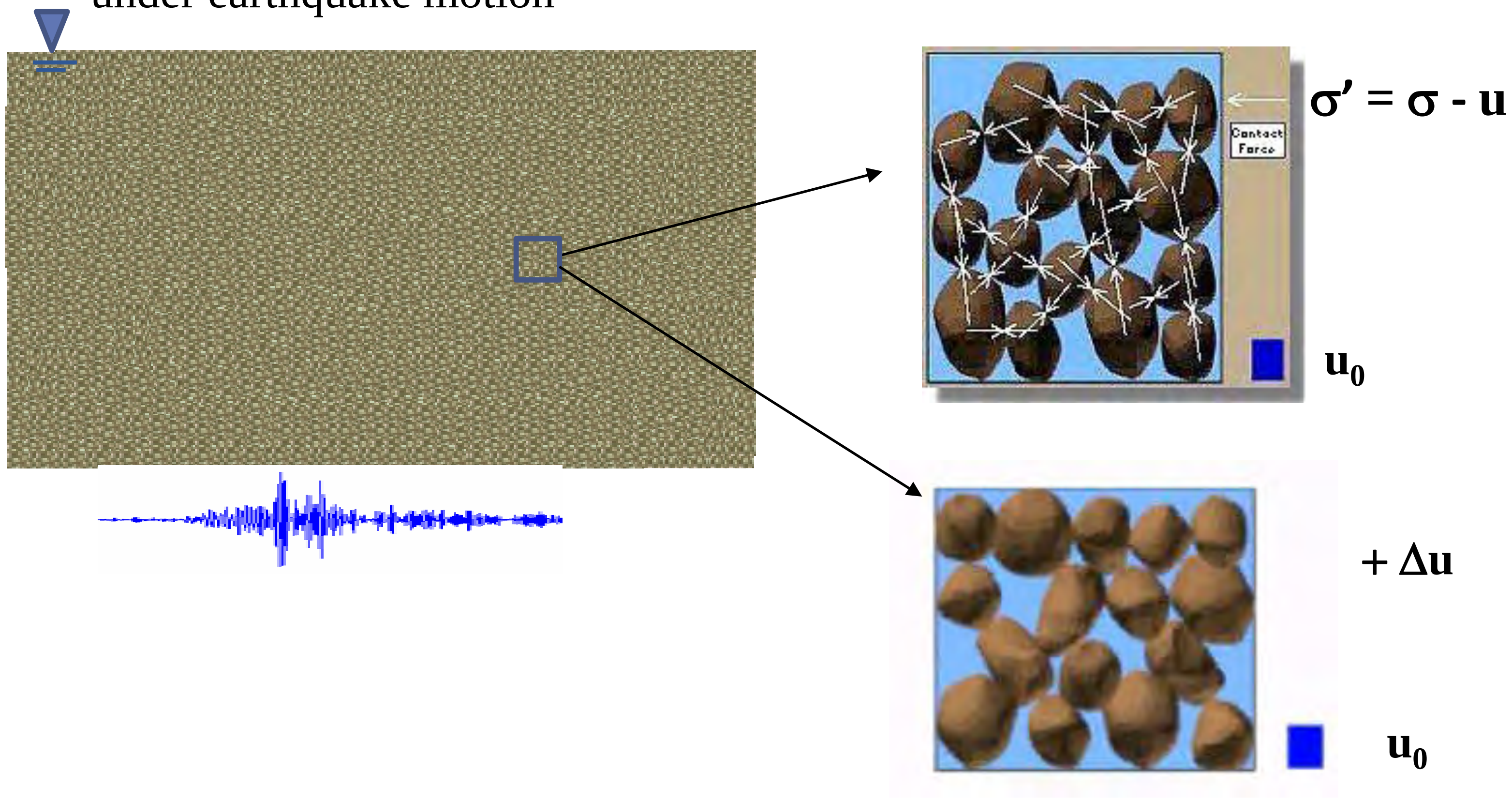
Sand boils near Niigata (photo by K. Steinbrugge,courtesy of EERC, Univ. of California


3 factors required for liquefaction to occur
1) Loose, granular sediment
2) Water saturated sediment
3) Strong shaking
San Francisco, 1906
IRIS

1. Undrained Response of Sands under Dynamic Loads

Mostly occurs in loose saturated sands. However studies show that silty sands and even silts may also liquefy

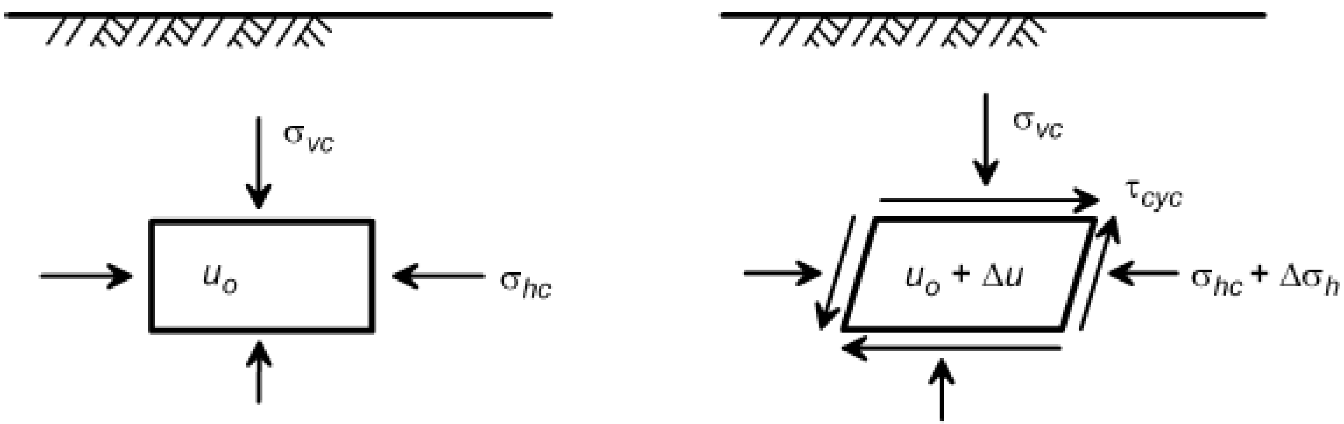
A loose saturaetd sand layer under earthquake motion



1. Undrained Response of Sands under Dynamic Loads

Liquefaction:

Tendency for densification under dynamic loading causes excess pore water pressures to develop under undrained conditions, which in trun reduces the effective stresses.



Consolidation stresses

$\sigma'_{vc} = \sigma_{vc} - u_o$

$\sigma'_{hc} = K_o\, \sigma'_{vc}$

Undrained cyclic loading

$\sigma'_{v} = \sigma_{vc} - (u_o + \Delta u) = \sigma'_{vc} - \Delta u$

$\sigma'_{h} = (\sigma_{hc} + \Delta\sigma_h) - (u_o + \Delta u)$

$\quad = \sigma'_{hc} + (\Delta\sigma_h - \Delta u) = K\, \sigma'_{v}$

1. Undrained Response of Sands under Dynamic Loads

Liquefaction occurrence in sands is affected by:

- Relative Density ,D_R
- Effective confining stress σ'_c, (initial stresses)
- Particle size, shape and gradation (influences volume change behavior)
- Stress history
- Fines content
- Mode of deposition,
- The distance from the epicenter and the magnitude of the EQ

Under Undrained Cyclic Loading:

2 Common Mechanisms:

1. Flow Liquefaction
2. Cyclic Softening: → Cyclic Mobility

   ↘ Cyclic Liquefaction

2 main factors defining liquefaction mechanism:

- Relative density
- Initial state of stress

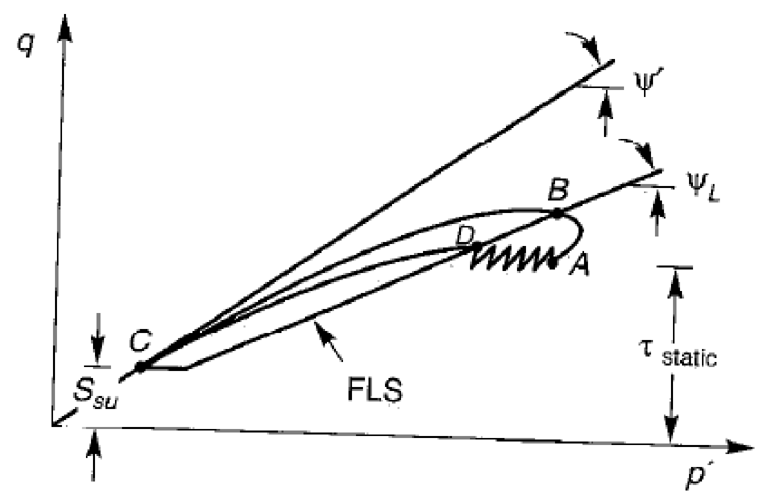
1. Undrained Response of Sands under Dynamic Loads

- **Flow Liquefaction**

  - Initial static shear stresses > Undrained residual shear strength
  - Failure before reaching 0 effective stress condition
  - Usually occurs in loose saturated sand, due to their strain softening behavior under undrained conditions.
  - <u>Failure Types:</u> Loss of bearing capacity (Kavagishi-cho Apt)
    (Foundation Failures)
    Slope Instability, e.g. dam failures

Initiation of flow liquefaction by cyclic loading

S_{su}= steady state shear strength or residual shear strength



Kramer 1996

- **Flow Liquefaction**

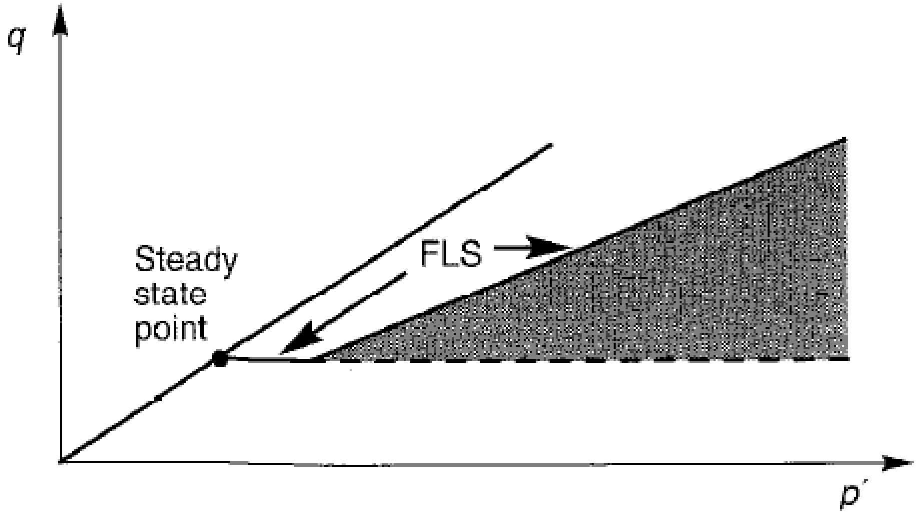


Figure 9.19 Zone of susceptibility to flow liquefaction. If initial conditions fall within the shaded zone, flow liquefaction will occur if an undrained disturbance brings the effective stress path from the point describing the initial conditions to the FLS.

Kramer 1996

Loss of Bearing Capacity, Tilting of Kavagishi-cho Apartment Buildings, 1964 Niiagata

Loss of Bearing Capacity, Tilting of a building during Adapazarı EQ 1999

- **Cyclic Softening**

  - Initial static shear stresses < Undrained residual shear strength
  - Occurs in both loose and dense saturated sands
  - Excessive deformations
  - 2 types: cyclic mobility (no zero shear stress condition)
    cyclic liquefaction (zero shear condition)
  - A special case is *"level ground liquefaction"*, which means without initial shear stresses, σ' ≈ 0 condition is reached
  - <u>Failure Types</u>: Lateral Spreading
    Level Ground Deformations; sand boils,
    settlements, differential transient deformation

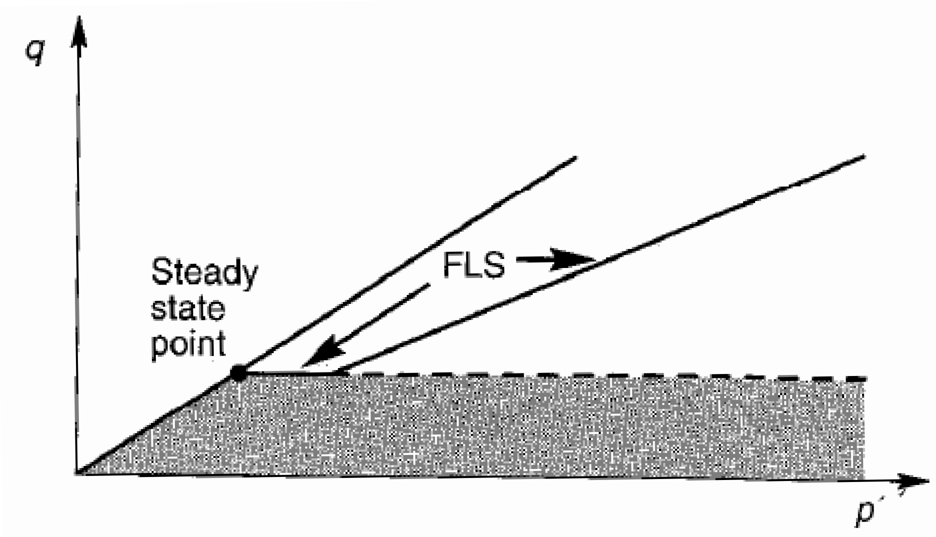


Figure 9.21 Zone of susceptibility to cyclic mobility. If initial conditions plot within shaded zone, cyclic mobility can occur.

Kramer 1996

- **Cyclic Mobility**

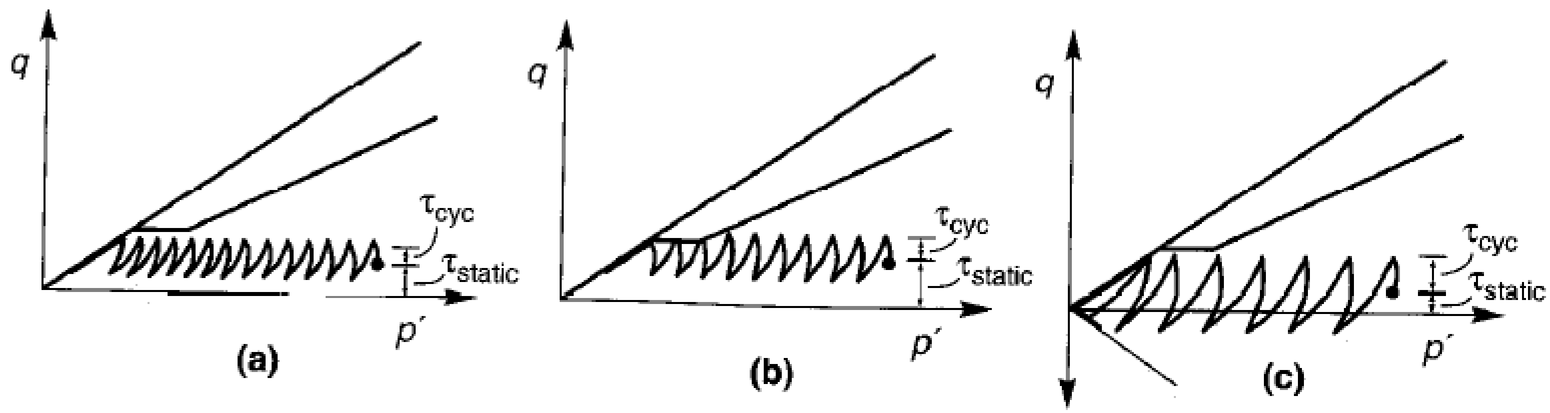
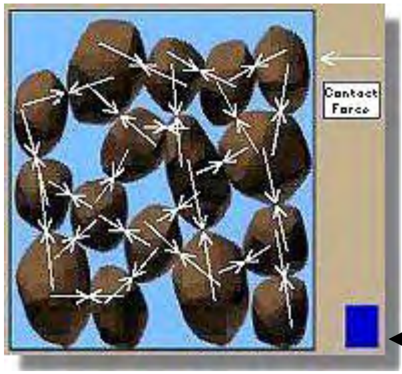


Figure 9.22 Three cases of cyclic mobility: (a) no stress reversal and no exceedance of the steady-state strength; (b) no stress reversal with momentary periods of steady-state strength exceedance; (c) stress reversal with no exceedance of steady-state strength.

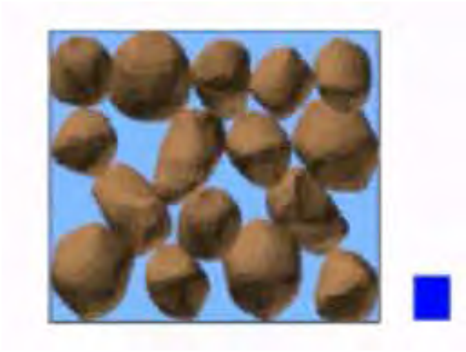
2. Initial Liquefaction

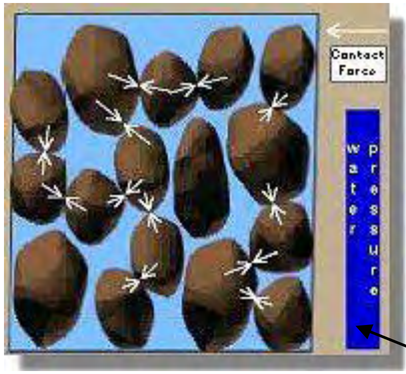
Governing Equation

$\sigma' = \sigma - u$



$\leftarrow u_0$





σ'_0 $+\Delta u$ σ'_f $u_f = u_0 + \Delta u$

Before Liquefaction:

$\sigma'_0 = \sigma - u_0$

After Liquefaction:

$\sigma'_f = \sigma - u_f$

<u>Level Ground (Initial) Liquefaction :</u>

$$\sigma'_f = \sigma - (u_0 + \Delta u) = 0$$

$$\sigma'_f = \sigma'_0 - \Delta u = 0$$

Pore pressure ratio: $r_u = \Delta u / \sigma'_0$

$$\sigma'_f = \sigma'_0 (1 - r_u) = 0$$

$$1 - r_u = 0$$

$$\mathbf{r_u = \Delta u / \sigma'_0 = 1}$$

Pictures: Simple Liquefaction Mechanism (from Jörgen Johansson and Steven Kramer)

2. Initial Liquefaction

Sand boils on the field along Hwy 98 during 1979 El Centro EQ

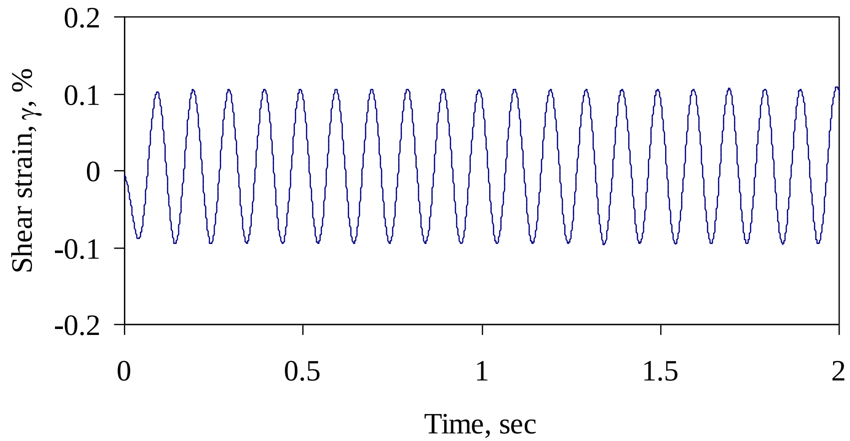
Sand boils near Niigata (photo by K. Steinbrugge,courtesy of EERC, Univ. of California

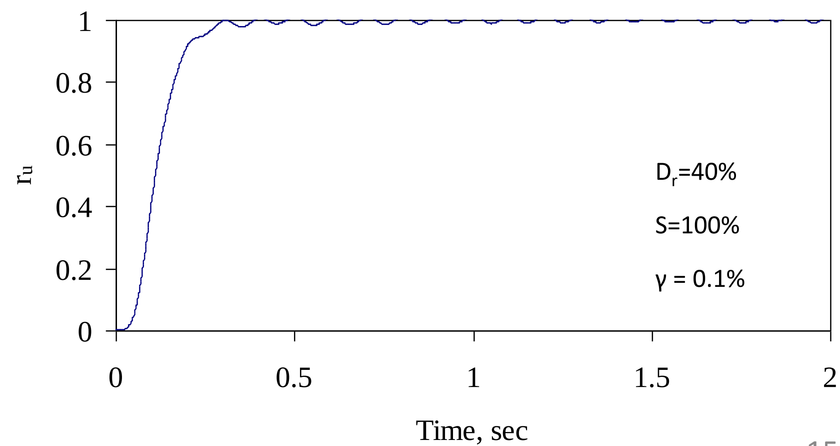
Laboratory Tests

- Cyclic triaxial, cyclic simple shear, torsional shear tests, shaking table and centrifuge tests.
- Initial Liquefaction Criteria :

  Number of cycles to Liquefaction, N_L = N for 5% DA for triaxial

  =N for r_u=1
- Stress-controlled or strain-controlled tests, r_u generation

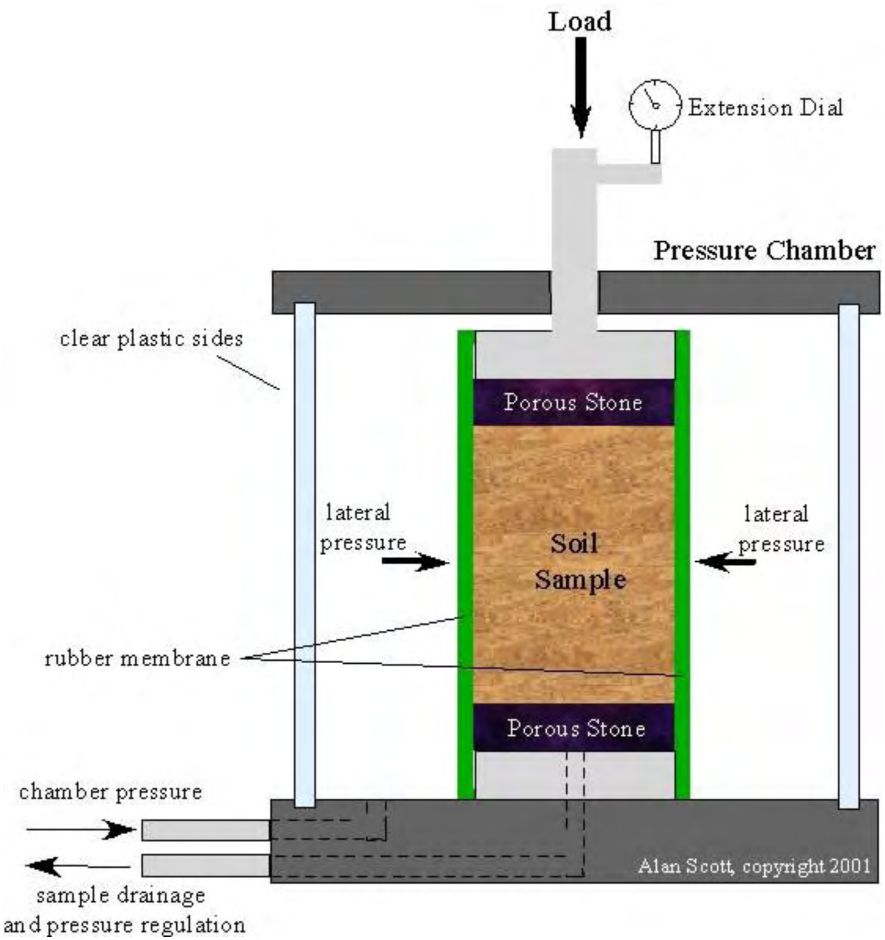


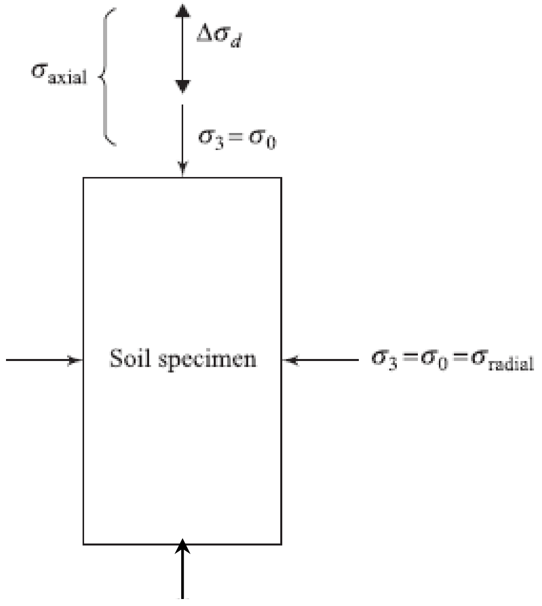


3. Evaluation of Initiation of Liquefaction

Laboratory Tests

Cyclic Triaxial Tests





Laboratory Tests

Cyclic Triaxial Tests

<u>Stress-Controlled</u>

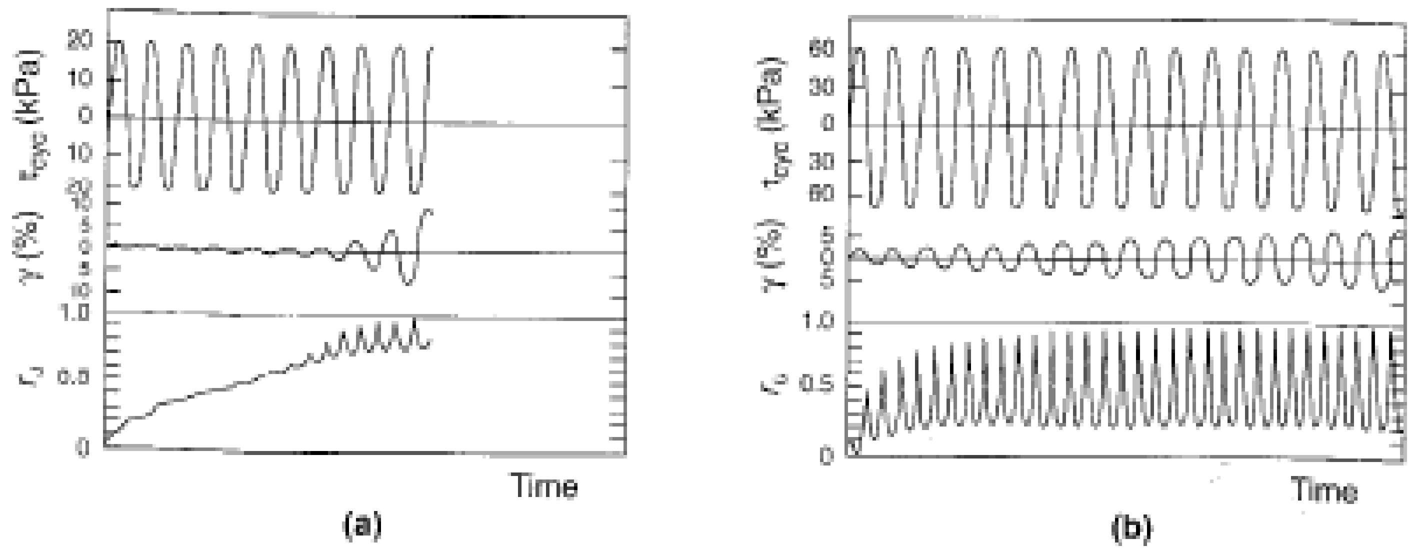


Figure 9.26 Results of torsional shear tests on isotropically consolidated ($\sigma_0' = 98$ kPa) specimens of (a) loose sand (47% relative density) and (b) dense sand (75% relative density). Loose specimen reached initial liquefaction ($r_u = 1.00$) on 10th loading cycle. Desite much higher loading, dense specimen has not quite reached initial liquefaction after 17 cycles. (After Ishihara, 1985; used by permission of Kluwer Academic Publishers.)

Laboratory Tests

Cyclic Triaxial Tests

<u>Stress-Controlled</u>

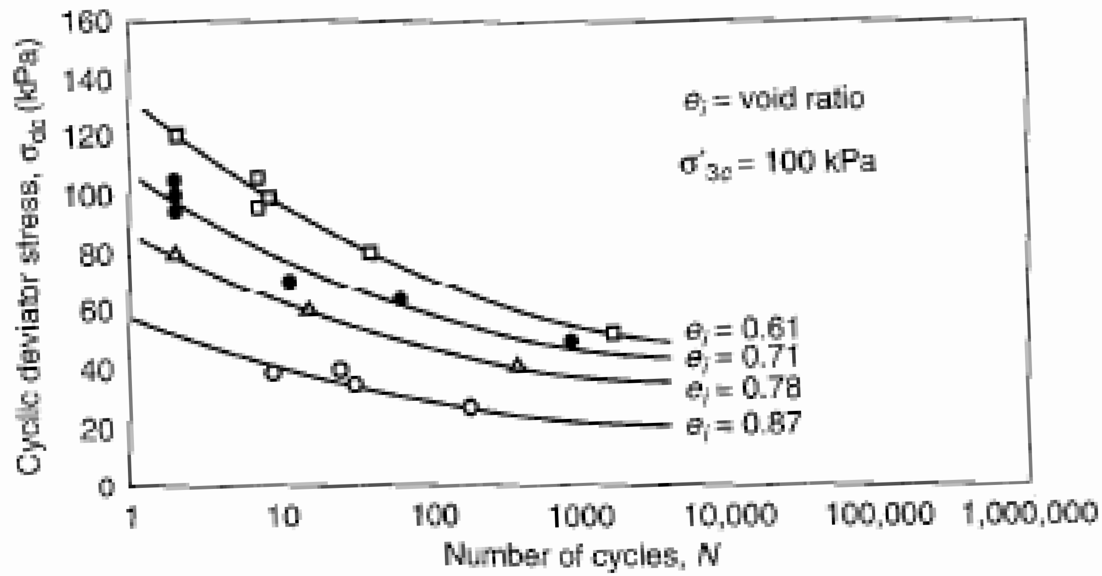
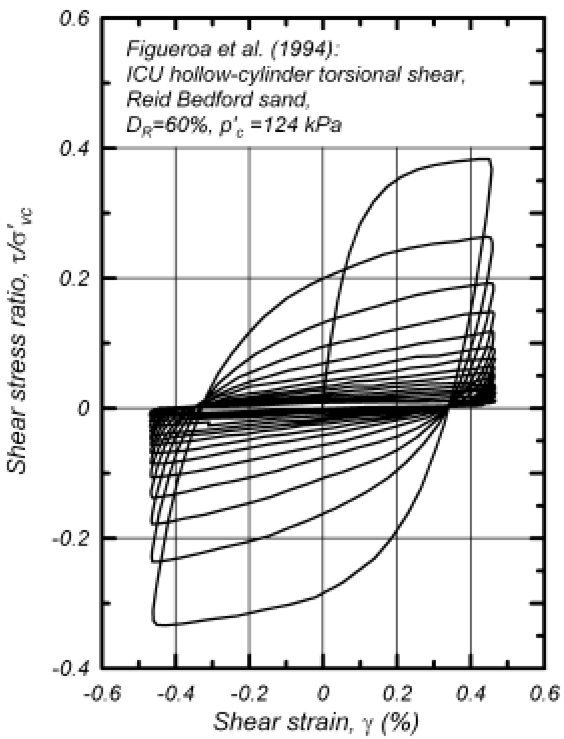


Figure 9.27 Cyclic stresses required to produce initial liquefaction and 20% axial strain in isotropically consolidated Sacramento River Sand triaxial specimens. (After Seed and Lee, 1965.)

Laboratory Tests

Cyclic Triaxial Tests

<u>Strain-Controlled</u>



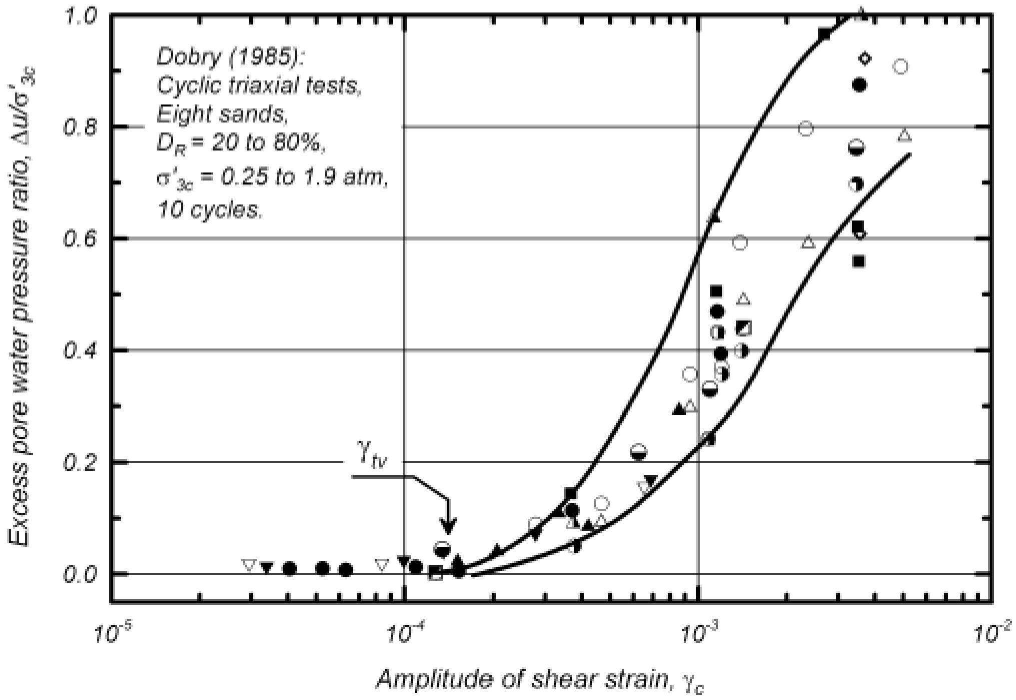
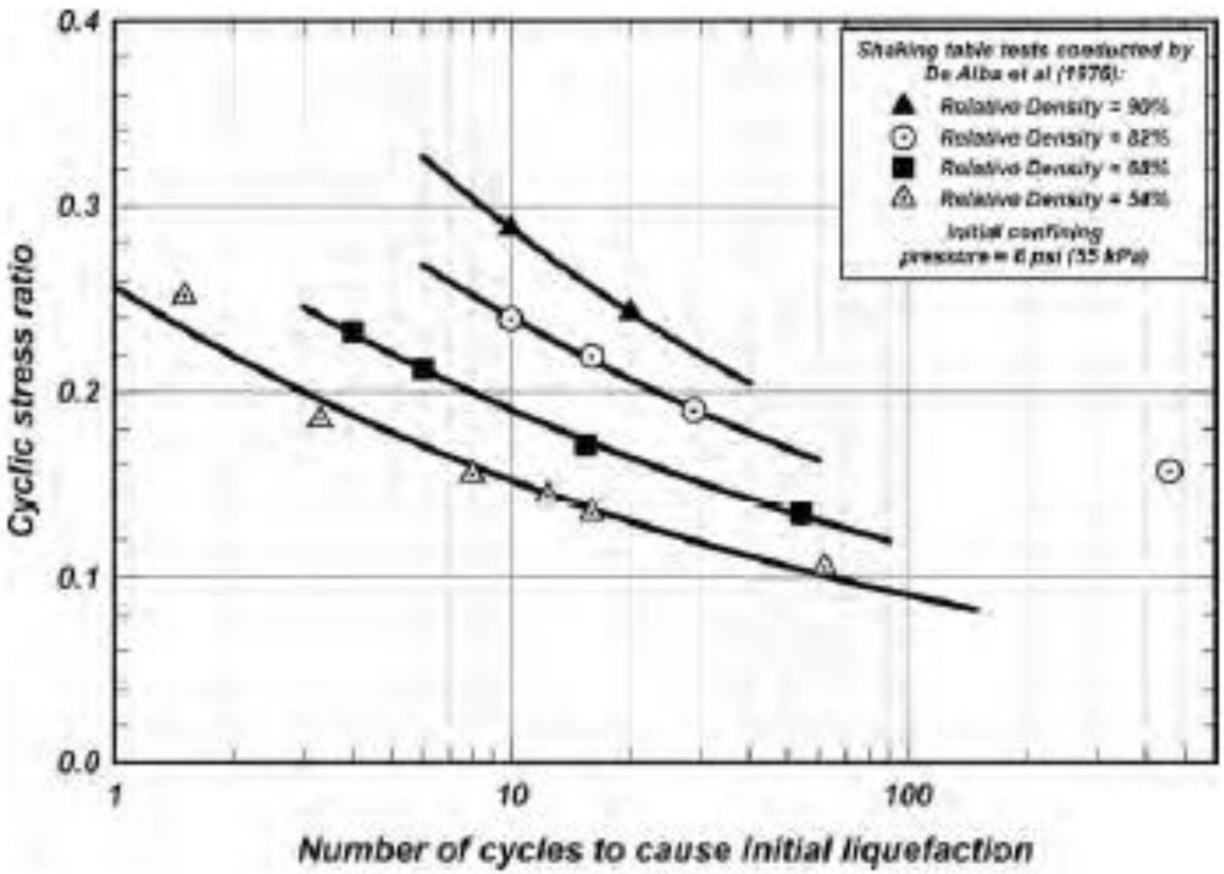


Figure 35. Excess pore water pressure generation versus shear strain amplitude in strain-controlled cyclic undrained triaxial tests on sand (NRC 1985).

Laboratory Tests

Cyclic Simple Shear Tests:

Stress-Controlled



$$CRR = a.N^{-b}$$

b is 0.34 for clean sands

a depends on many factors

De Alba 1975

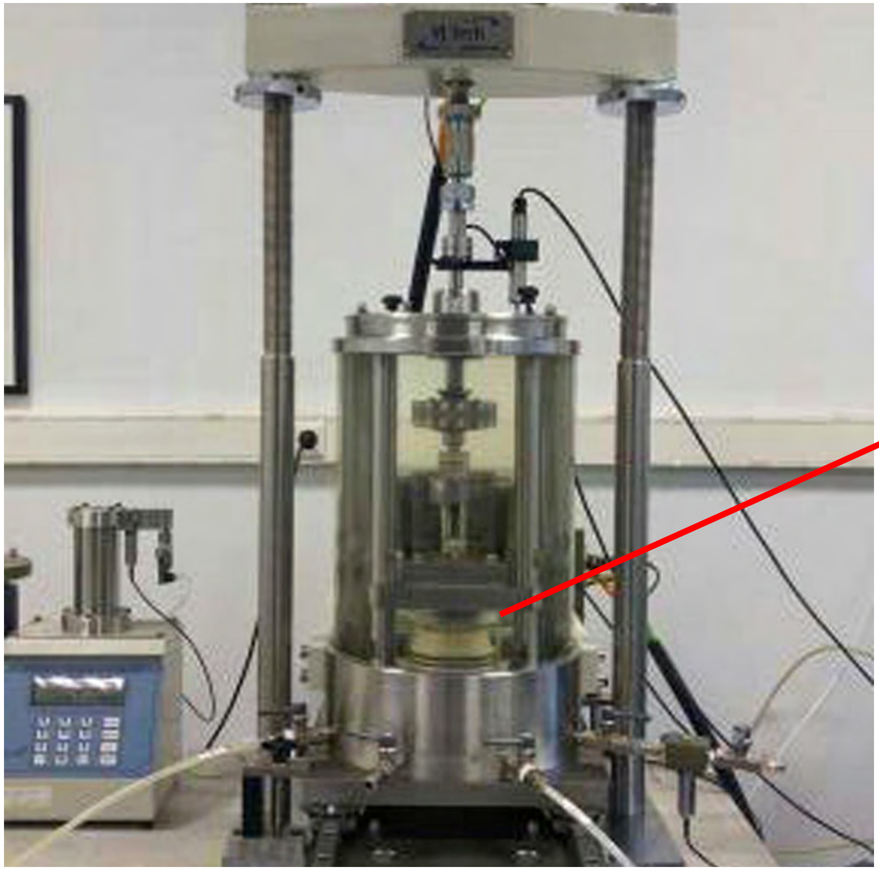
3. Evaluation of Initiation of Liquefaction

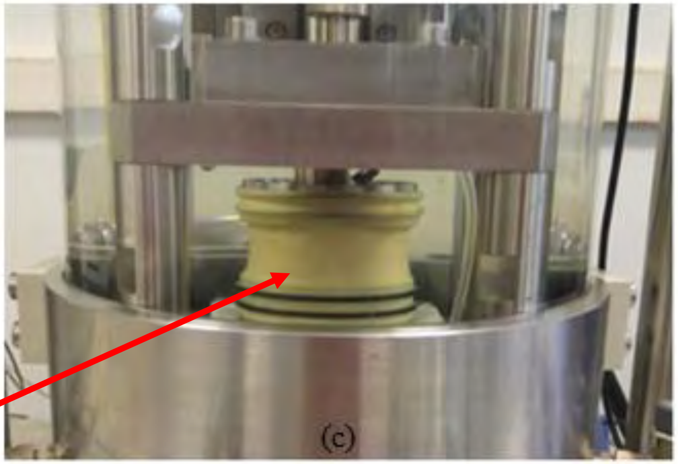
Laboratory Tests

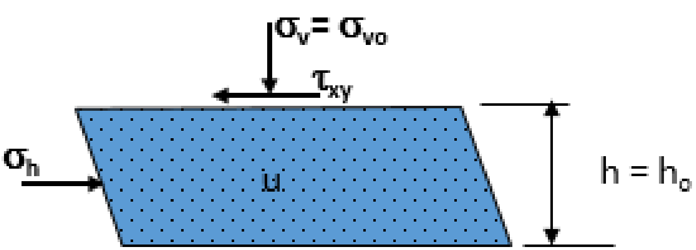
Cyclic Simple Shear Tests:

Dynamic Simple Shear (DSS) with Confining Pressure Testing System

- Simple Shear with flexible membrane, cell pressure and pore pressure measurements

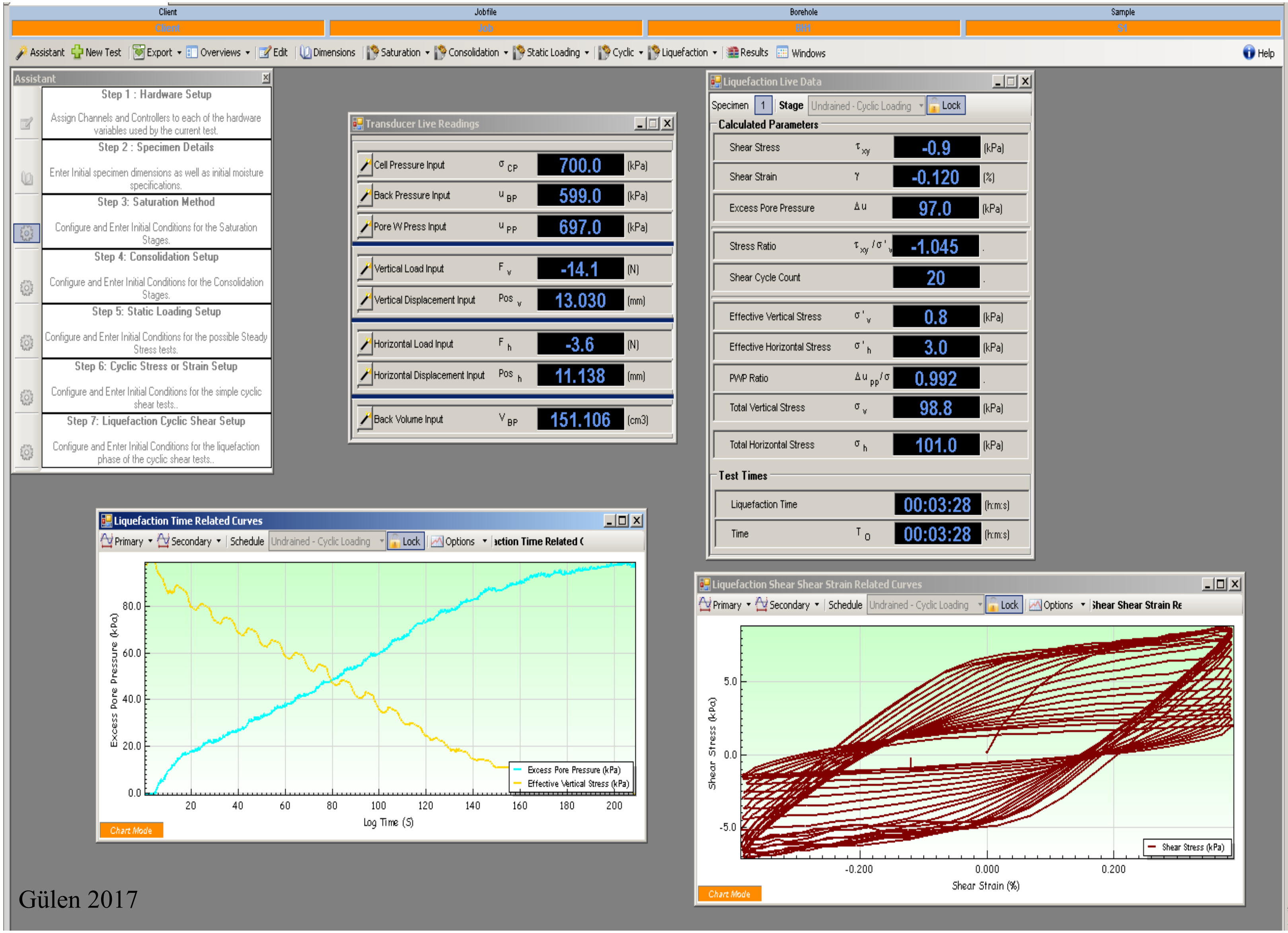






Laboratory Tests

DSS with Confining Pressure Testing System

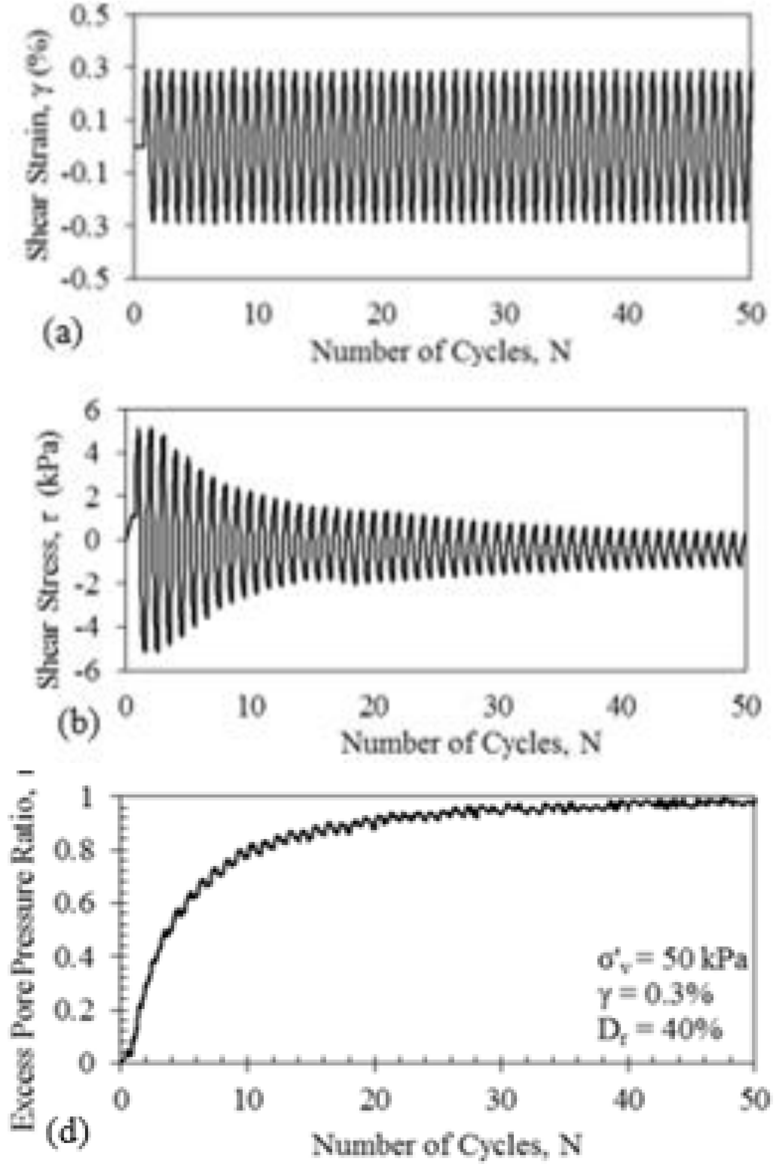


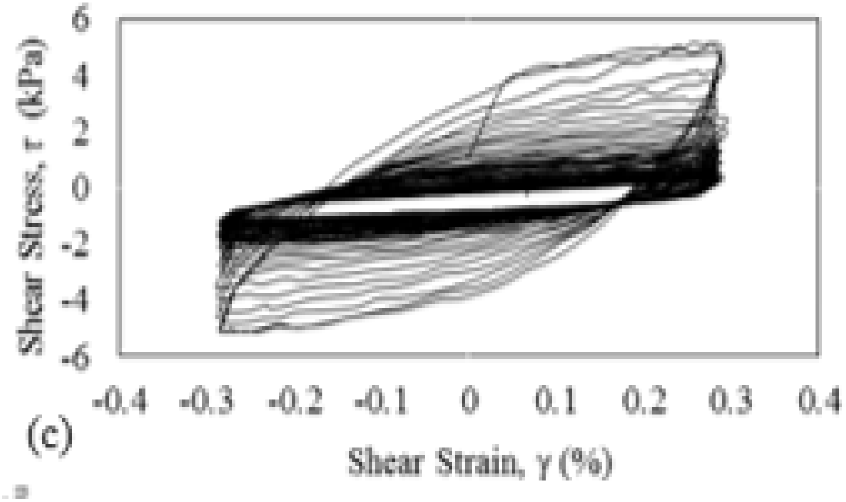
Gülen 2017

Laboratory Tests

DSS with Confining Pressure Testing System

<u>Strain-Controlled</u>

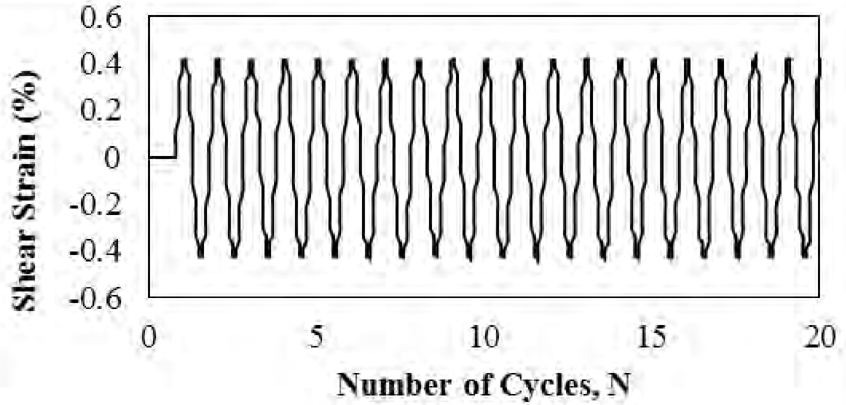


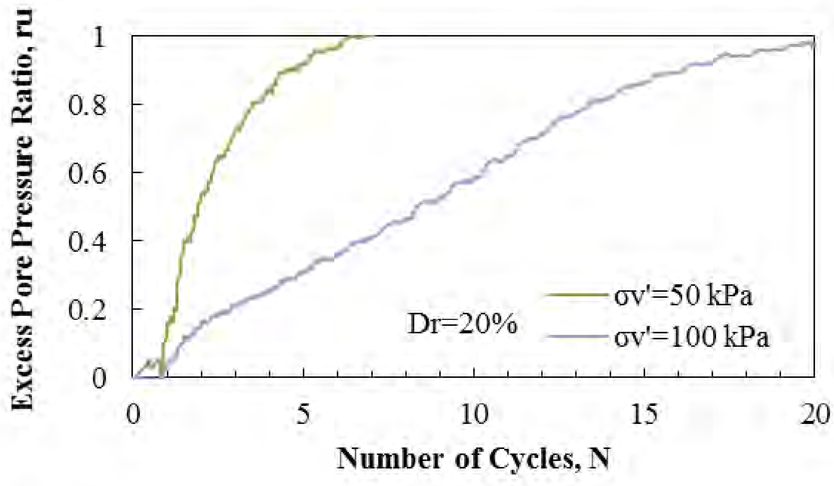


3. Evaluation of Initiation of Liquefaction

Laboratory Tests

DSS with Confining Pressure Testing System



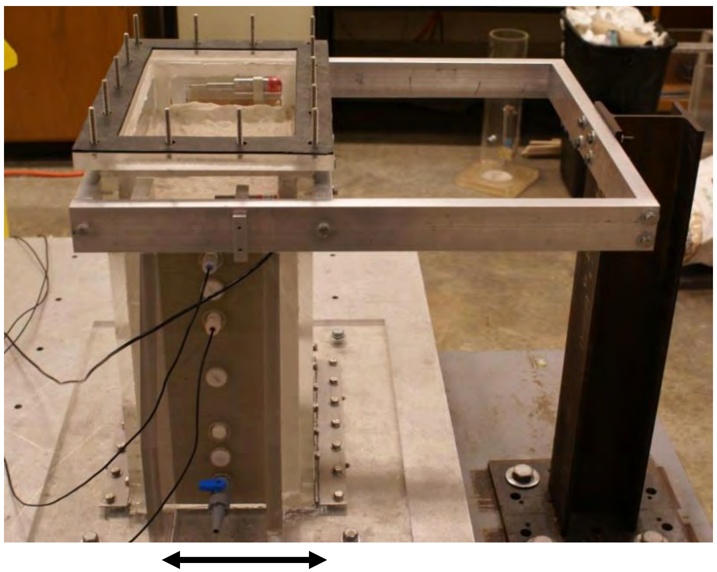


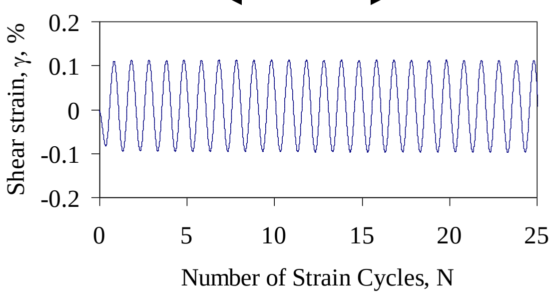
Laboratory Tests:

Model Tests :

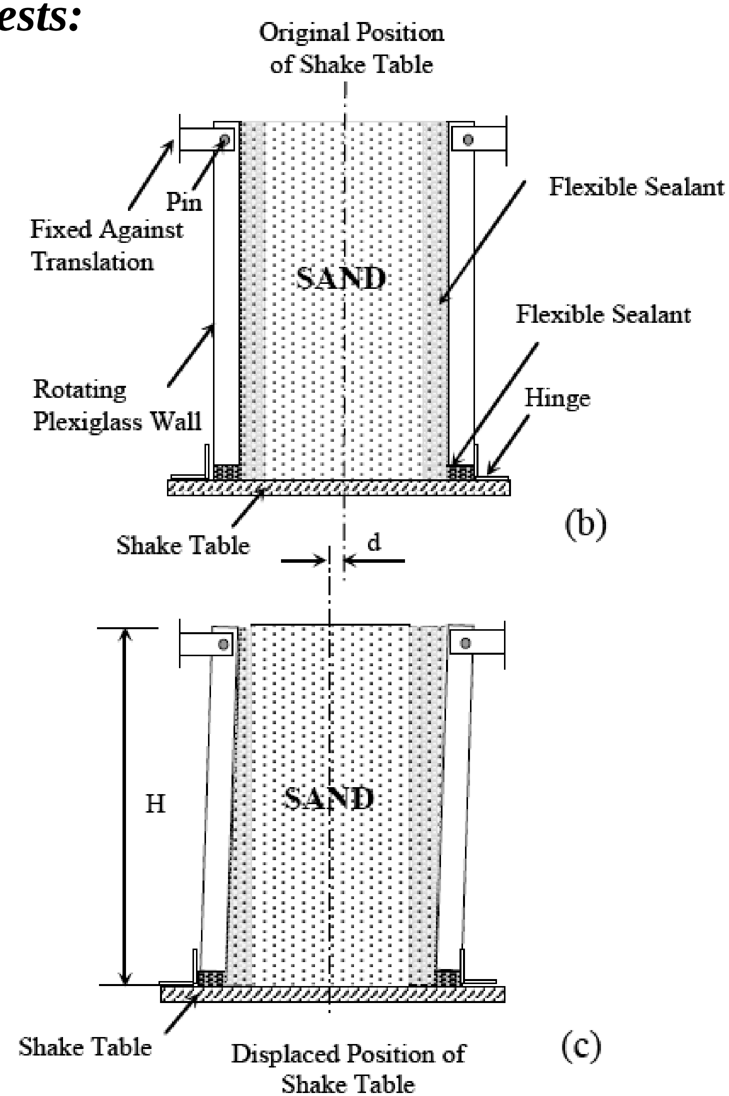
1. Shaking table tests:

Cyclic Simple Shear Liquefaction Box CSSLB





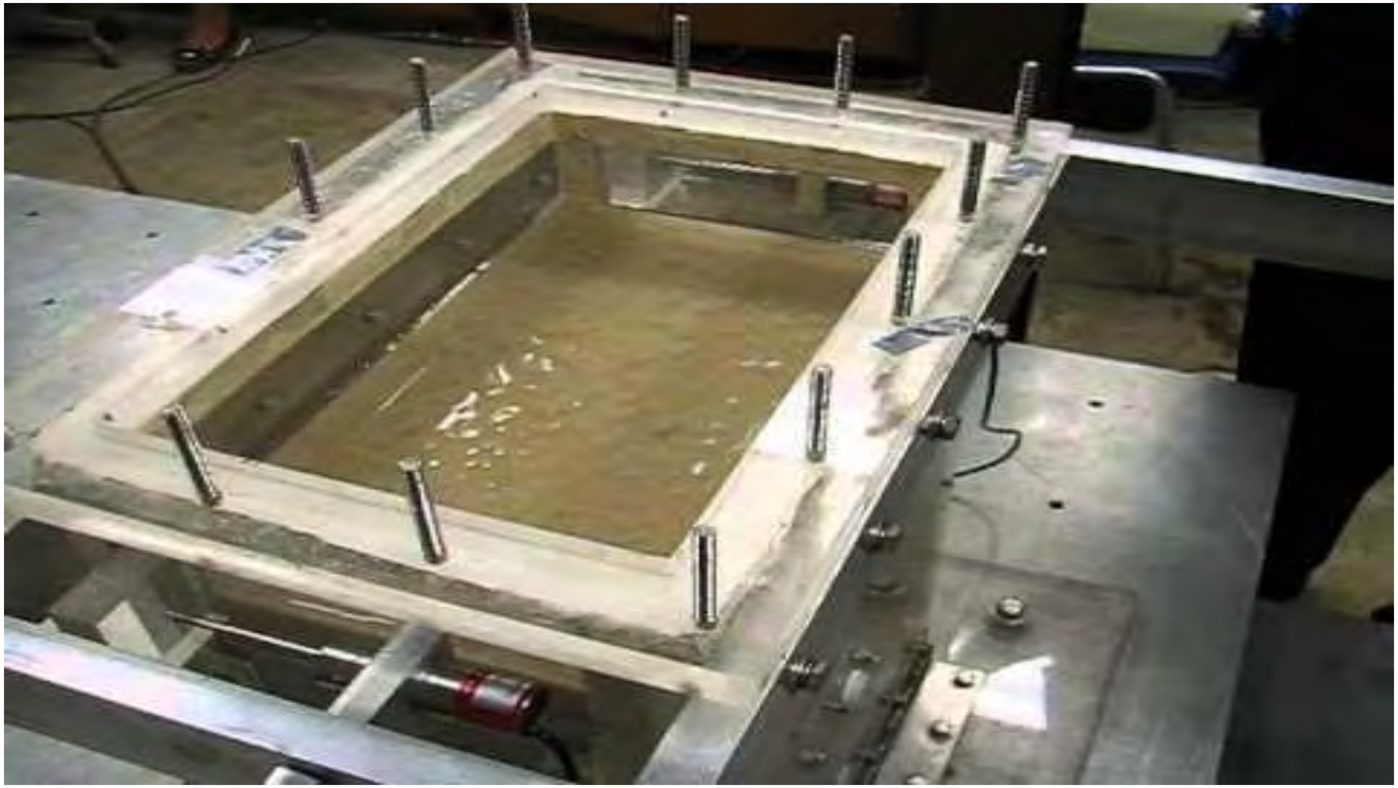
Eseller-Bayat et al. 2013



Laboratory Tests:

Model Tests :

1. ***Shaking table tests:***



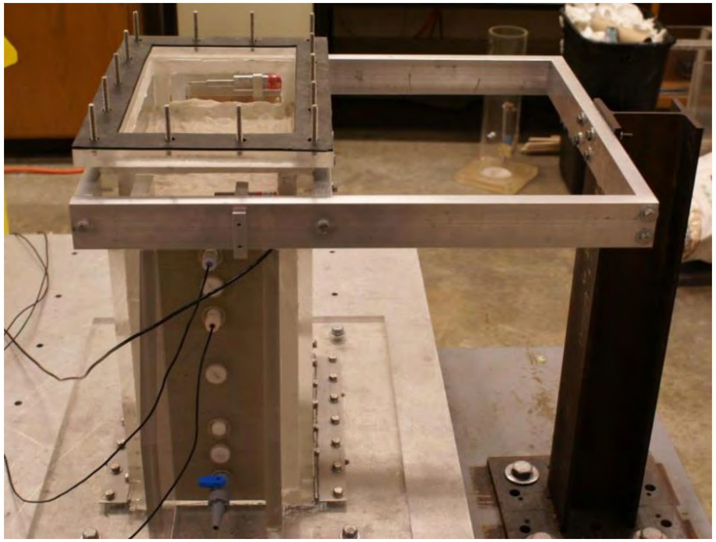
Eseller-Bayat et al. 2013

Laboratory Tests:

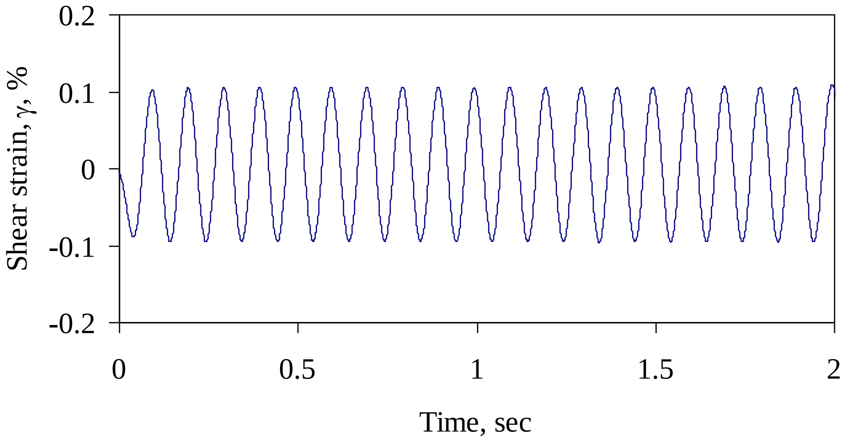
Model Tests :

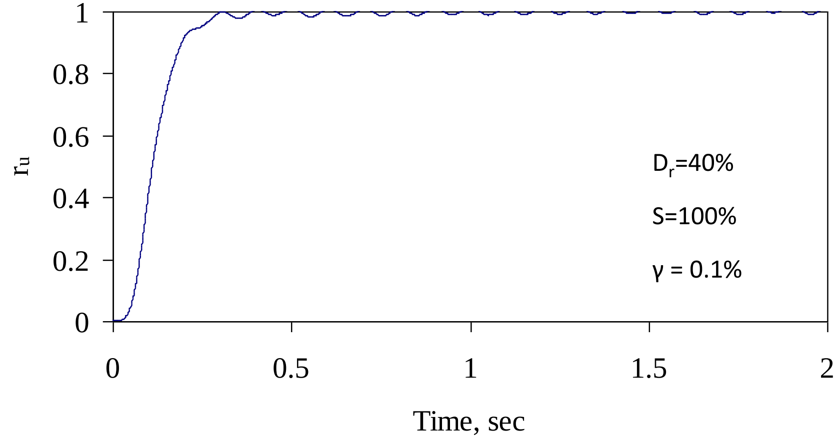
1. Shaking table tests:

Cyclic Simple Shear Liquefaction Box CSSLB



Eseller-Bayat et al. 2013





Laboratory Tests:

Model Tests :

1. Shaking table tests:

Laminar Box

height of six meters.

- The enclosed volume can be filled with a saturated sand or soil to a maximum capacity of 82.5 cubic meters, using a hydraulic slurry pump and distribution system.
- A supply of Ottawa (F-55) sand is stored in three 50 cubic yard outdoor storage containers and may be available for use.
- The structure consists of 39 rings or laminates (I-beam-cross sections) stacked vertically to form a rectangular box. Two base rings are available:
    1. A level ring for assembling and testing a vertical soil column
    2. A sloped ring that allows the testing of a soil column with a 2-degree incline

UB EQ Eng Lab

5 m x 2.75 m

Laboratory Tests:

Model Tests :
2. Centrifuge tests:

- 1/N scale model at a distance r from the axis of a centrifuge
- Rotated at a rotaional speed $\Omega=\sqrt{(N/r)}$ which provides N times the acceleration of gravity.
- Viscous fluid such as glycerin are often used as pore fluids since dissipation of pore water is 10000 faster than the real in the field.

Table 6-2 Scaling Factors for Centrifuge Modeling[a] .

Type of Event	Quantity	$\frac{\text{Model Dimension}}{\text{Prototype Dimension}}$
All events	Stress	1
	Strain	1
	Length	$1/N$
	Mass	$1/N^3$
	Density	1
	Force	$1/N^2$
	Gravity	N
Dynamic events	Time	$1/N$
	Frequency	N
	Acceleration	N
	Strain rate	N
Diffusion events	Time	$1/N^2$
	Strain rate	N^2

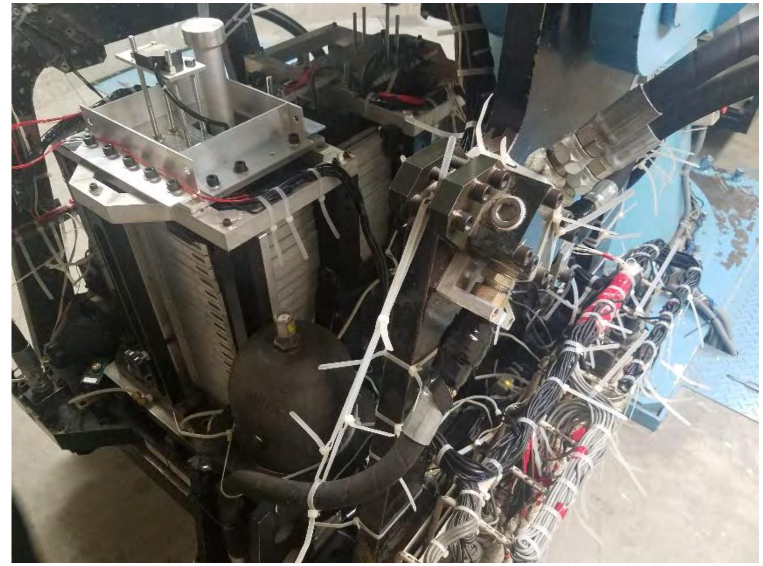
Source: After Kutter and James (1989).
[a]Values are based on the assumption that the same soils and fluid are used in the model and the prototype and that the soil properties are not rate dependent.

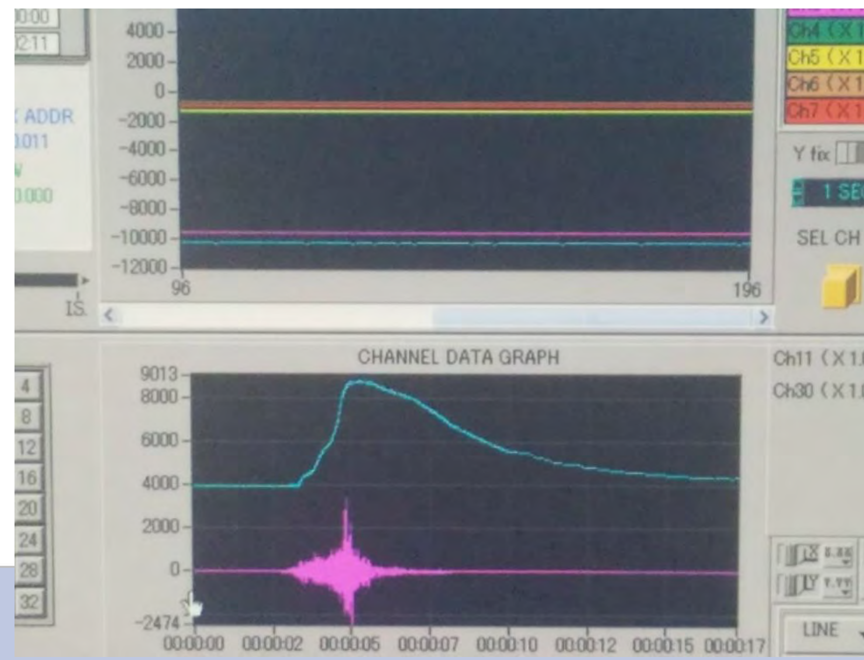
Laboratory Tests:

Model Tests :

2. Centrifuge tests:

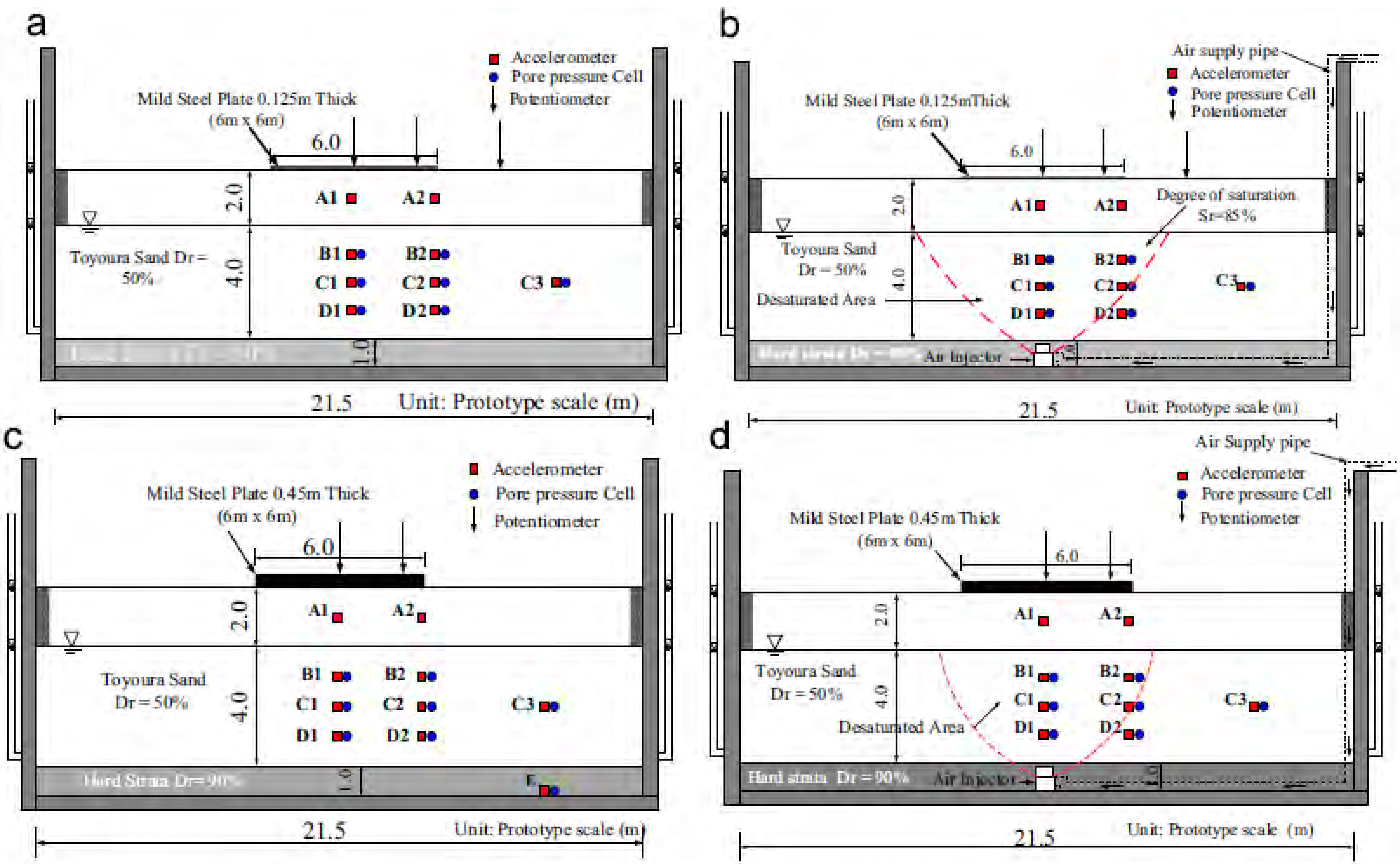
Tokyo Institute of Technology





Laboratory Tests:

Model Tests :
2. Centrifuge tests:

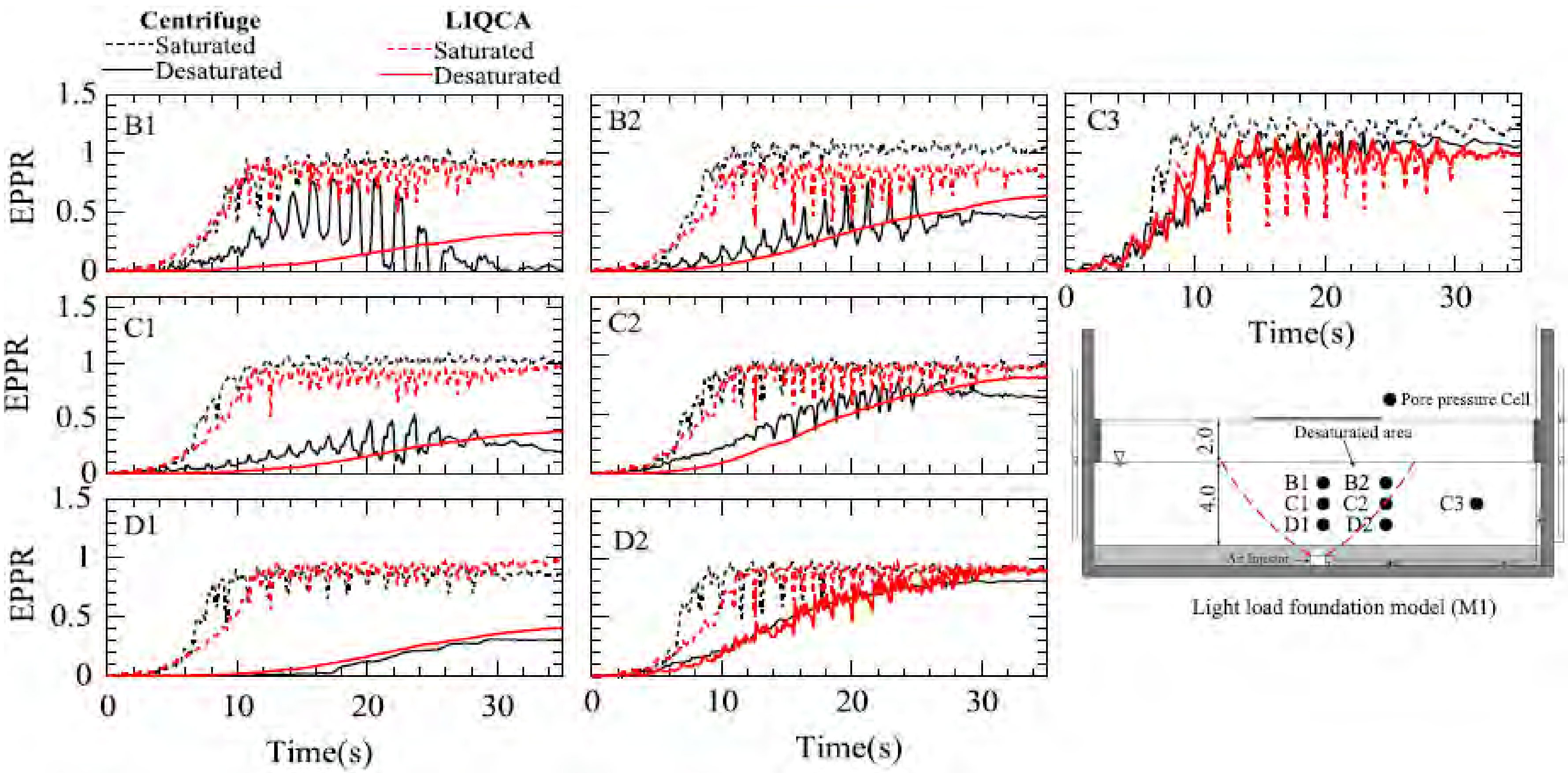


Centrifuge model configurations. (a) M1-1 (b) M1-2 (c) M2-1 and (d) M2-2.

Marasini and Okamura (2015)

Laboratory Tests:

Model Tests :
2. Centrifuge tests:



Marasini and Okamura (2015)

Field Evaluation:

1. Cyclic Stress Approach

$$FS = \frac{CRR}{CSR}$$

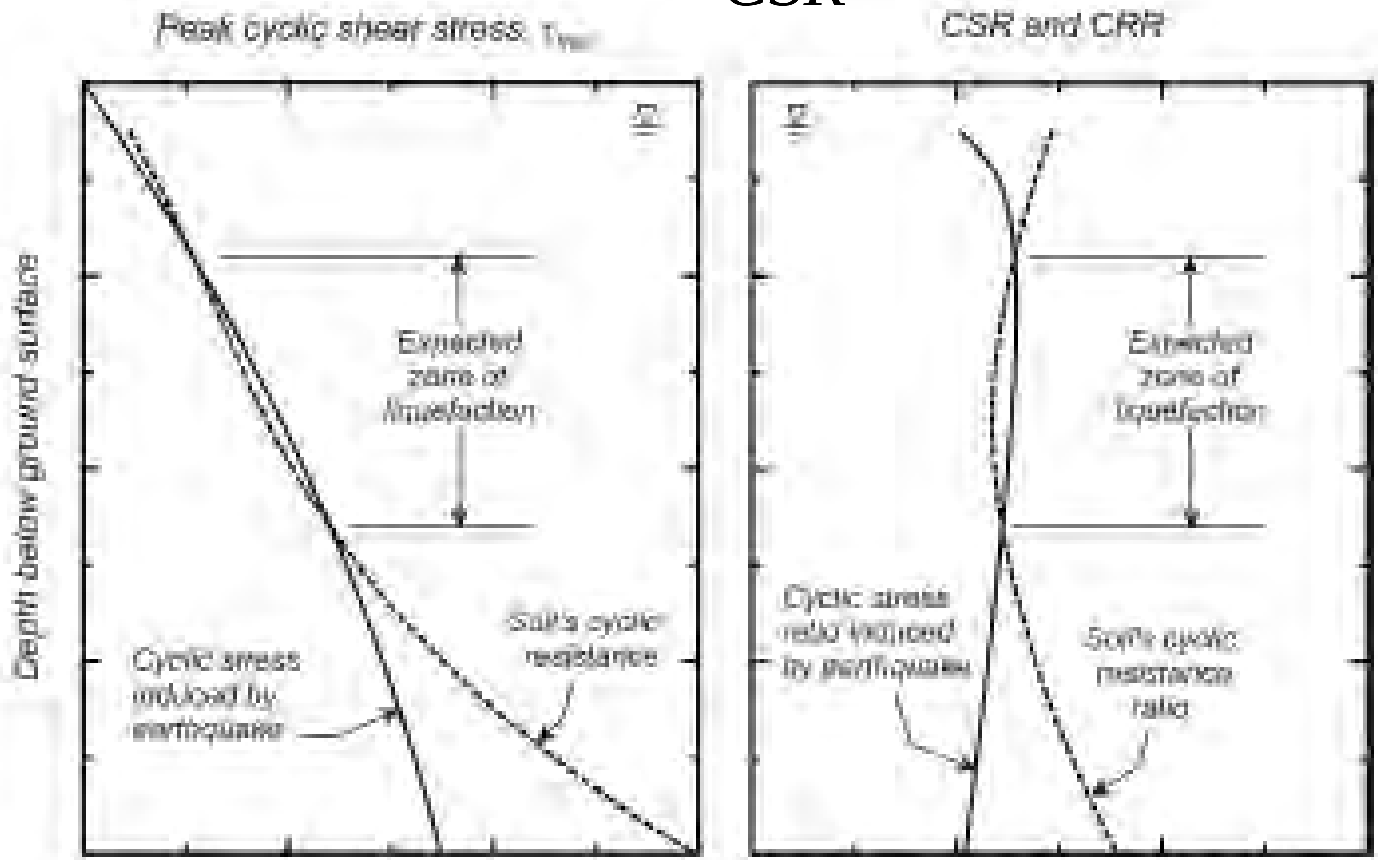


Figure 48. The expected zone of liquefaction is determined by comparing the earthquake-induced cyclic stresses with the cyclic resistances of the soil.

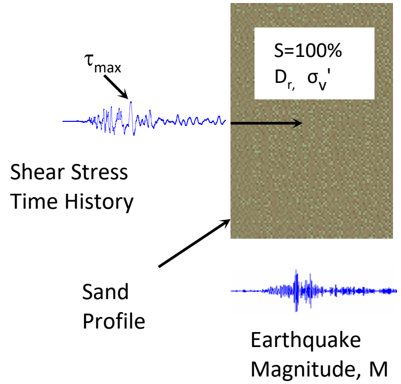
Boulanger and Idriss 2008

Field Evaluation:

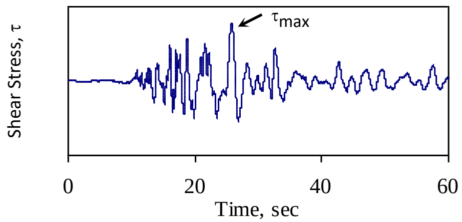
1. Cyclic Stress Approach

Estimation of shear stresses induced at the site for CSR:

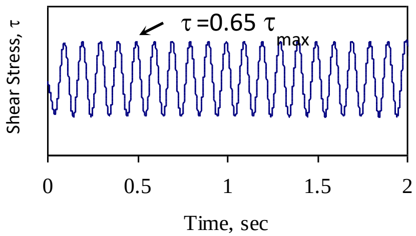
1. ***From ground response analysis***



2. ***Simplified Procedure for Estimating Earthquake Induced Stresses (Seed and Idriss)***



Irregular shear stress record within a soil profile from ground response analysis due to an earthquake with magnitude M



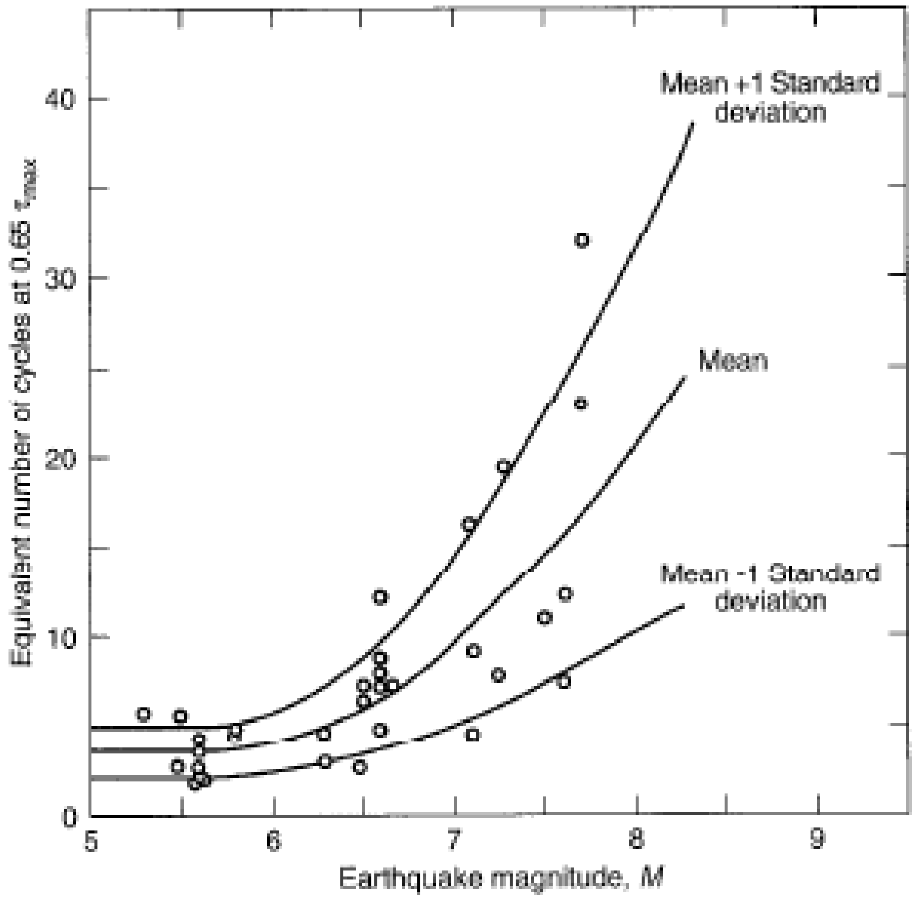
Equivalent uniform stress cycles at 0.65 τ_{max} stress level (for M=7.5: 15 equivalent cycles)

$$\tau_{cyc} = 0.65 \frac{a_{max}}{g} \sigma_v r_d$$

1. Cyclic Stress Approach

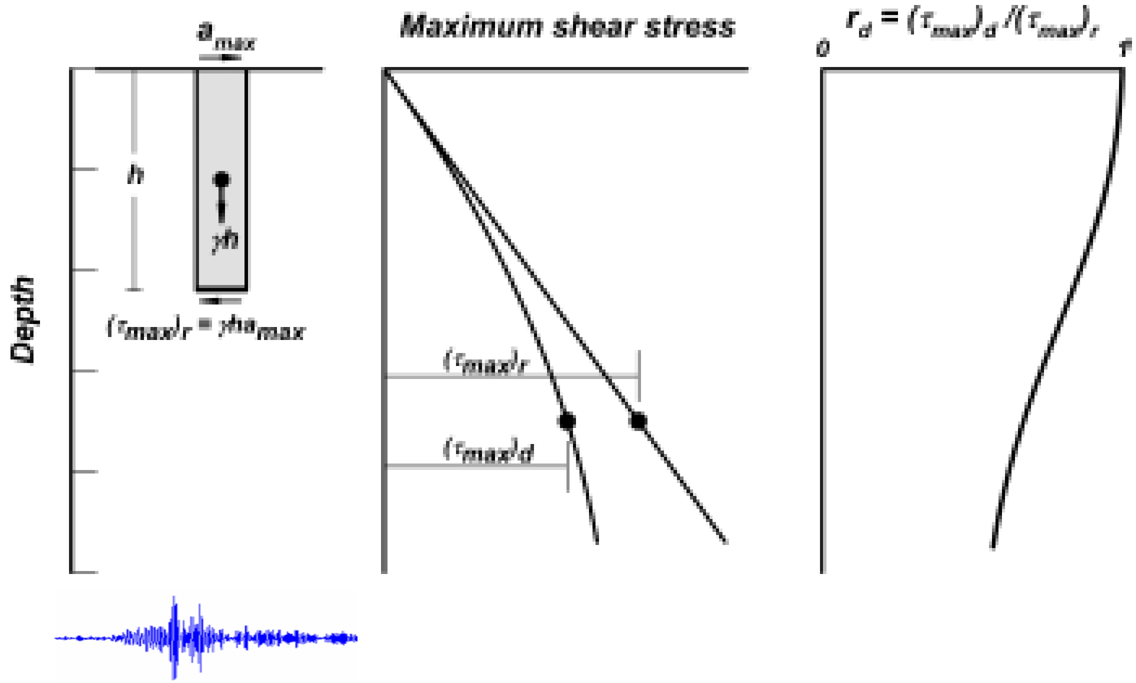
Simplified Procedure for Estimating Earthquake Induced Stresses (Seed and Idriss)

Irregular earthquake-induced loading is characterized by a level of uniform cyclic shear stress that is applied for an equivalent number of cycles



1. Cyclic Stress Approach

Simplified Procedure for Estimating Earthquake Induced Stresses (Seed and Idriss)



$$\tau_{cyc} = 0.65 \frac{a_{max}}{g} \sigma_v r_d$$

$$CSR = \frac{\tau_{cyc}}{\sigma'_v} = 0.65 \frac{a_{max}}{g} \frac{\sigma_v}{\sigma'_v} r_d$$

1. Cyclic Stress Approach

Simplified Procedure for Estimating Earthquake Induced Stresses (Seed and Idriss)

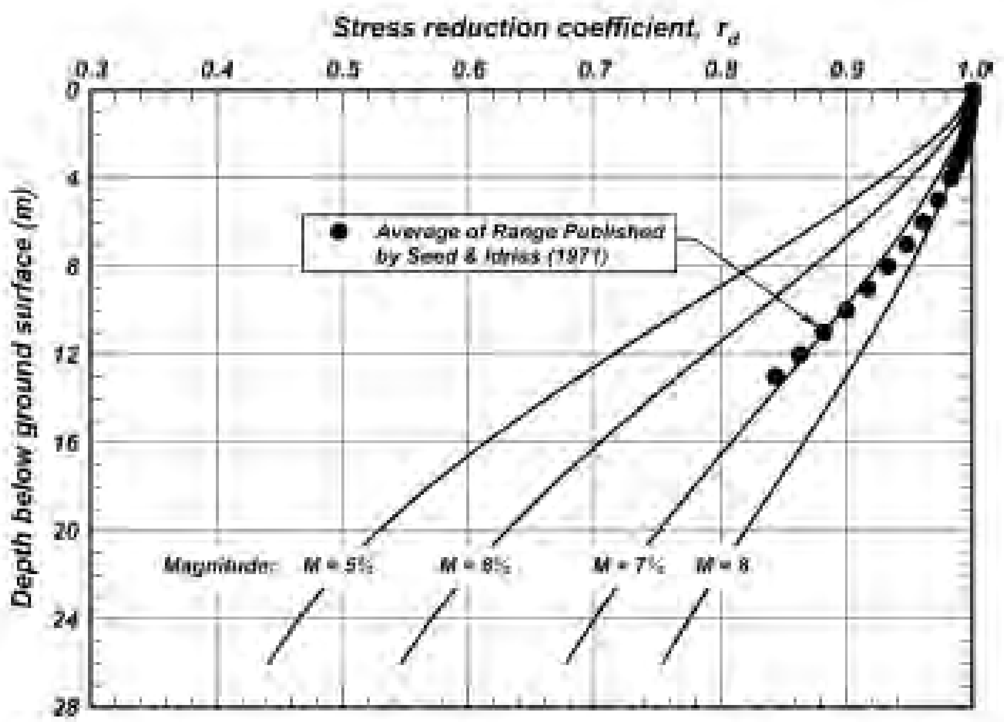


Figure 51. Variations of the stress reduction coefficient r_d with depth and earthquake magnitude (Idriss 1999).

$$\tau_{cyc} = 0.65 \frac{a_{max}}{g} \sigma_v r_d$$

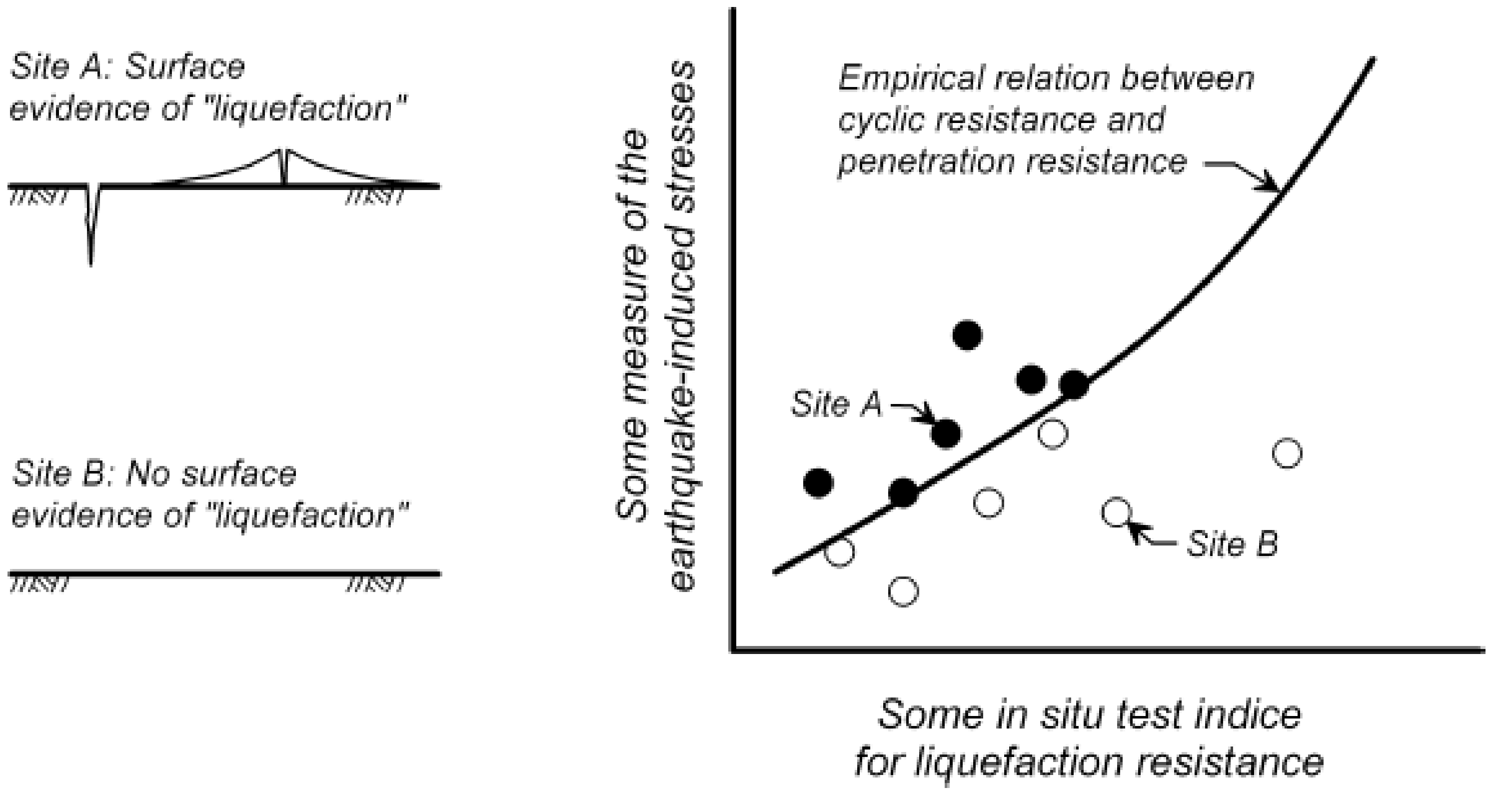
$$r_d = \exp(\alpha(z) + \beta(z)M)$$

$$\alpha(z) = -1.012 - 1.126 \sin\left(\frac{z}{11.73} + 5.133\right)$$

$$\beta(z) = 0.106 + 0.118 \sin\left(\frac{z}{11.28} + 5.142\right)$$

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):



Boulanger and Idriss 2008

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

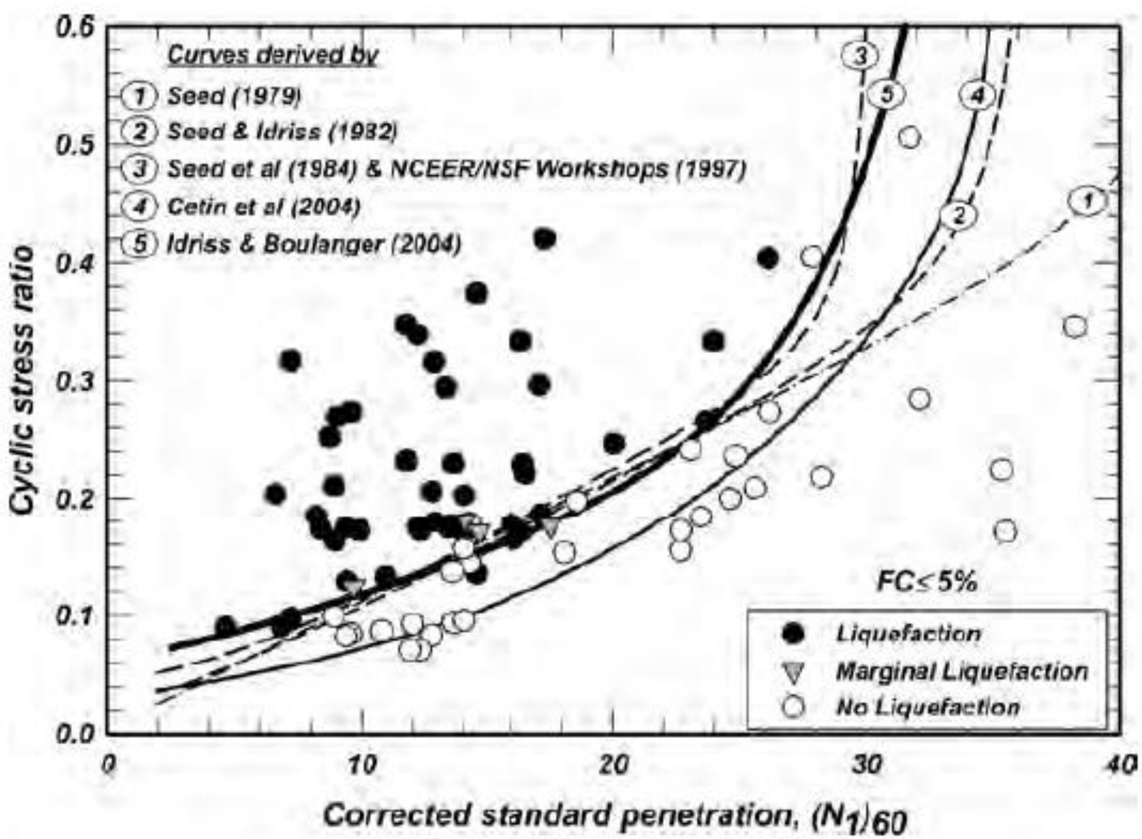


Figure 66. Curves relating the CRR to $(N_1)_{60}$ for clean sands with $M = 7.5$ and $\sigma'_{vc} = 1$ atm.

Field Tests

Standard Penetration Tests

- The sampler is driven into the soil by hammer (**140 lb or 63.5 kg**) blows to the top of the drill rod from a specific height (**0.76 m-30 in.**)
- # of blows (N) for **6 in or 15.24 cm** penetrations are recorded

For 3 "6 in." intervals
N for Last 2 intervals are used to get
"Standard Penetration Number (SPT N)"
=Total of N's for the last 2 intervals
N per12in. or N per feet

STANDARD PENETRATION TEST

http://www.youtube.com/watch?v=yRoBXfrA9sw&feature=related


Hammer 63.5 kg.
hollow stem auger

USGS

http://www.youtube.com/watch?v=2sAXjeL_pAM&feature=related

Field Tests: High Strain Tests

Standard Penetration Tests

- Correction for test conditions:
  - ✓ N_{60} : based on the standard (rope and pulley release and safety hammer type) 60% of the input energy is transferred to the sampler

$$E_r(\%) = \frac{\text{actual hammer energy to the sampler}}{\text{input energy}}$$

input energy=Wh

  - ✓ correction factors needed when test conditions are different than the standard.

$$N_{60} = \frac{N\eta_H\eta_B\eta_S\eta_R}{60}$$

1. Variation of η_H

Country	Hammer type	Hammer release	η_H (%)
Japan	Donut	Free fall	78
	Donut	Rope and pulley	67
United States	Safety	Rope and pulley	60
	Donut	Rope and pulley	45
Argentina	Donut	Rope and pulley	45
China	Donut	Free fall	60
	Donut	Rope and pulley	50

2. Variation of η_B

Diameter mm	Diameter in.	η_B
60–120	2.4–4.7	1
150	6	1.05
200	8	1.15

3. Variation of η_S

Variable	η_S
Standard sampler	1.0
With liner for dense sand and clay	0.8
With liner for loose sand	0.9

4. Variation of η_R

Rod length m	Rod length ft	η_R
>10	>30	1.0
6–10	20–30	0.95
4–6	12–20	0.85
0–4	0–12	0.75

Das 2006

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

Based on in-situ Test:

- Using Standard Penetration Test

$$CRR_{M=7.5,\sigma=1,\alpha=0} = e^{\left\{\frac{(N_1)_{60}}{14.1} + \left(\frac{(N_1)_{60}}{126}\right)^2 - \left(\frac{(N_1)_{60}}{23.6}\right)^3 + \left(\frac{(N_1)_{60}}{25.4}\right)^4 - 2.8\right\}}$$

$$(N_1)_{60} = (N)_{60} \times C_N$$

$$C_N = \left(\frac{P_a}{\sigma'_{vc}}\right)^m$$

$$m = 0.784 - 0.521 \times D_R$$

Modified from Liao and Whitman 1986

Simpler version is m=0.5 and σ'_v is in kg/cm^2(100 kPa)

or correlate D_R to $(N_1)_{60}$ $\quad D_R = \sqrt{\frac{(N_1)_{60}}{46}}$

$$C_N = \left(\frac{P_a}{\sigma'_{vc}}\right)^{0.784 - 0.0768 \times \sqrt{(N_1)_{60}}} \leq 1.7$$

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

Based on in-situ Test:

- Using Cone Penetration Test

$$CRR_{M=7.5,\sigma'_v=1,\alpha=0} = e^{\left\{\frac{q_{c1N}}{540}+\left(\frac{q_{c1N}}{67}\right)^2-\left(\frac{q_{c1N}}{80}\right)^3+\left(\frac{q_{c1N}}{114}\right)^4-3\right\}}$$

$$q_{c1} = (q_c)\times C_N$$

$$C_N = \left(\frac{P_a}{\sigma'_{vc}}\right)^m$$

$$m = 0.784 - 0.521\times D_R$$

$$D_R = 0.478(q_{c1N})^{0.264} - 1.063$$

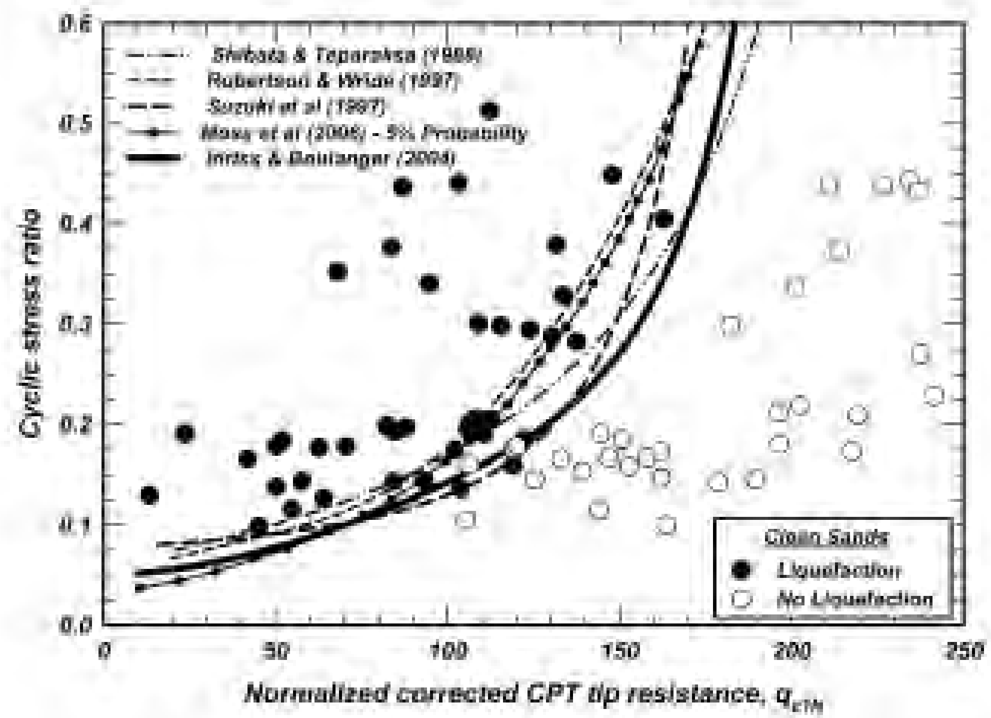


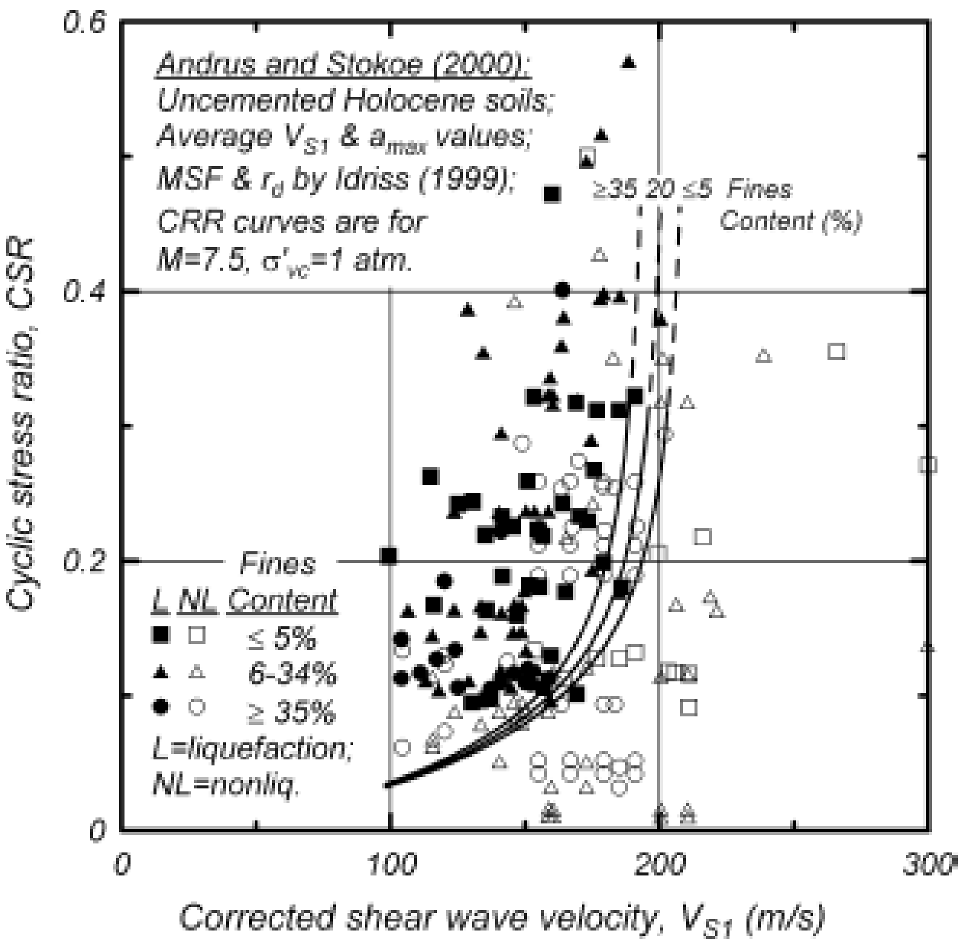
Figure 67. Curves relating the CRR to q_{c1N} for clean sands with M = 7.5 and σ'_{vc} = 1 atm.

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

<u>Based on in-situ Test:</u>

- Using V_s Shear wave velocity measurments



V_{s1} is normalized to 1 tons/ft^2 (96kPa)

$$V_{s1} = \frac{V_s}{(\sigma'_{vo})^n}$$

n = 3 by Tokimatsu et al. 1991

= 4 by Finn 1991

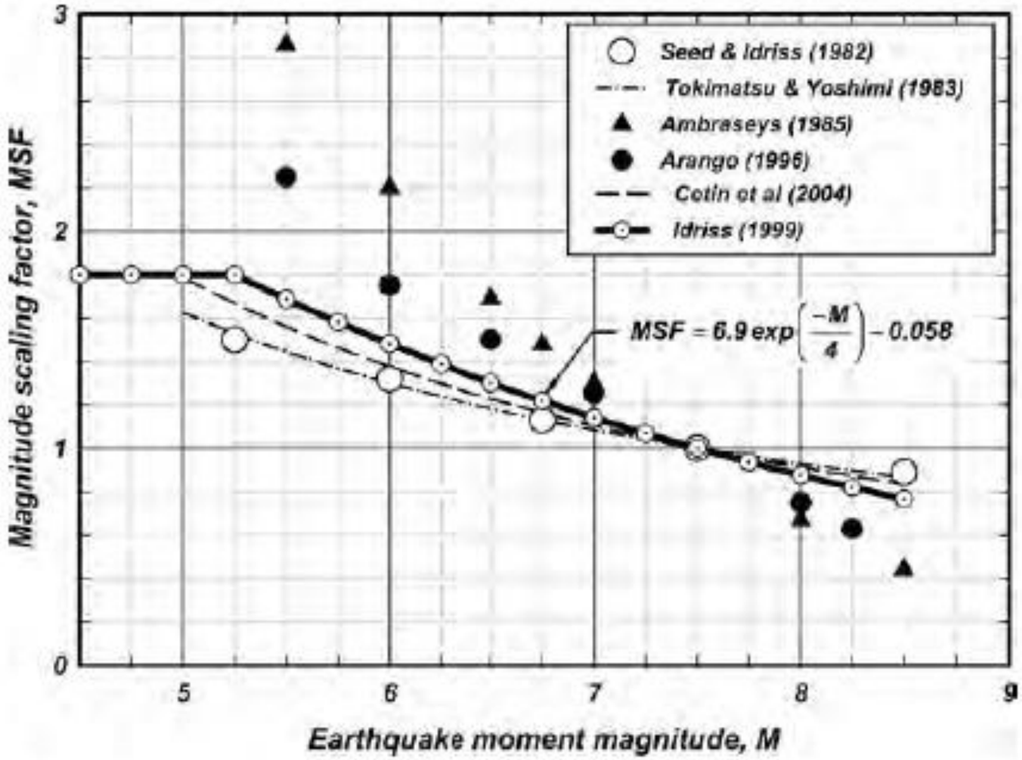
σ'_{vo} is in tons/ft^2

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

$$CRR = CRR_{M=7.5} \times MSF \times K_{\sigma} \times K_{\alpha}$$

Magnitude Scaling Factor MSF:



$$MSF = \frac{CRR_M}{CRR_{M=7.5}}$$

$$MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058$$

$$MSF \le 1.8$$

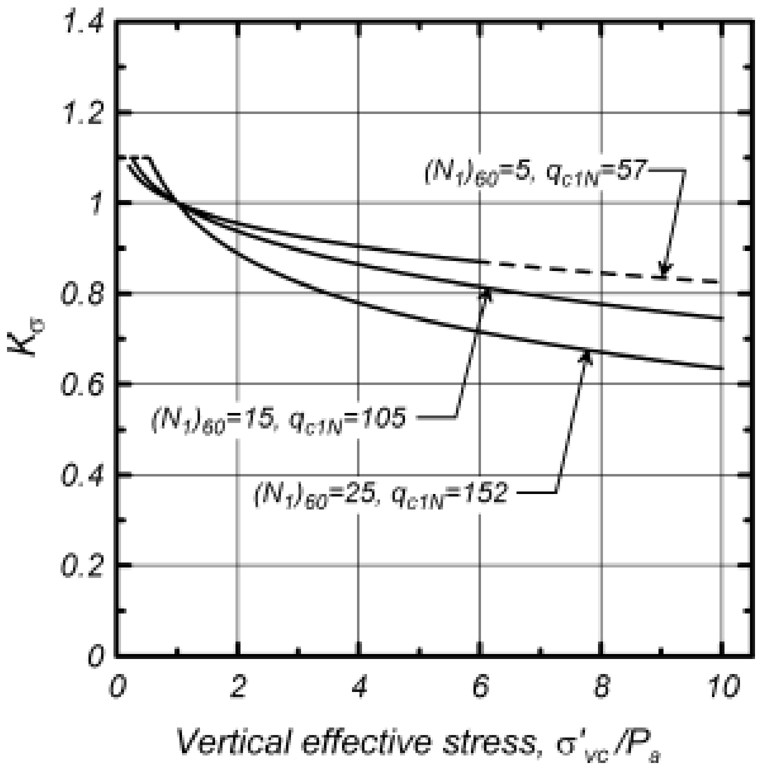
Boulanger and Idriss 2008

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

$$CRR = CRR_{M=7.5} \times MSF \times K_\sigma \times K_\alpha$$

Correction for Overburden stress K_σ:



$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{vc}}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{18.9 - 17.3 D_R} \le 0.3$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}} \le 0.3$$

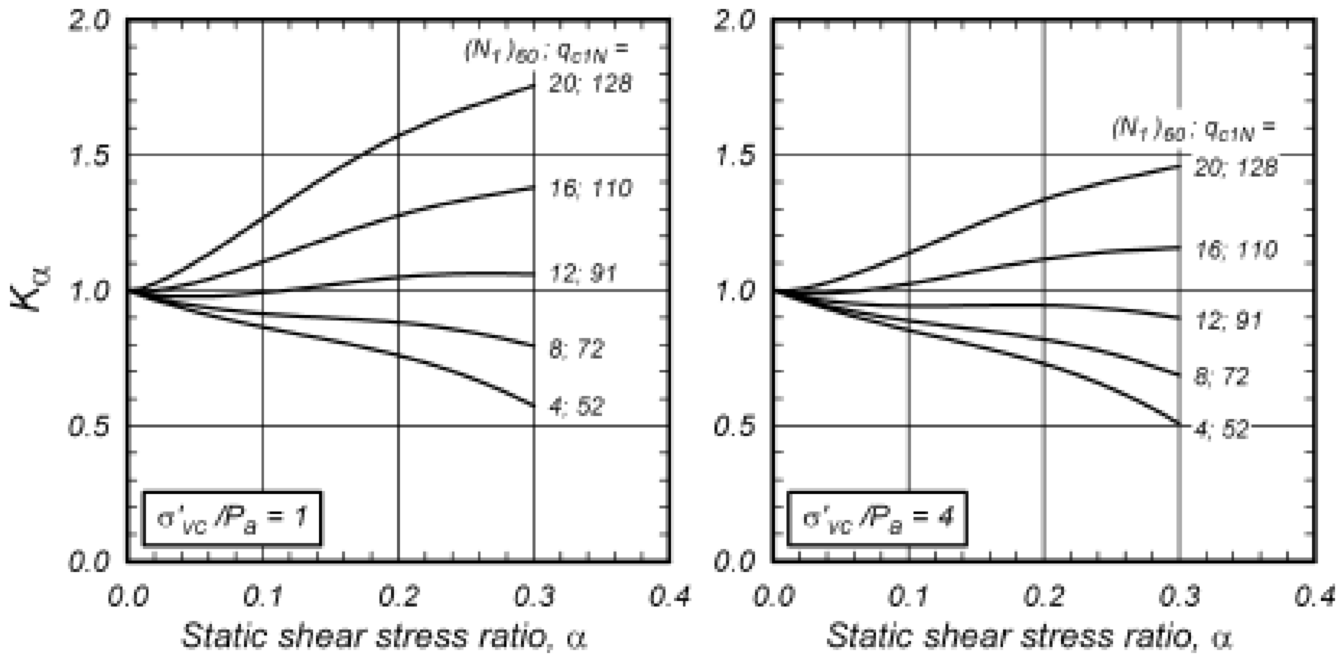
$$C_\sigma = \frac{1}{37.3 - 8.27 (q_{c1N})^{0.264}} \le 0.3$$

Boulanger and Idriss 2008

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

$$CRR = CRR_{M=7.5} \times MSF \times K_{\sigma} \times K_{\alpha}$$



$$\alpha = \frac{\tau_{static}}{\sigma'_{vo}}$$

Figure 65. Variations of K_α with SPT and CPT penetration resistances at effective overburden stresses of 1 and 4 atm.

Boulanger and Idriss 2008

1. Cyclic Stress Approach

Estimation of liquefaction resistance of the soil (CRR):

$$CRR = CRR_{M=7.5} \times MSF \times K_{\sigma} \times K_{\alpha}$$

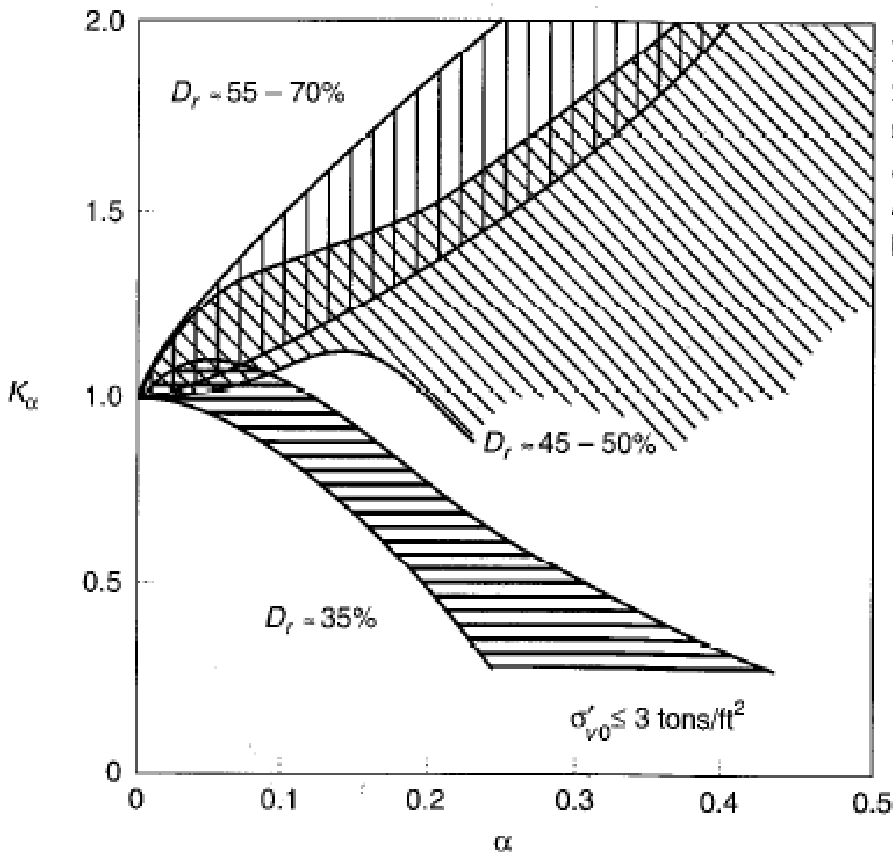


Figure 9.32 Variation of correction factor, K_α, with initial shear/normal stress ratio. (After Seed and Harder, 1990. *H. Bolton Seed Memorial Symposium Proceedings*, Vol. 2, p. 364. Used by permission of BiTech Publishers, Ltd.)

Kramer 1996

1. **Cyclic Stress Approach**

$$FS = \frac{CRR}{CSR} or$$

$$= \frac{\tau_{cyc,L}}{\tau_{cyc}} = \frac{\text{cyclic shear stress required to cause liquefaction}}{\text{equivalent cyclic shear stress induced by earthquake}}$$

$$CSR = \frac{\tau_{cyc}}{\sigma'_{vo}} = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d$$

$$CRR = CRR_{M=7.5} \times MSF \times K_{\sigma} \times K_{\alpha}$$

1. Cyclic Stress Approach

Estimation of liquefaction resistance of silty sands (CRR):

Based on in-situ Test:

- Using Standard Penetration Test

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

$$\Delta(N_1)_{60} = e^{\left(1.63+\frac{9.7}{FC+0.01}-\left(\frac{15.7}{FC+0.01}\right)^2\right)}$$

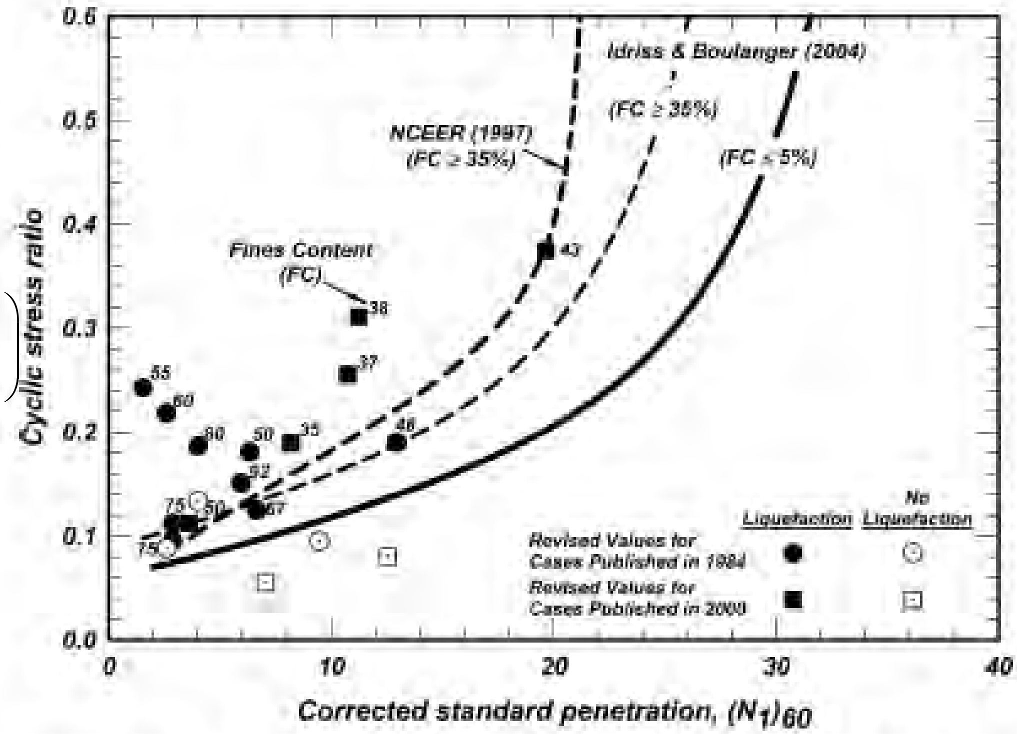


Figure 71. SPT case histories of cohesionless soils with FC ≥ 35%, the NCEER/NSF workshop curve (Youd et al. 2001), and the recommended curves for both clean sands and for FC ≥ 35% for M = 7.5 and $\sigma'_{vc} = 1$ atm.

Boulanger and Idriss 2008

1. Cyclic Stress Approach

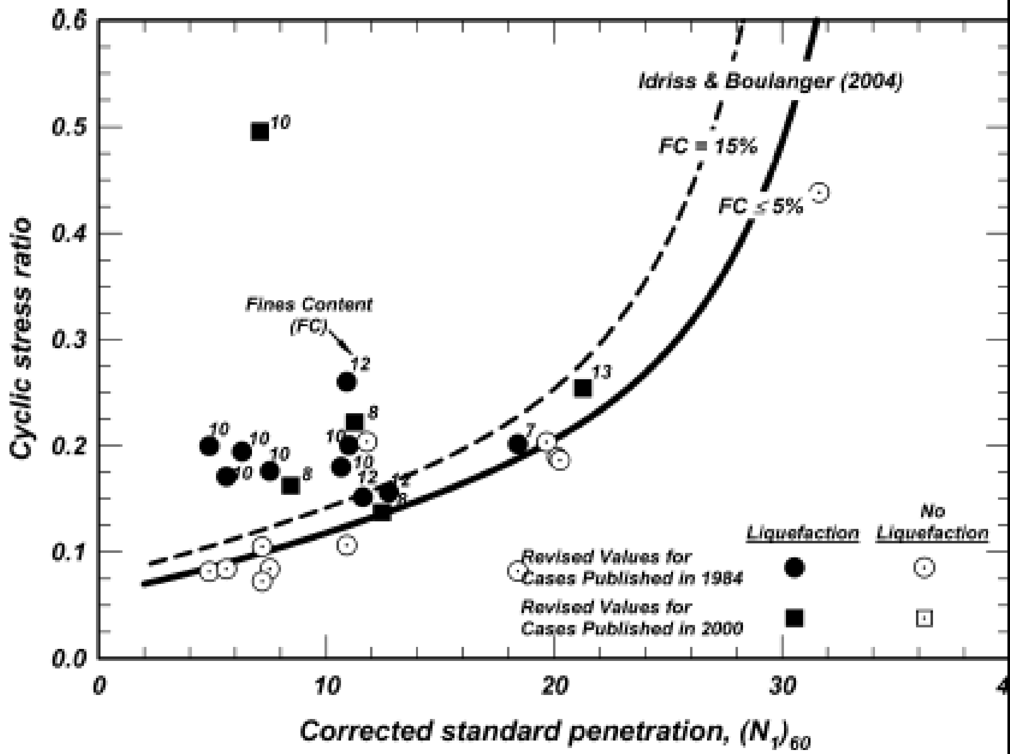
Estimation of liquefaction resistance of silty sands (CRR):

<u>Based on in-situ Test:</u>

- Using Standard Penetration Test

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

$$\Delta(N_1)_{60} = e^{\left(1.63+\frac{9.7}{FC+0.01}-\left(\frac{15.7}{FC+0.01}\right)^2\right)}$$



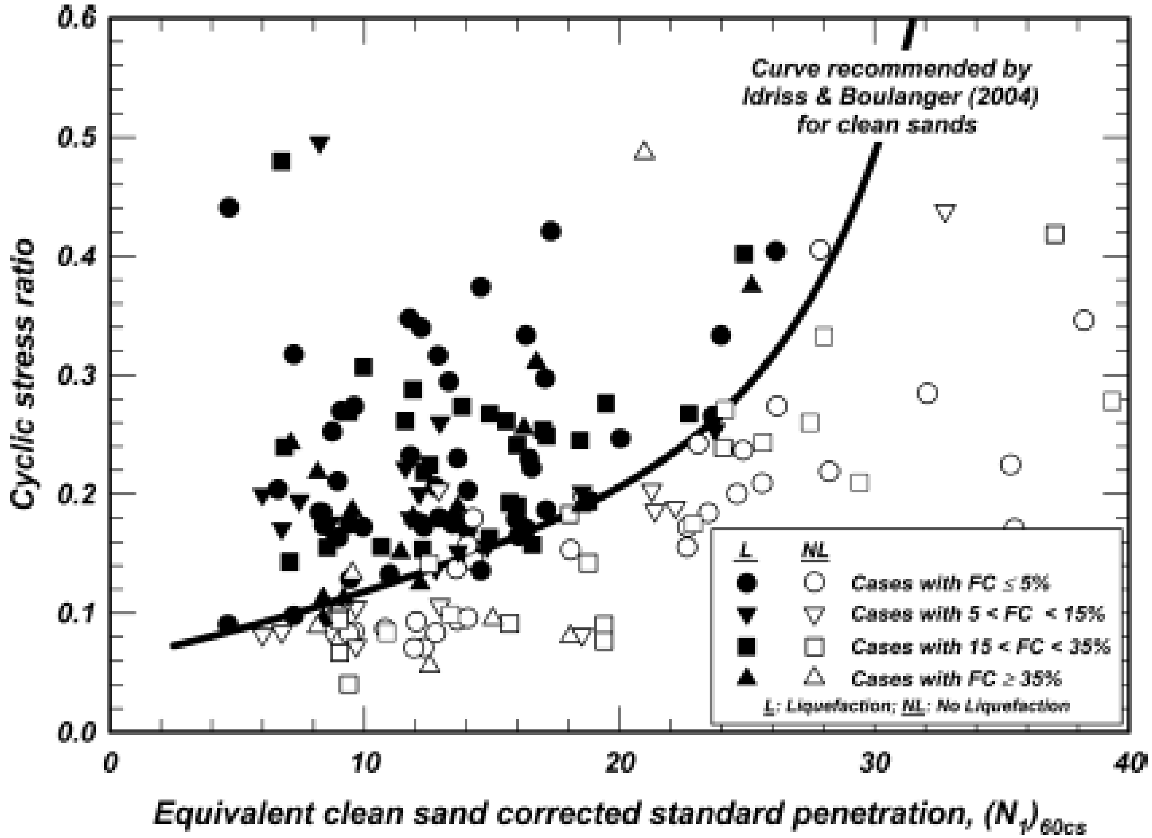
Boulanger and Idriss 2008

1. Cyclic Stress Approach

Estimation of liquefaction resistance of silty sands (CRR):

Based on in-situ Test:

- Using Standard Penetration Test



Boulanger and Idriss 2008

1. Cyclic Stress Approach

Estimation of liquefaction resistance of silty sands (CRR):

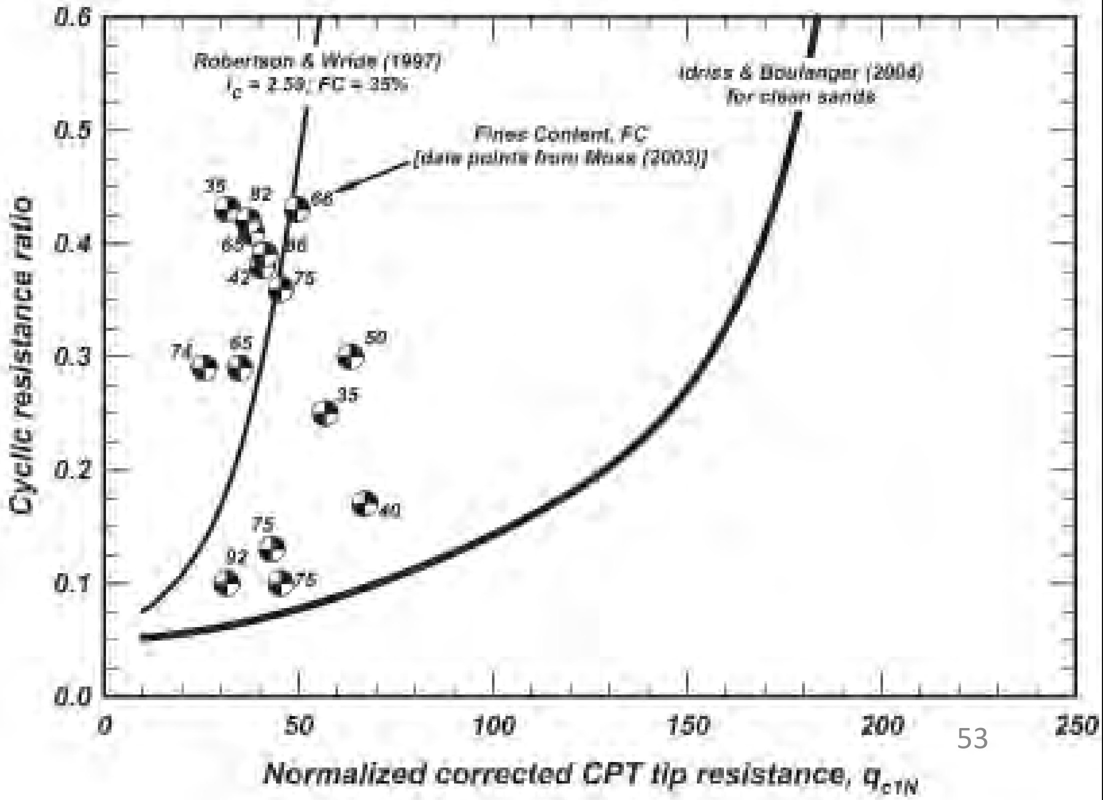
Based on in-situ Test:

- Using Cone Penetration Test

$$(q_{c1N})_{cs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(5.4 + \frac{q_{c1N}}{16}\right) \times e^{\left(1.63 + \frac{9.7}{FC+0.01} - \left(\frac{15.7}{FC+0.01}\right)^2\right)}$$

Boulanger and Idriss 2008



1. Cyclic Stress Approach

Estimation of liquefaction resistance of silty sands (CRR):

<u>Based on in-situ Test:</u>

- Using Cone Penetration Test

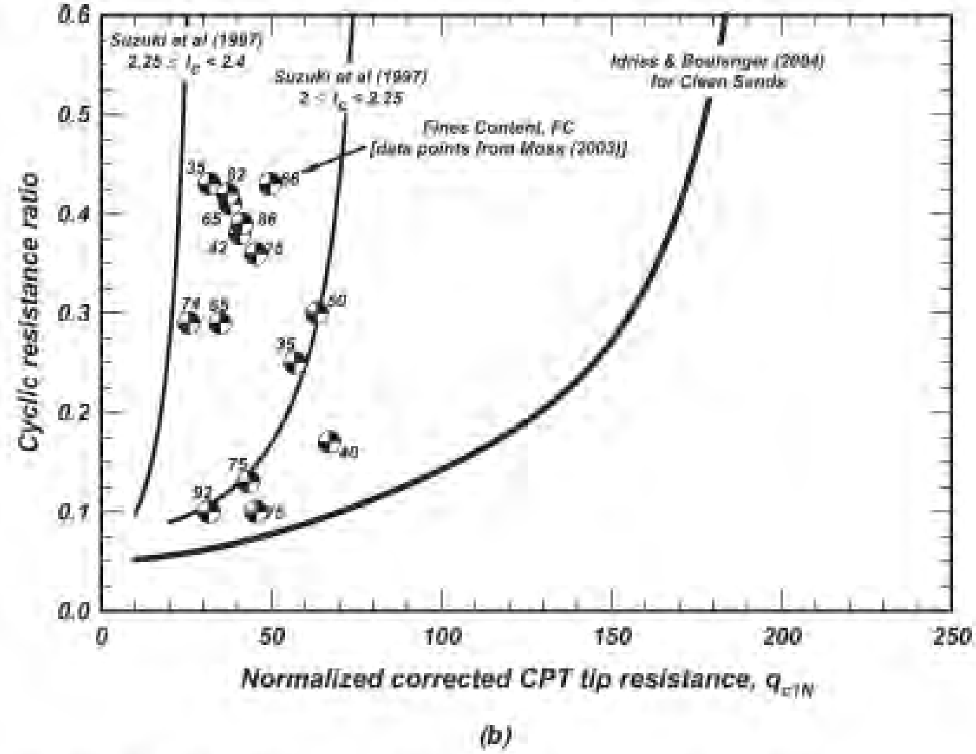


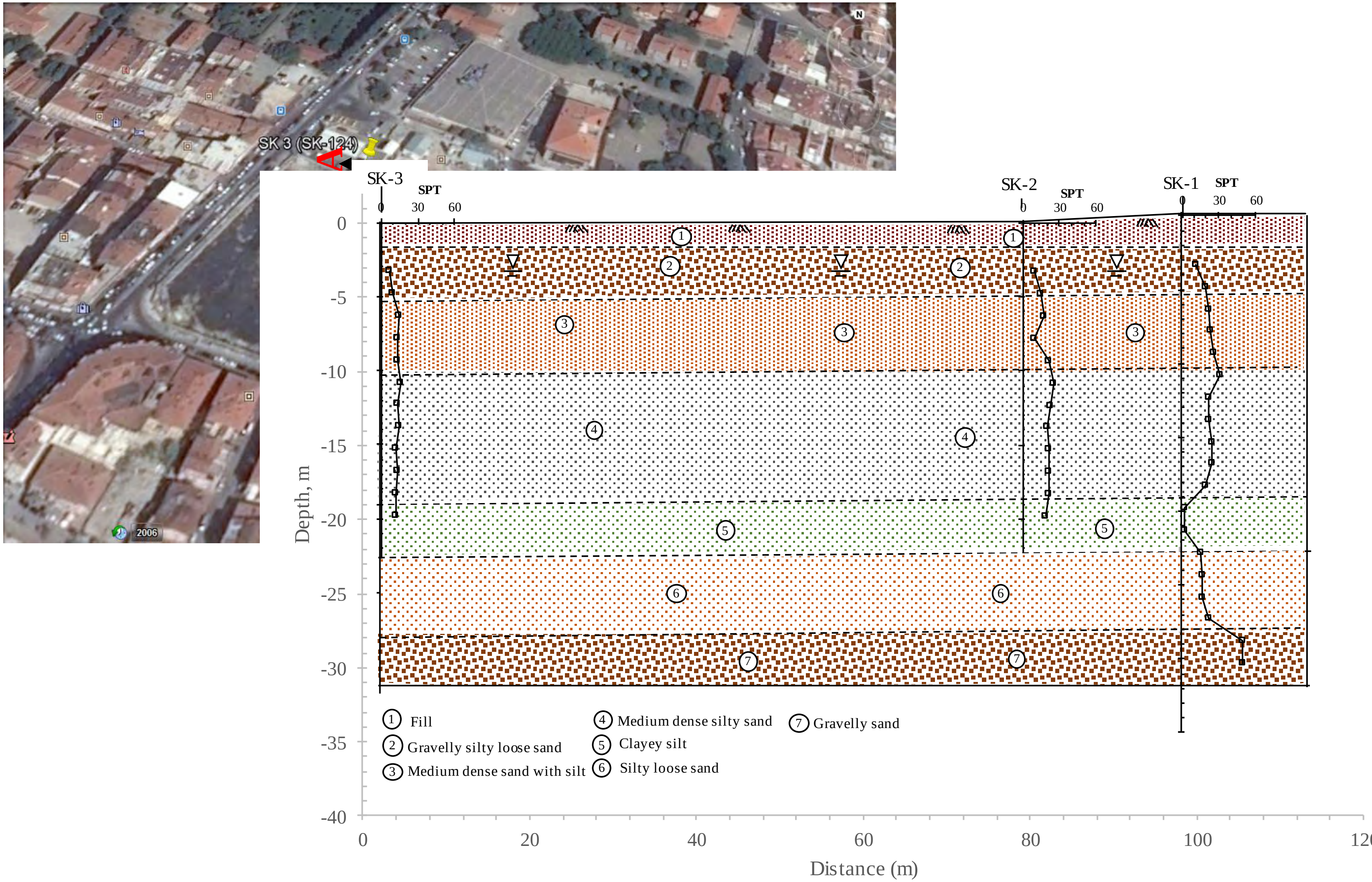
Figure 77. Comparison of field case histories for cohesionless soils with high fines content and the curves proposed by (a) Robertson and Wride (1997) for soils with $I_c = 2.59$ (apparent FC = 35%) and (b) Suzuki et al. (1997) for I_c values of 2.0–2.4.

Boulanger and Idriss 2008

3. Evaluation of Initiation of Liquefaction

1. Cyclic Stress Approach

Example 1.



1. Cyclic Stress Approach

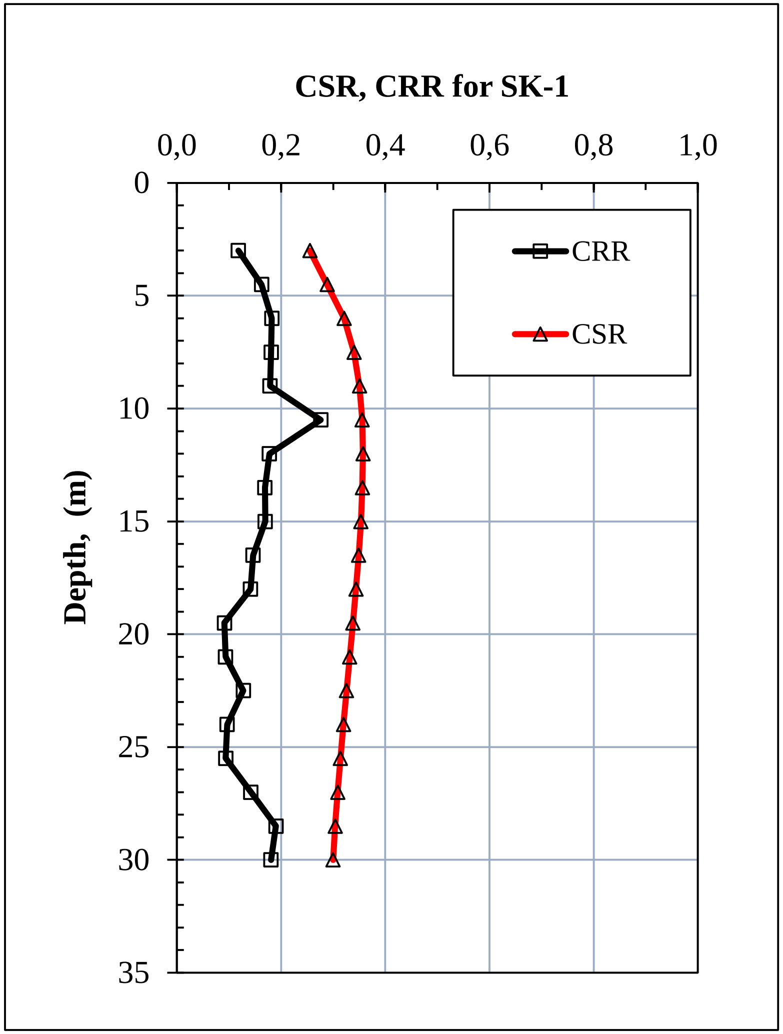
Example 1.

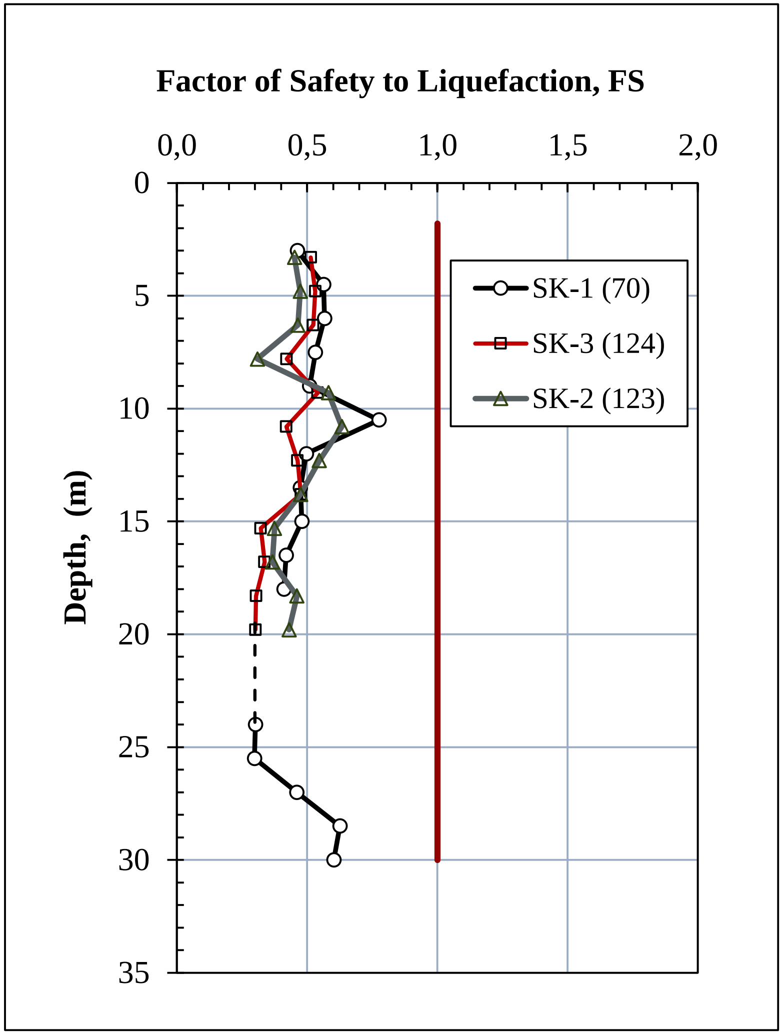
M_w=7.5 a_{max}=0.4 g, γ_d=16 kN/m^3, γ_{sat}=20 kN/m^3

Depth (m)	N_{60}	FC (%)	σ_v (kPa)	σ'_v (kPa)	r_d	C_N	$(N_1)_{60}$	CSR	CRR	FS
3.00	11	6	48.0	48.0	0.982	1.44	9	0.255	0.118	0.46
4.50	19	6	76.0	66.2	0.966	1.23	15	0.289	0.162	0.56
6.00	22	6	106.0	81.5	0.949	1.11	17	0.321	0.182	0.57
7.50	24	8	136.0	96.8	0.930	1.02	18	0.340	0.181	0.53
9.00	26	8	166.0	112.0	0.910	0.94	18	0.351	0.179	0.51
10.50	31	18	196.0	127.3	0.889	0.89	25	0.356	0.276	0.78
12.00	22	19	226.0	142.6	0.867	0.84	18	0.357	0.178	0.50
13.50	22	19	256.0	157.9	0.845	0.80	17	0.356	0.169	0.47
15.00	25	15	286.0	173.2	0.822	0.76	18	0.353	0.169	0.48
16.50	25	11	316.0	188.5	0.800	0.73	15	0.349	0.146	0.42
18.00	19	20	346.0	203.8	0.778	0.70	14	0.344	0.141	0.41
19.50	2	63	376.0	219.0	0.757	0.68	7	0.338	0.091	--
21.00	3	54	406.0	234.3	0.737	0.65	7	0.332	0.093	--
22.50	15	53	436.0	249.6	0.717	0.63	13	0.326	0.128	--
24.00	17	8	466.0	264.9	0.699	0.61	8	0.320	0.096	0.30
25.50	17	7	496.0	280.2	0.682	0.60	8	0.314	0.094	0.30
27.00	22	40	526.0	295.5	0.667	0.58	15	0.309	0.142	0.46

1. Cyclic Stress Approach

Example 1.





2. Probabilistic Approach

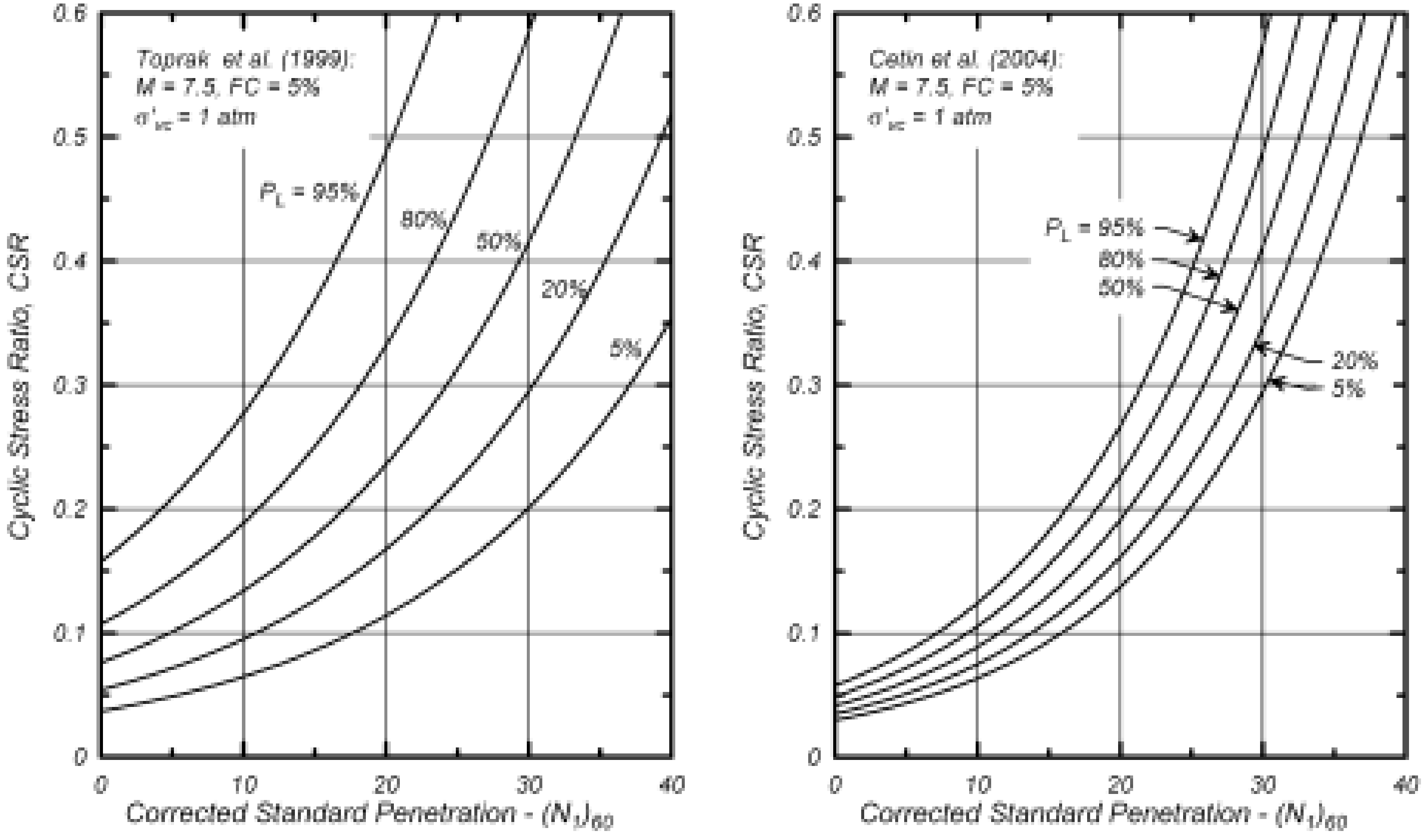
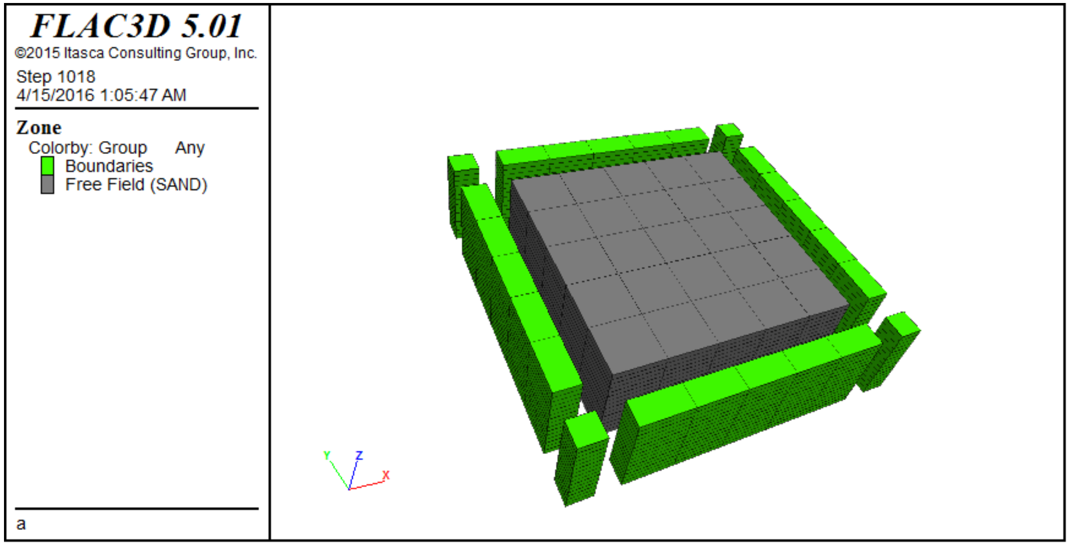


Figure 81. SPT-based probabilistic correlations for the CRR of clean sands for M = 7.5: (a) Toprak et al. (1999) and (b) Cetin et al. (2004, with permission from ASCE).

Boulanger and Idriss 2008

3. Numerical Modeling



Finn Model (uncoupled)

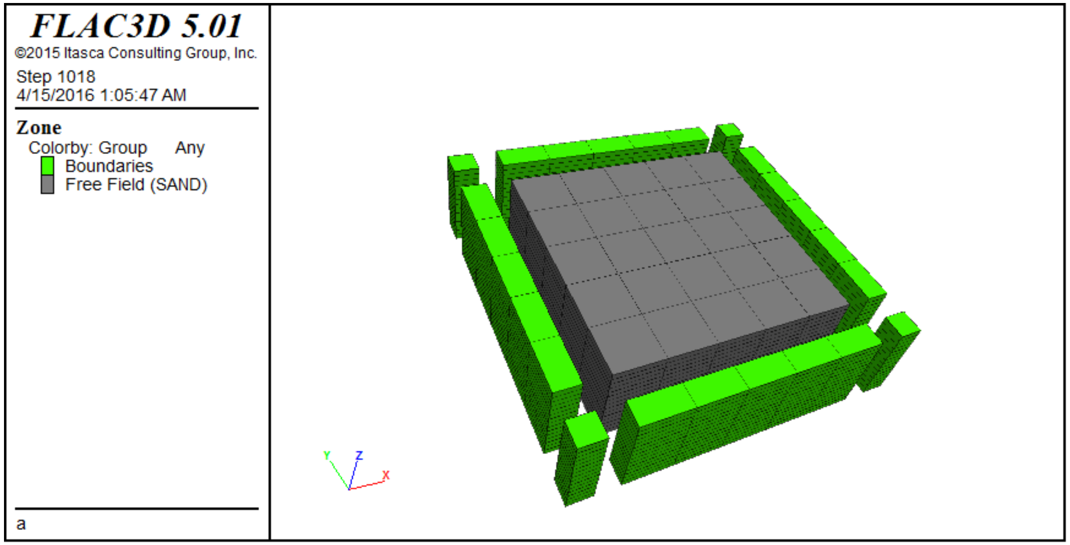
Δu in undrained condition
is related
to
Volumetric strain
developped in drained
condition

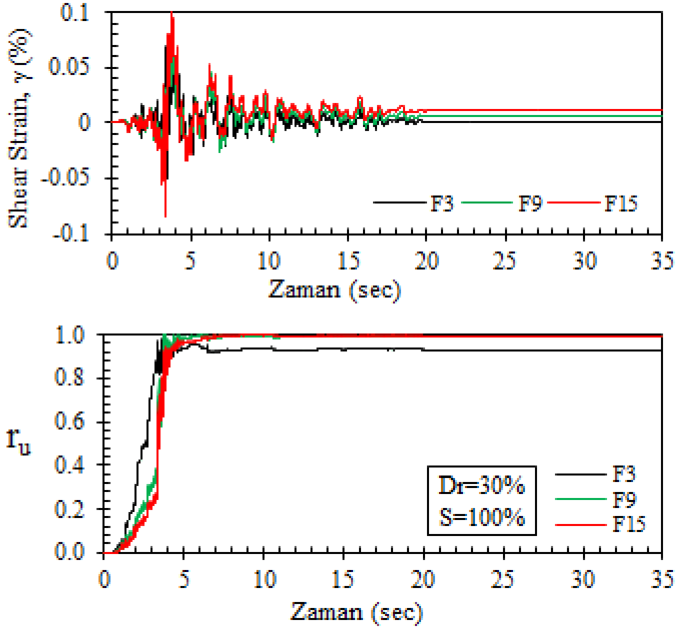
$$\Delta u = \frac{\Delta \varepsilon_{vd}}{\frac{1}{\overline{E}_r} + \frac{n_p}{K_w}}$$

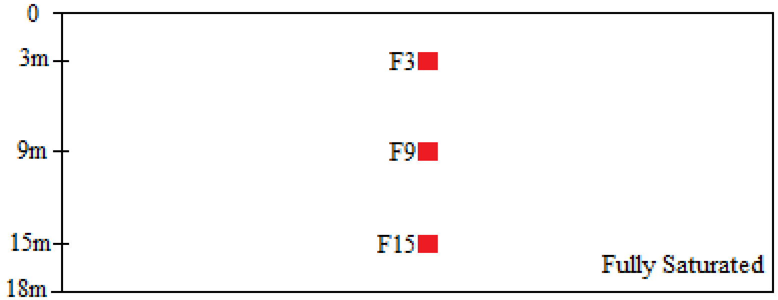
$$\Delta\varepsilon_{vd} = C_1(\gamma - C_2\varepsilon_{vd}) + \frac{C_3 + \varepsilon_{vd}^2}{\gamma + C_4\,\varepsilon_{vd}}$$

More coupled models are developed: UBCSand, PM4SAND, NUTASAND..etc.

3. Numerical Modeling







4. Consequences of Soil Liquefaction

Liquefaction Failures

Bearing capacity failure of a building after Izmit EQ, 1999

Failure of a bridge due to span loosing support in the Prince William Sound, Alaska Earthquake

Retaining wall damage and lateral spreading, Kobe 1995

Settlements and tilting of the buildings

4. Consequences of Soil Liquefaction

1. Sand Boils

Sand boils near Niigata (photo by K. Steinbrugge,courtesy of EERC, Univ. of California

Excess pore pressure dissipates by upward flow of pore water.
When the hydraulic gradient reaches to critical value vertical effective stress will reach to zero. In this case water velocity will be sufficient to carry sand particles to the surface.
It depends on:

- magnitude of excess pore pressure,
- thickness,
- density,
- permeability

1. Sand Boils

1. Sand Boils

2. Settlement

The post-liquefaction reconsolidation strains are computed by using relationships that are largely derived from laboratory studies but which have been found to provide reasonably good agreement with field observations (Lee and Albaisa 1974, Tokimatsu and Seed 1987, Ishihara 1996). They concluded it depends on:

- Relative Density or SPT N
- Maximum shear strain

1) Tokimatsu and Seed 1987)

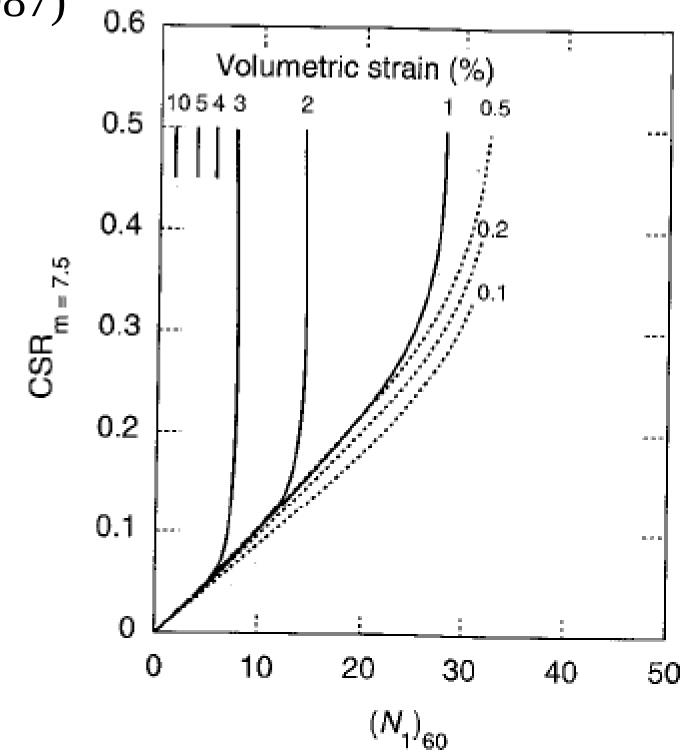


Figure 9.53 Chart for estimation of volumetric strain in saturated sands from cyclic stress ratio and standard penetration resistance. (After Tokimatsu and Seed, 1987. Evaluation of settlements in sand due to earthquake shaking, *Journal of Geotechnical Engineering*, Vol. 113, No. 8. Reprinted by permission of ASCE.)

$$CSR_{M=7.5}=\frac{CSR_{M=6.6}}{MSF}$$

2. Settlement

2) Ishihara and Yoshimine 1992 approach

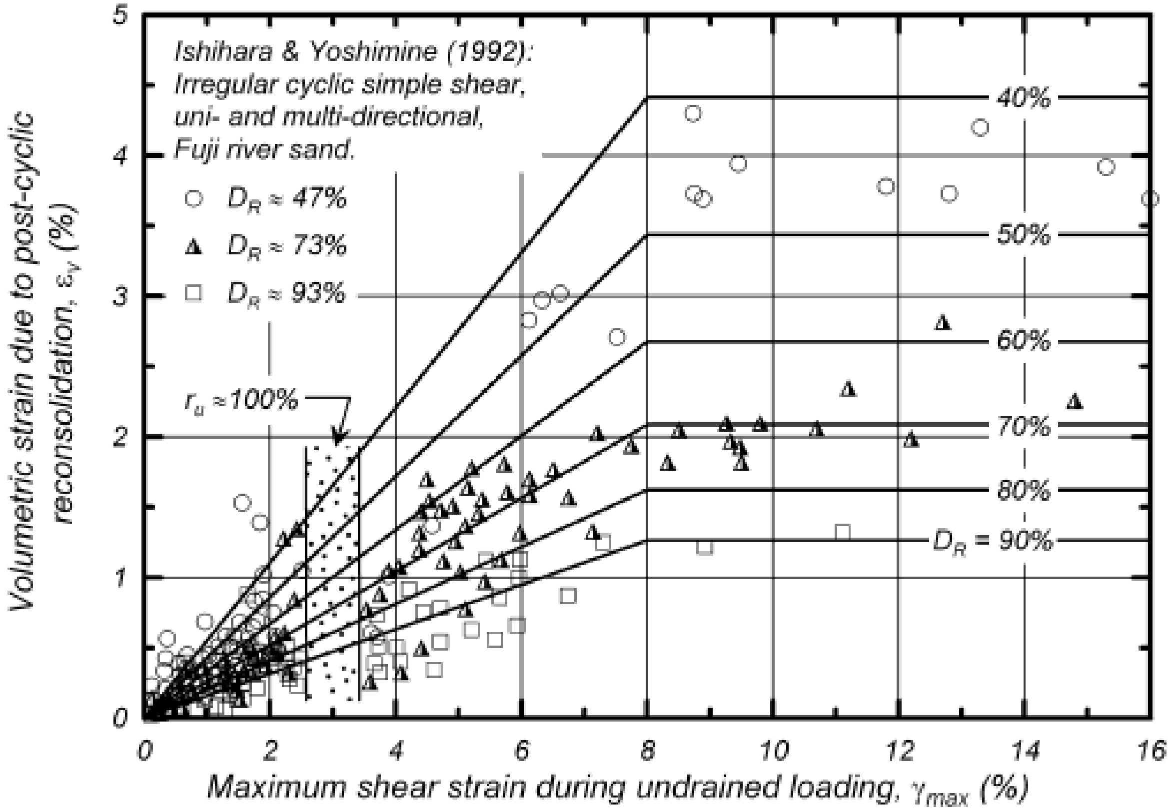


Figure 102. Relationship between post-liquefaction volumetric strain and the maximum shear strain induced during undrained cyclic loading of clean sand (after Ishihara and Yoshimine 1992).

2. Settlement

2) Ishihara and Yoshimine 1992 approach

$$\varepsilon_v = 1.5 \times e^{(-2.5 D_R) \times \min(0.08, \gamma_{\max})}, \text{ or}$$

$$\varepsilon_v = 1.5 \times e^{(-0.369\sqrt{(N_1)_{60cs}}) \times \min(0.08, \gamma_{\max})}, \text{ or}$$

$$\varepsilon_v = 1.5 \times e^{(2.551 - 1.147 (q_{c1Ncs})^{0.264}) \times \min(0.08, \gamma_{\max})}$$

$$q_{c1Ncs} \geq 21$$

2. Settlement

2) Ishihara and Yoshimine approach

The same relationship can be also represented by FS vs volumetric strain

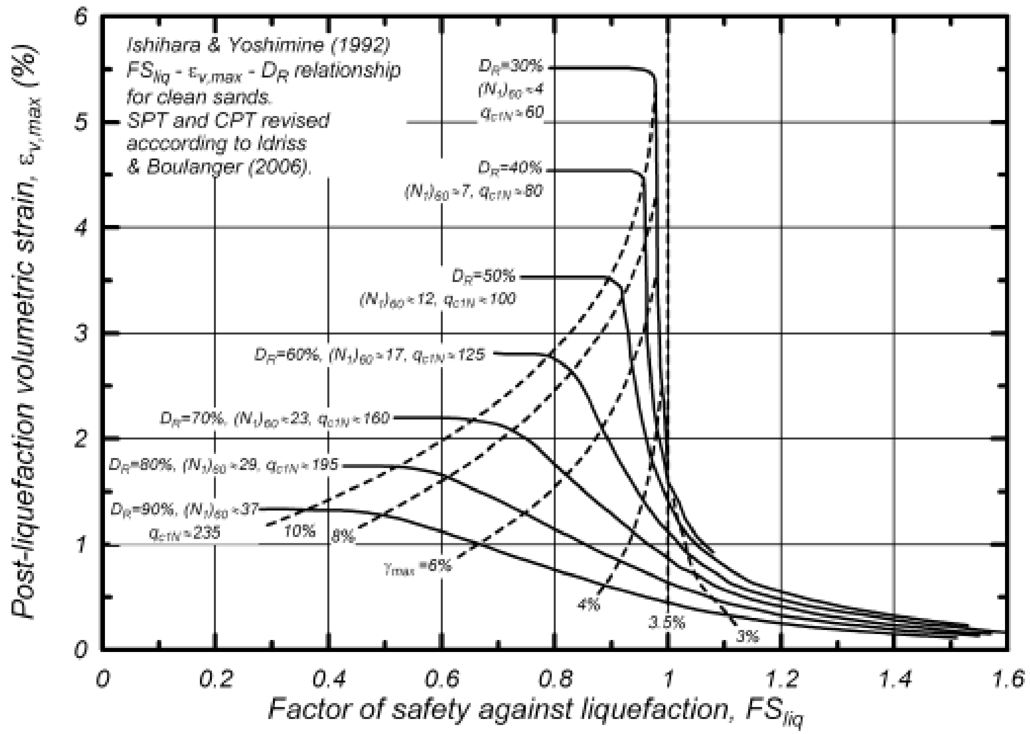


Figure 103. Post-liquefaction volumetric strains versus the factor of safety against triggering of liquefaction (r_u = 100%) for clean sands of different initial relative densities (after Ishihara and Yoshimine 1992).

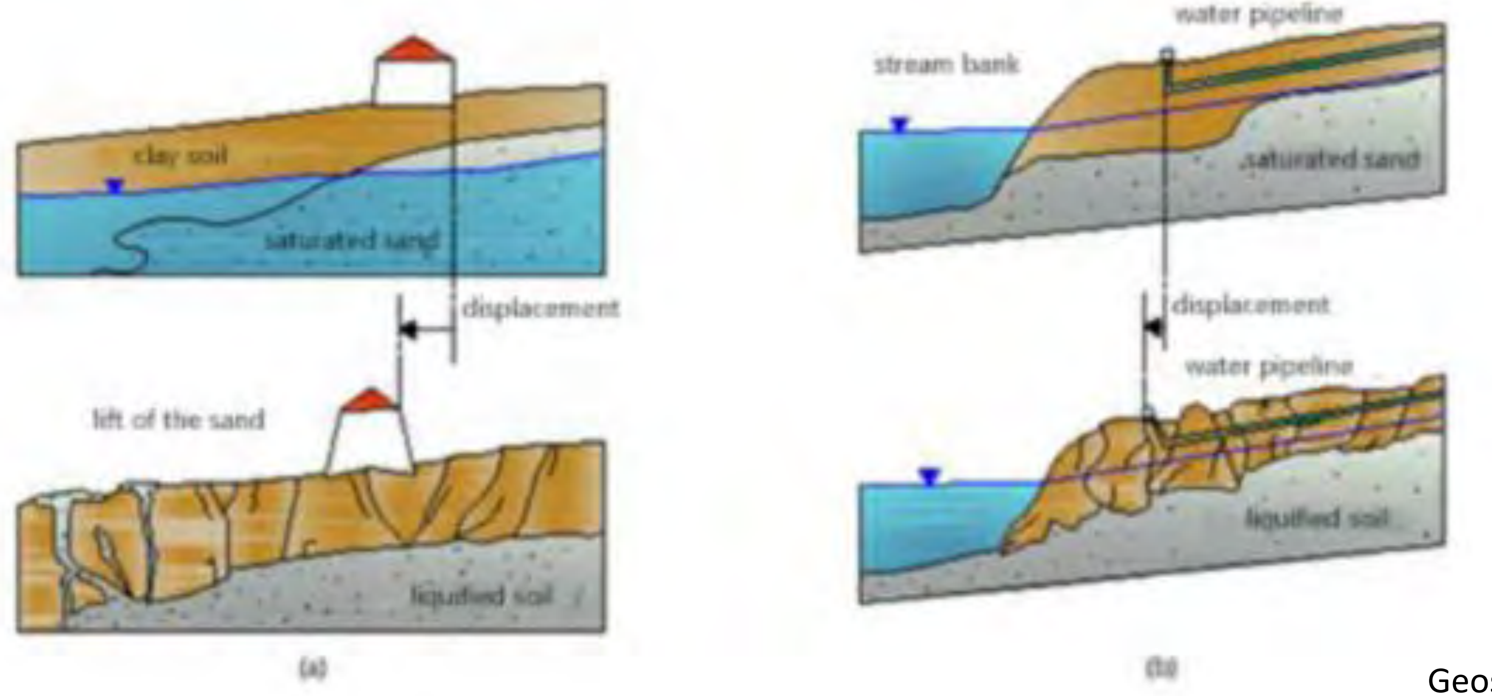
2. Settlement

Example 2.: Same site in Example 1.

Depth (m)	N_{60}	FC (%)	$(N_1)_{60}$	CSR	CRR	FS	Tokimatsu and Seed ε_v	Tokimatsu and Seed ΔH, m	Ishihara and Yoshimine ε_v	Ishihara and Yoshimine ΔH, m
3.00	11	6	9	0.255	0.118	0.46	0.0272	0.041	0.0414	0.062
4.50	19	6	15	0.289	0.162	0.56	0.0186	0.028	0.0314	0.047
6.00	22	6	17	0.321	0.182	0.57	0.0168	0.025	0.0279	0.042
7.50	24	8	18	0.340	0.181	0.53	0.0161	0.024	0.0264	0.040
9.00	26	8	18	0.351	0.179	0.51	0.016	0.024	0.0264	0.040
10.50	31	18	25	0.356	0.276	0.78	0.0122	0.018	0.015	0.023
12.00	22	19	18	0.357	0.178	0.50	0.0162	0.024	0.0264	0.040
13.50	22	19	17	0.356	0.169	0.47	0.0161	0.024	0.0279	0.042
15.00	25	15	18	0.353	0.169	0.48	0.0162	0.024	0.0264	0.040
16.50	25	11	15	0.349	0.146	0.42	0.01875	0.028	0.0314	0.047
18.00	19	20	14	0.344	0.141	0.41	0.0185	0.028	0.0328	0.049
19.50	2	63	7	0.338		--				
21.00	3	54	7	0.332		--				
22.50	15	53	13	0.326		--				
24.00	17	8	8	0.320	0.096	0.30	0.0291	0.044	0.0428	0.064
25.50	17	7	8	0.314	0.094	0.30	0.0291	0.044	0.0428	0.064
27.00	22	40	15	0.309	0.142	0.46	0.0188	0.028	0.0314	0.047

Total Settlement, $\Sigma\Delta h$: 0.4 m 0.65 m

3. Lateral Spreading



Geostru 2015

- The top layer will fracture into blocks and the material that constitutes the lower layer (liquefied) goes to fill the fractures.
- The fractured soil moves laterally toward the free surface with even metric displacements.
- Only empirical estimations

3. Lateral Spreading Sapanca Hotel, 16 August 1999

George Gazetas

3. Lateral Spreading Sapanca Hotel, On August 18, 1999

George Gazetas

3. Lateral Spreading
Pile Failure

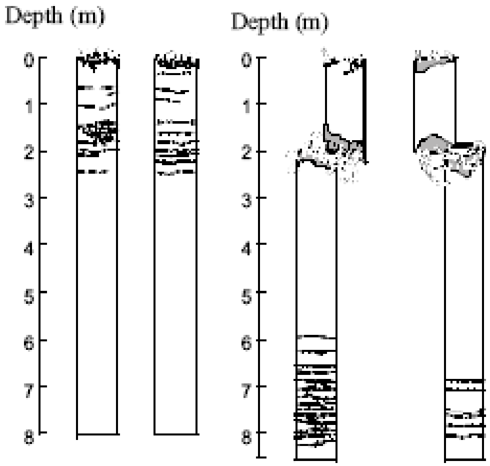


Fig. 24.62 Extent of damage in piles due to lateral displacement of liquefied subsoil (sketch by Prof. Nozomu Yoshida)

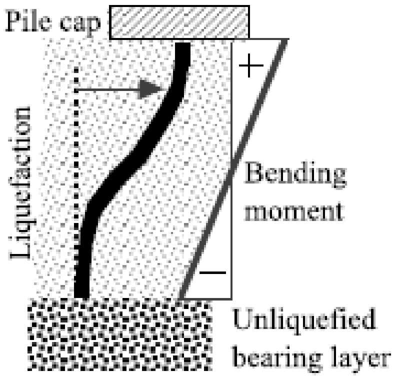


Fig. 24.63 Inferred mode of deformation along damaged pile

Fig. 24.64 Fallen Showa Bridge crossing Shinano River (Department of Civil Engineering, University of Tokyo)

Towhata 2008

3. Lateral Spreading
Pile Failure

https://civil-engg-world.blogspot.com.tr/2015/05/Geologic-Geotechnical-Investigation-Seismic-Design-Foundation.html

4. Bearing Capacity Failure

Adapazarı, Turkey

Bearing capacity of the foundation drops due to the reduction in effective stresses consequently in the shear modulus of the foundation soil

5. Ground Response

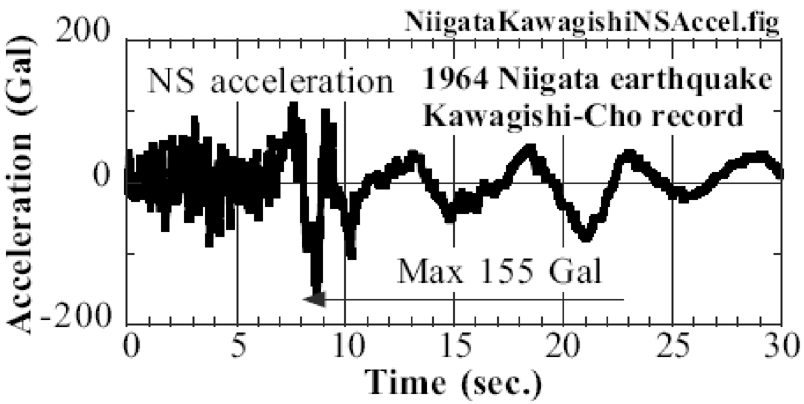


Fig. 17.45 Niigata Kawagishi-cho NS motion

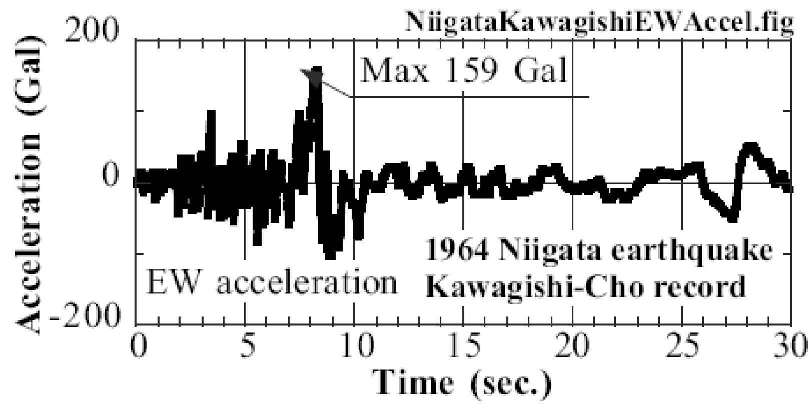


Fig. 17.46 Niigata Kawagishi-cho EW motion

Towhata 2008

The maximum response is at long period since the soil shear modulus becomes lower as the excess pore pressures increases.

5. Ground Response

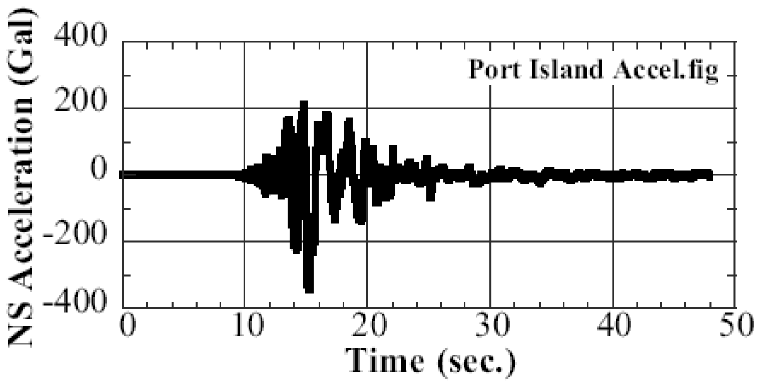


Fig. 17.47 Acceleration time history in Kobe Port Island (Kobe City Development Bureau)

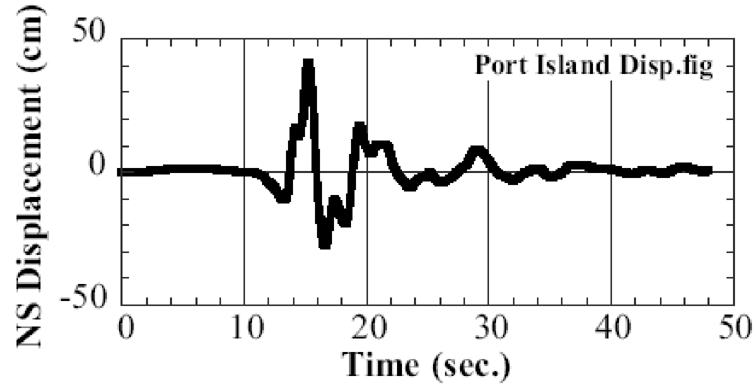


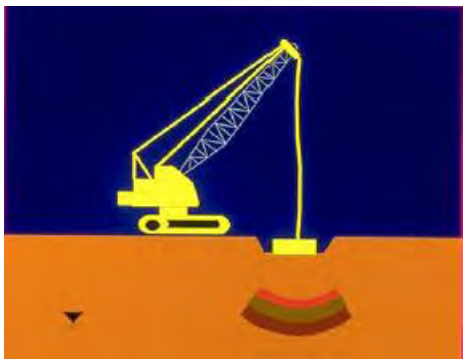
Fig. 17.48 Displacement time history in Kobe Port Island

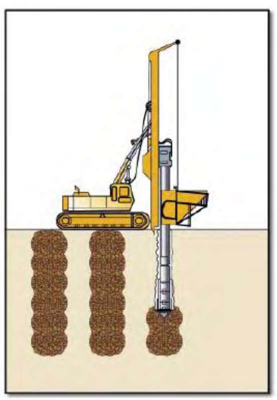
Towhata 2008

- reduces the acceleration at the surface,
- increases the surface displacement amplitude.
- Thus, displacement seems to be a more appropriate index of subsurface liquefaction.

❖ Performance-based analysis/design is needed

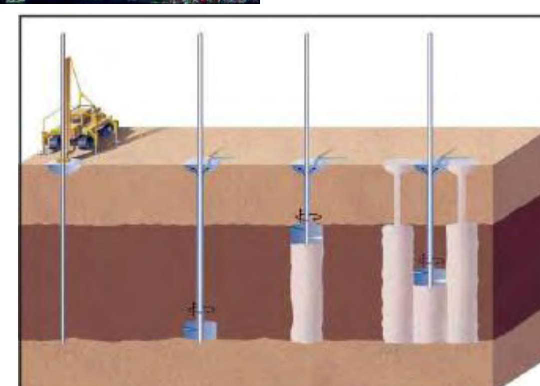
Liquefaction Mitigation Techniques





In-situ soil densification

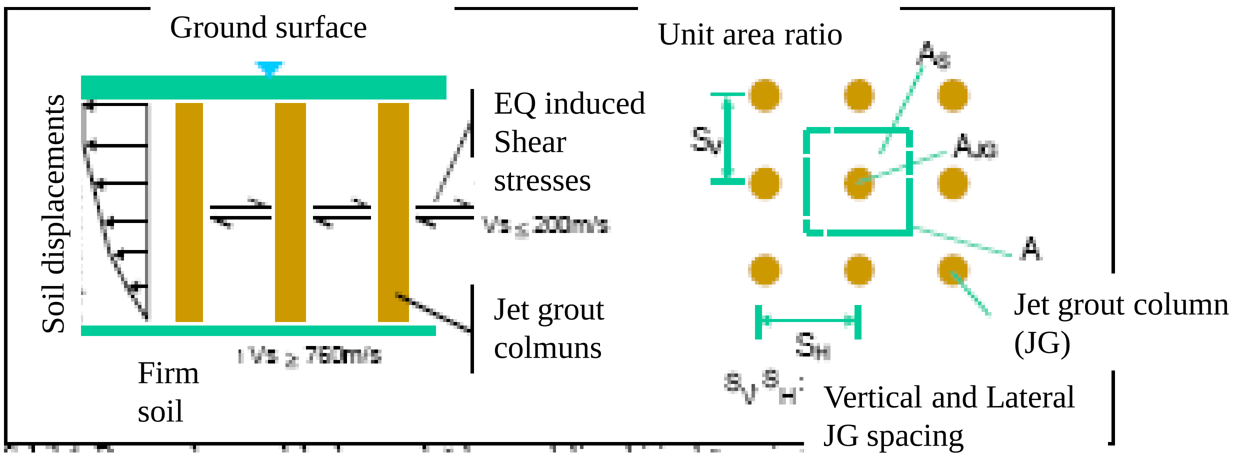
Stone Columns or Wick Drains



Chemical Injection and Grouting

Liquefaction Mitigation Techniques

Jet grout



$$a_r = \frac{A_{JG}}{A}$$

$$a_S = \frac{A_S}{A} = 1 - a_r$$

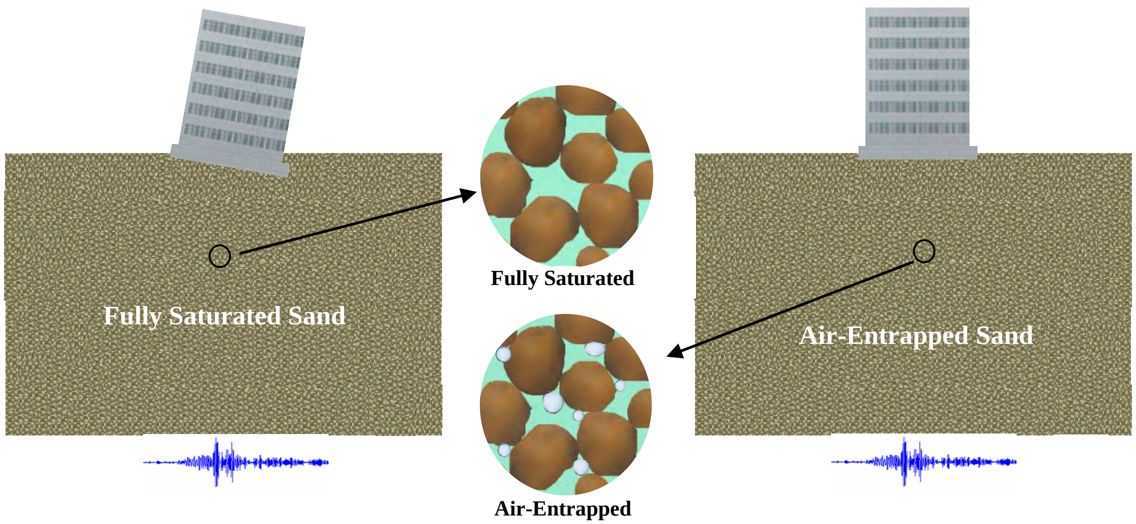
Özsoy and Durgunoğlu 2003

$$\tau = \tau_S a_S + \tau_{JG} a_r$$

$$\tau = \tau_S (1 - a_r) + \tau_{JG} a_r$$

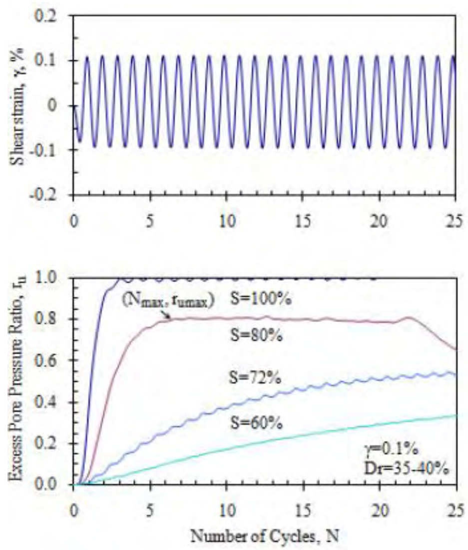
Induced Partial Saturation

- Reduction of degree of saturation by the injection of sodium perborate which creates partially saturated sands in liquefiable areas, even under the existing buildings.



Building Response in Fully Saturated Sand

Building Response in Air-Entrapped Sand



Eseller-Bayat 2013

$$C_{aw} = \frac{1}{K_{aw}}$$

$$\Delta u = \frac{\Delta \varepsilon_{vd}}{\frac{1}{\overline{E}_r} + n_p C_{aw}}$$

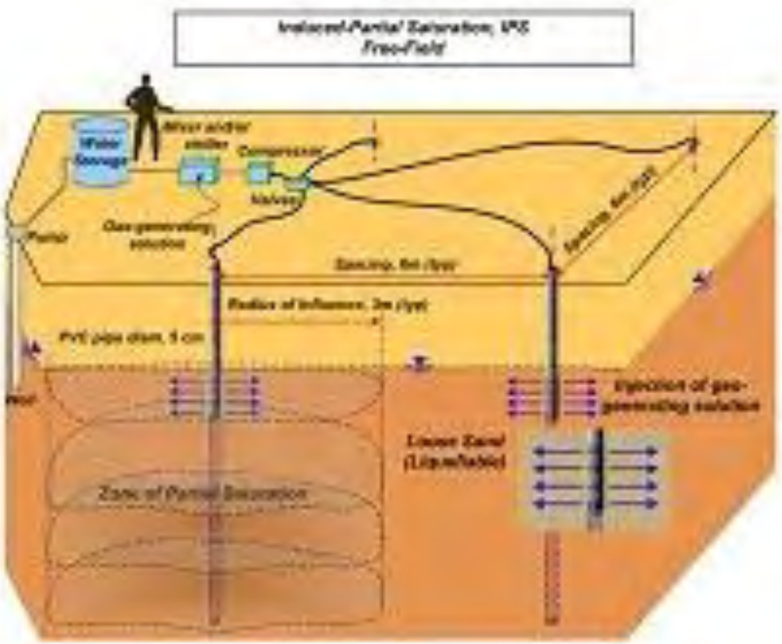
$$\Rightarrow$$

$$\Delta u = \frac{\Delta \varepsilon_{vd}}{\frac{1}{\overline{E}_r} + n_p \left[S C_w + \frac{(1-S)}{u_a} \right]}$$

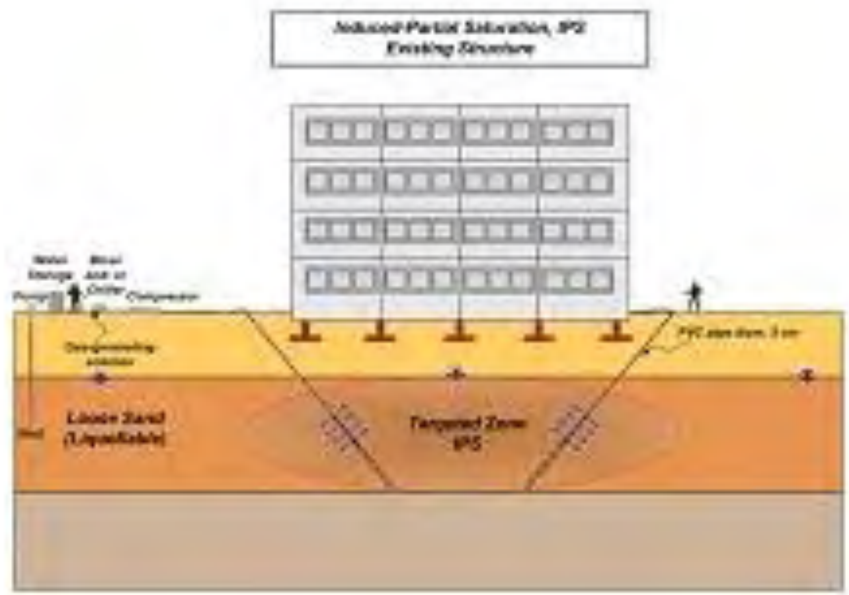
Induced Partial Saturation

➢ Reduction of degree of saturation by the injection of sodium perborate which creates partially saturated sands in liquefiable areas, even under the existing buildings.

Schematic of the IPS application in free field



Schematic of the IPS application under an existing structure



M. K. Yegian

Homework Assignment

Choose one of the following 2 questions for the HW:

1) For a site in Turkey, the following standard penetration test (SPT) data is given. Ground water table (GWT) is at 1 m below the ground surface. The characteristic earthquake for the site has 7.5M and 0.4g peak acceleration. Estimate the total settlement of the ground due to liquefaction, if there is any. Assume fine content (FC) <5%. Unit weight of the soil: γ_{dry}=18 kN/m^3 γ_{sat}=20 kN/m^3.

Depth, m	$(N_1)_{60}$
1.80	15.8
3.30	14.8
6.30	41.1
7.80	41.9
10.80	11.7
13.80	13.1
16.80	11.1

2) Please answer the following questions for your home-country:
   i. What is the most common liquefaction assesment (evaluation) procedure or method used according to the codes or application in practice ?
   ii. What type of liquefaction-induced failures observed most?
   iii. What is the most common mitigation technique applied in practice.

Your HW should be submitted before Oct. 28, to the address below.
Email : homework@quake.enveng.titech.ac.jp