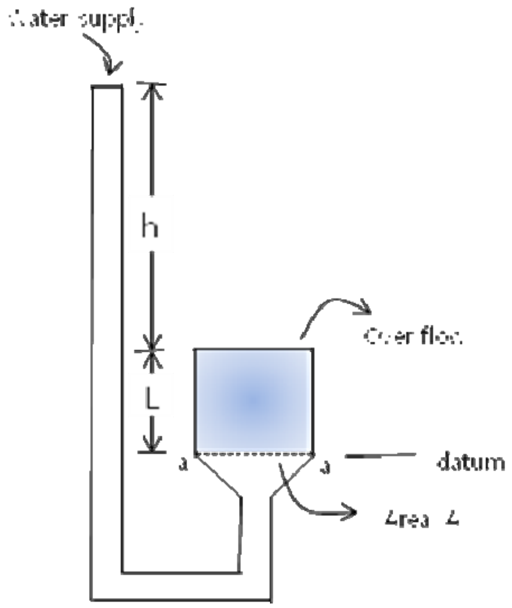
Liquefaction of Soils

- Liquefy : soils that has lost all shear strength behaves like a viscous fluid

  → quick condition (in static condition)



- the total upward water force across the surface $\overline{aa}'$ (seepage force)

  $= (h+L)\gamma_w A \quad \ldots①$

- the weight of the saturated soil mass above this surface

  $= \frac{G+e}{1+e}\gamma_w LA \quad \ldots②$

- quick condition occurs if ① ≥ ②, I.e. ,

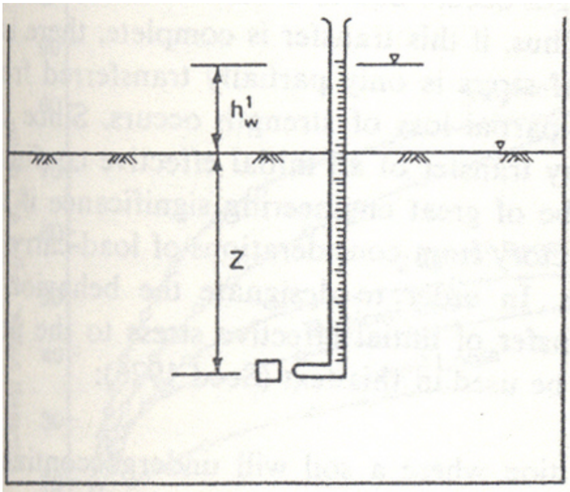
$$(h+L)\gamma_w A \geq \frac{G+e}{1+e}\gamma_w LA$$

$$\rightarrow \frac{h}{L} \geq \frac{G-1}{1+e} \quad \text{(normally } \approx 1\text{)}$$

↑

Hydraulic gradient

- Theoretical Background(for the liquefaction)



- the shear strength of sand,

$$\tau = (\sigma - u)\tan\bar{\phi} \quad \ldots①$$

σ : total normal stress

u : pore water pressure

$\bar{\phi}$: angle of internal friction in terms of effective stresses

- at depth z,

$$\sigma = \gamma_{sat} \cdot z \ , \ \ u = \gamma_w \cdot z$$

- thus,

$$\tau_f = (\gamma_{sat} - \gamma_w) z \cdot \tan\bar{\phi}$$

- if there is an increase in pore water pressure of $\Delta u = \gamma_w \cdot h_w{}'$ due to the earthquake loading

$$\tau_f = (\gamma_b \cdot z - \Delta u) \cdot \tan\bar{\phi}$$

$$= (\gamma_b \cdot z - \gamma_w h_w{}') \cdot \tan\bar{\phi}$$

- For a complete loss of strength

$$\gamma_b z = \gamma_w \cdot h_w{}' \ \rightarrow \ \frac{h_w{}'}{z} = \frac{\gamma_b}{\gamma_w} = \frac{G-1}{1+e} = i_{cr}$$

i_{cr} : critical hydraulic gradient

Terminologies connected to the Liquefaction.

Initial Liquefaction :

A condition where, during the course of cyclic stress applications, the residual pore-water pressure becomes equal to the applied confining pressure on completion of any full stress cycle.

(True) Liquefaction :

A condition where a soil will undergo continued deformation due to the build up and maintenance of high pore water pressures which reduce the effective confining pressure to a very low value.

Initial Liquefaction with Limited Strain Potential (or cyclic mobility of cyclic liquefaction) :

A condition where Subsequent stress application cause limited strains to develop

(In dense sand, due to dilation of soil → vol. increase → pore pressure drops → Soil stabilize)

● Criterion of Liquefaction

Casagrande(1936)

Proposed the critical void ratio (e_{cr}) as a possible criterion of Liquefaction

(critical void ratio=a void ratio at which there in no change in volume at failure)

However, ① $e_{cr} = f(\sigma_3)$, i.e., not a unique property (material constant)of the soil

② loading conditions(drained static) are entirely different from the field condition

Maslov(1957)

The concept of critical acceleration(a_{cr})

However, ① a_{cr} =f(D_R , Amplitude & frequency of oscillation σ_n),

i.e., not a unique property

Florin & Ivanov(1961)

Field blasting test

Their interpretation may need refinement

Remark : SPT value N may solve this riddle

(or cyclic triaxial test)

- Factors affecting liquefaction potential

1. Grain size distribution of sands

   - fine and uniform sands are more prone to liquefaction

   (∵ permeability, density)

2. Initial relative density

3. Vibration characteristics

   - shock loading or steady-state, frequency, pulse form, horizontal or vertical, duration…

4. Location of drainage & dimensions of deposit.

   - in large dimensions → undrained condition for a quick loading

   (gravel drains may be introduced, $k_d \geq 200 k_s$)

5. Magnitude & nature of superimposed loads

   - for a large initial effective stress to be transferred to the pore pressure → need a large intensity or number of stress cycles of vibration

6. Method of Soil Formation

   - characteristic soil structure

7. Period under sustained load →

   - the age of a sand deposit

8. Previous strain history

9. Trapped air

   - helps reduce the possibility

- Typical Results

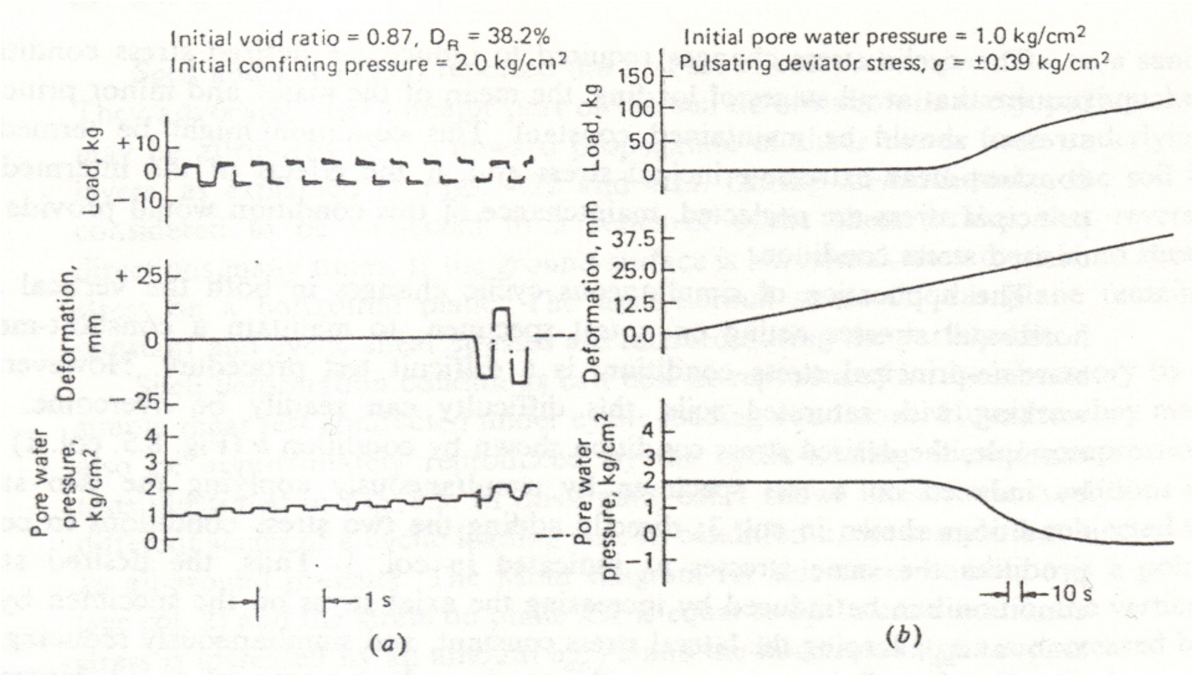


Figure 8.6 (a) Cyclic triaxial test on loose sand (b) Static test after liquefaction.(After Seed and Lee, 1966)

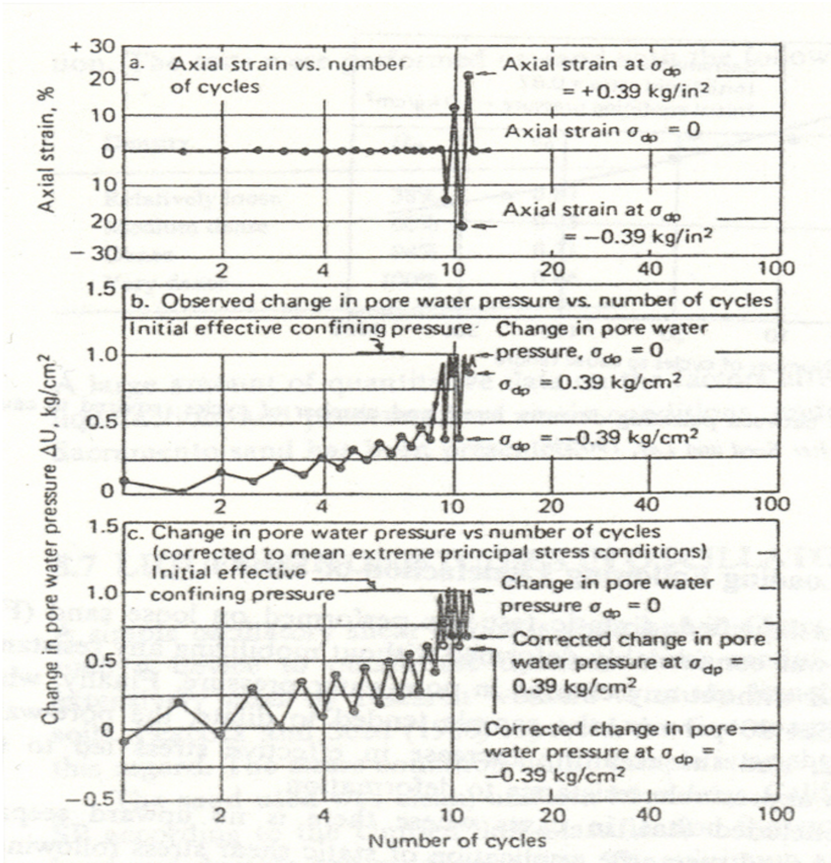


Figure 8.7 Typical pulsating load test on loose sand ; e=0.87, D_R=38%,σ_3= 1kg/cm², σ_{dp}=0.391kg/cm². (After Seed and Lee, 1966)

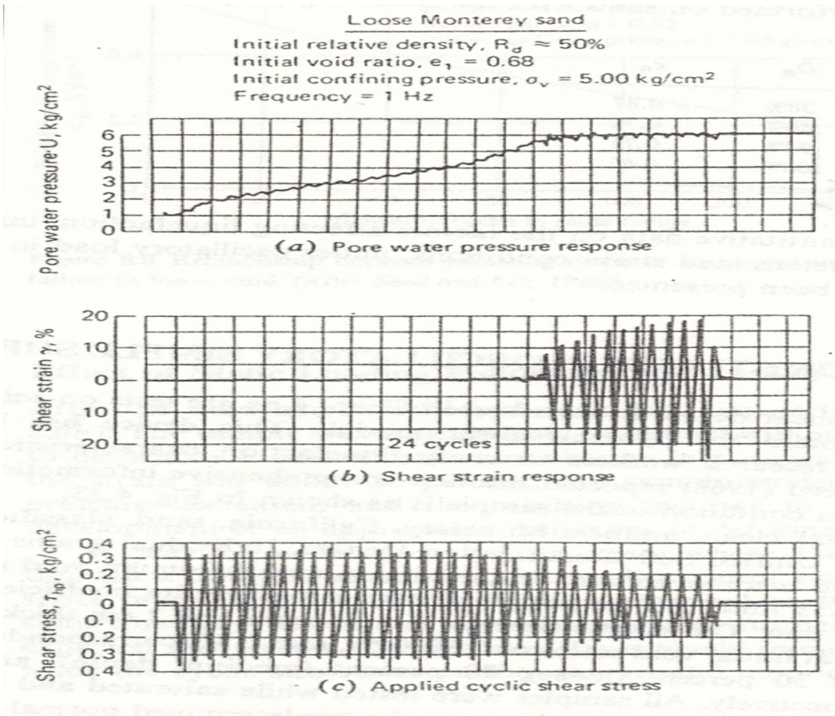


Figure 8.9 Record of a typical pulsating load test on loose sand in simple shear conditions. (After Peacock and Seed, 1968)

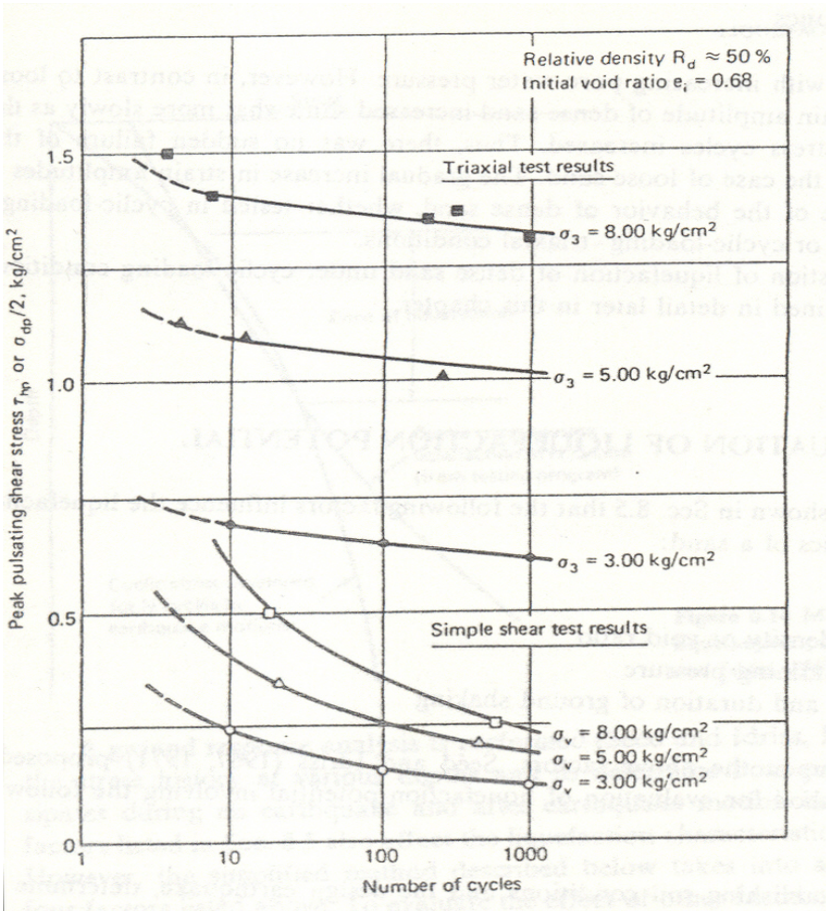


Figure 8.12 Cyclic stresses required to cause initial liquefaction of Monterey sand at three confining pressure in triaxial and simple shear tests.(After Reacock and Seed, 1968)

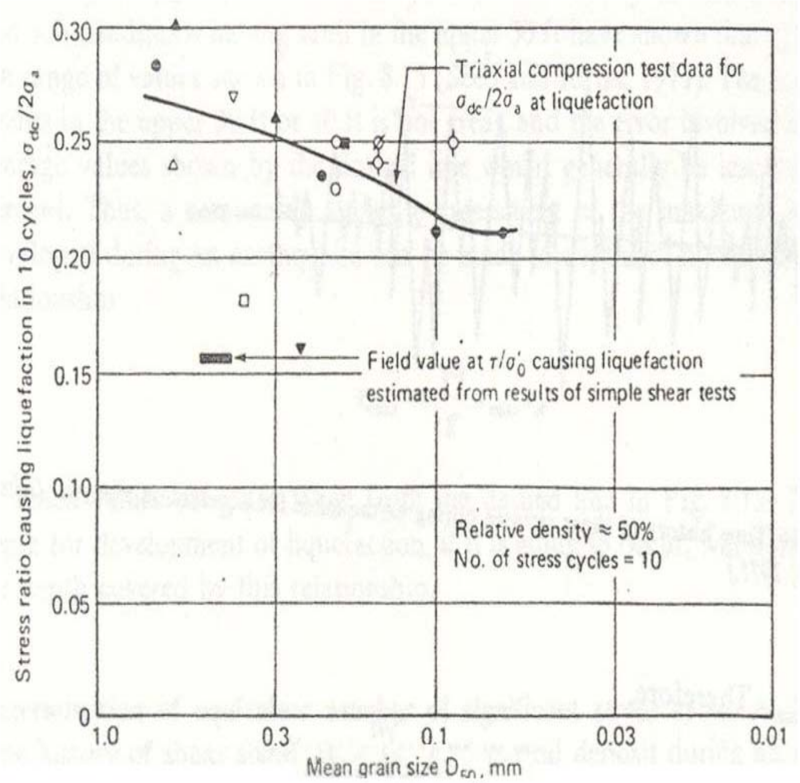


Figure 8.17 Stress condition causing liquefaction of sands in 10 cycles.(After Seed and Idriss, 1971)

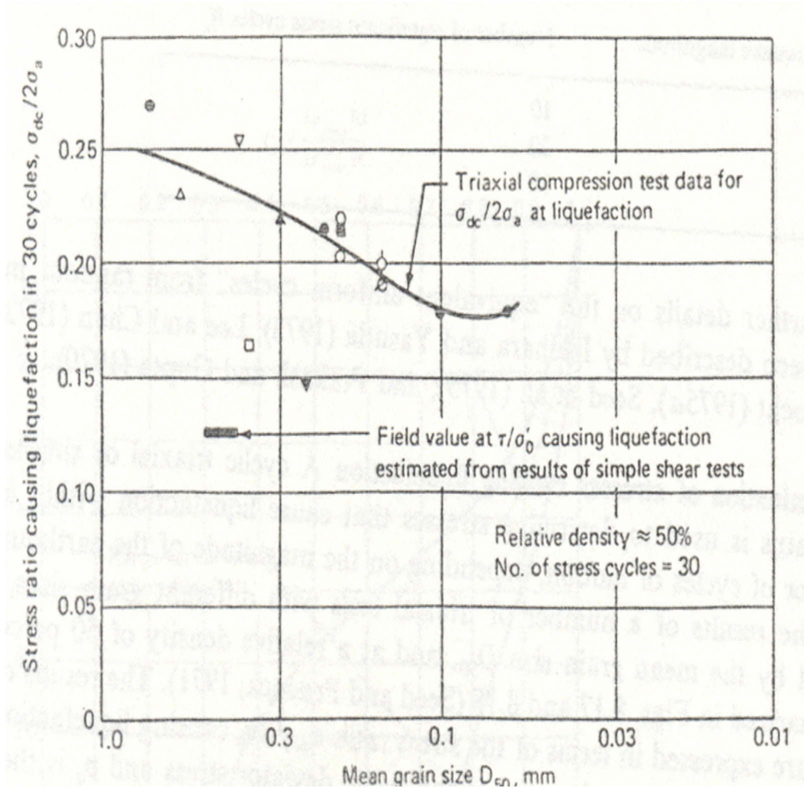


Figure 8.18 Stress condition causing liquefaction of sands in 30 cycles.(After Seed and Idriss, 1971)

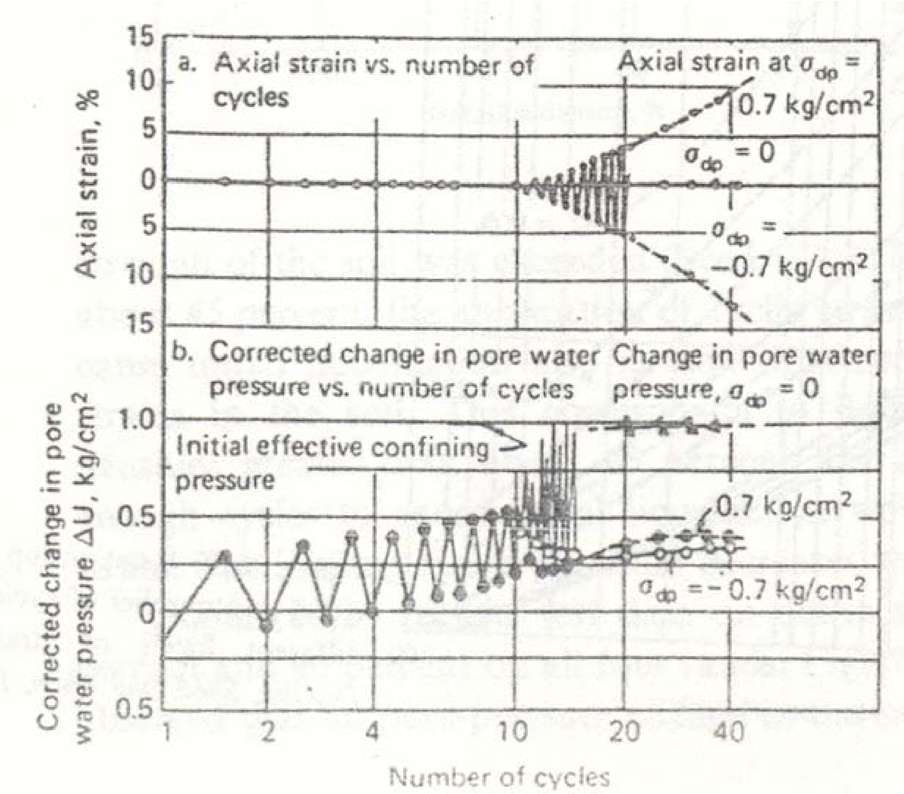


Figure 8.35 Typical pulsating load test on a dense sand.(After Seed and Lee, 1966)

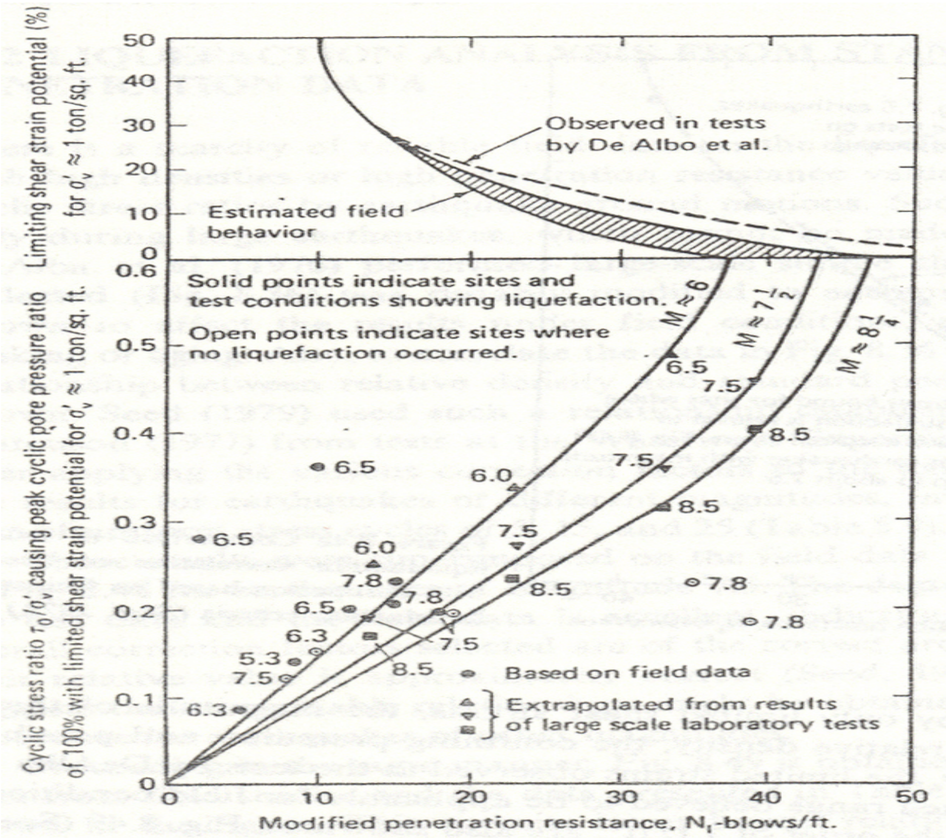
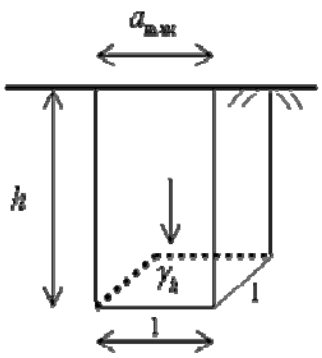


Figure 8.49 Correlation between field liquefaction behavior of sands for level ground conditions and penetration resistance supplemented by data from large scale tests. (After Seed, 1979)

Evaluation of Liquefaction Potential

(simplified procedure, Seed & Idriss, 1971)



1. Computation of maximum shear stresses in the deposit

Assuming rigid soil element,

$$(\tau_{max})_r = \frac{\gamma h}{g} a_{max}$$

a_{max} : maximum ground surface acceleration

Deformable body,

$$(\tau_{max})_d = \gamma_d (\tau_{max})_r$$

γ_d : a stress reduction coefficient

$$\rightarrow \tau_{max} = \frac{\gamma h}{g} a_{max} \cdot \gamma_d$$

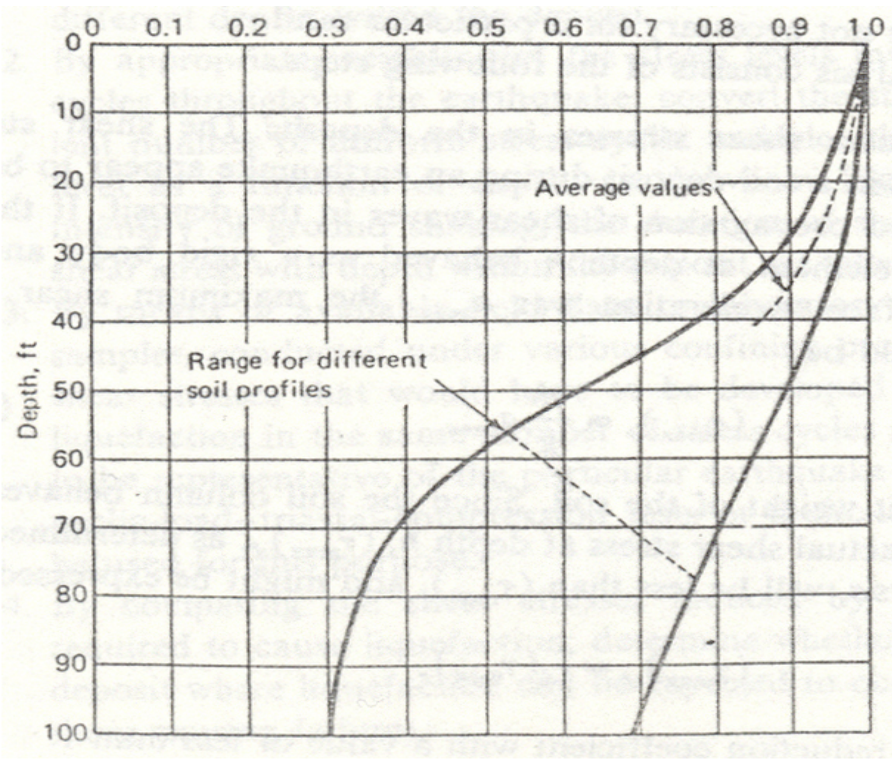


Figure 8.15 Range of values of γ_d for different soil profiles in liquefaction analysis.

(After Seed and Idriss, 1971)

2. Determination of equivalent number of significant stress cycles, N_c

- $\tau_{av} \fallingdotseq 0.65\tau_{\max}$

$$\fallingdotseq 0.65 \times \frac{\gamma h}{g} a_{\max} \times \gamma_d \qquad \ldots ①$$

- N_c depends on the duration of shaking i.e. the magnitude of the earthquake

magnitude	N_c
7	10
7.5	20
8	30

3. Determination of stress causing liquefaction

- Determine $\frac{\sigma_{dc}}{2\sigma_a}$ from Fig, 8.17 or 8.18 for appropriate D_{50} & N_c (D_{50} =mean grain size)
- Obtain the field value (from simple shear test)

$$(\frac{\tau}{\bar{\sigma}_0})_l = (\frac{\sigma_{dc}}{2\sigma_a})_l c_r \,, \quad c_r \text{ = a correction factor}$$

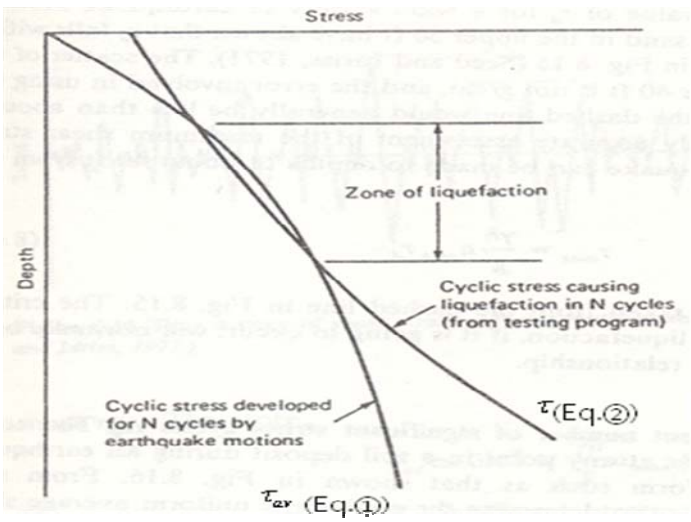
D_r	C_r
0-50	0.57
60	0.60
80	0.68

- Considering the relative density

$$(\frac{\tau}{\bar{\sigma}_0})_{lD_r} = (\frac{\sigma_{dc}}{2\sigma_a})_{l_{50}} c_r \cdot \frac{D_r}{50} \quad \text{(good up to } D_r = 80\%\text{)} \qquad \ldots ②$$

$\bar{\sigma}_0$:initial effective overburden pressure

4. Evaluation of liquefaction potential compare τ_{av} from Eq. ① with τ from Eq. ②



● Liquefaction analysis from SPT data.

1. Compare the correction average penetration resistance, N_1 (under an effective overburden pressure of 1 t/ft^2)

   $N_1 = C_N N$

   $C_N = 0.77 \log \dfrac{20}{\bar{P}}$, $\bar{P}$: effective overburden pressure in t/ft^2

   N : SPT value

2. Determine τ_{av} from Eq. ①

3. Determine $\dfrac{\tau_0}{\bar{\sigma}_v}$ from Fig. 8.49 and compute $\tau_0 \, (\bar{\sigma}_v = 1t / ft^2)$

4. comparison of τ_{av} and τ_0

● Remark

- A reduction of 10% in the shear stress causing initial liquefaction may be used to account for the 3-direction vibration effect

- It appears that correlation of standard penetration records with the behavior of sands may be the solution to liquefaction problems of the future