Liquefaction Evaluation

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Acknowledgements

The Liquefaction Evaluation section is prepared mainly following:

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•Bozorgnia, Y. and Bertero, V.V. , Eds. (2004). "Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering," Ch. 4: Geotechnical Aspects of Seismic Hazards, by S. L. Kramer and J. Stewart, CRC Press, 976 pages.
•Youd, T. L., and Idriss, I. M., eds. (1997). NCEER Workshop Proc. on Evaluation of Liquefaction Resistance of Soils, Natl. Center for Earthquake Engineering Research (NCEER), State Univ. of New York at Buffalo, NY.
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•Martin, G. R., and Lew, M., eds. (1999). Recommended procedures for implementation of DMG Special Publication 117: Guidelines for analyzing and mitigating liquefaction hazards in California, Southern, California Earthquake Center, University of Southern California, Los Angeles, California.

Updates and newer additional information can be found in:

•R. B. Seed, K. O. Cetin, R. E. S. Moss, A. M. Kammerer, J. Wu, J. M. Pestana, M. F. Riemer, R.B., Sancio, J.D. Bray, R. E. Kayen, and A. Faris (2003). "Recent Advances in SOIL Liquefaction Engineering: A Unified Consistent Framework," 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Keynote Presentation, H.M.S. Queen Mary, Long Beach, California, April 30, 2003.

•Idriss, I. M., and Boulanger, R. W. (2004). "Semi-Empirical Procedures for Evaluating Liquefaction Potential During Earthquakes", Invited paper, 11th Intl. Conf. on Soil Dyn. and Eq. Eng ., and 3rd Intl. Conf. on Eq. Geotech. Eng., Jan. 7-9, Berkeley, CA, pp 32-56.

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Types of liquefaction

1. Flow liquefaction

• Occurs when shear stress required for equilibrium of a soil mass (the static shear stress) is greater than the shear strength (residual strength) of the soil in its liquefied state.

• Potentially very large post-liquefaction lateral deformations are driven by the static shear stress.

Types of liquefaction (cont'd)

2. Cyclic mobility

• Occurs when the static shear stress is less than the shear strength of the liquefied soil.

• Deformations are driven by both cyclic and static shear stresses.

• Deformations develop incrementally during earthquake shaking.

When is the soil liquefied ….

At a given site, typically manifestations include sand boils, large lateral deformation, and significant settlement.

For technical assessments, the "liquefaction" state is reached when the effective confining stress goes down to zero (i.e., the original effective confining stress has gradually decreased and has become "excess pore-water pressure" known as u_e).

At this state, the value of the "excess pore pressure ratio" r_u is 1.0 where $r_u = u_e / \sigma'_v$ and σ'_v is the initial effective vertical stress.

In addition, technically, liquefaction may be described by a soil sample building up pore-pressure and reaching a shear strain of 3%-5% or more in a laboratory shear test.

Why does liquefaction occur

If the soil is loose and is being shaken, the particles will settle due to gravity. When the soil is saturated, the pore-water is unable to move out of the way quickly enough (because the soil permeability is relatively low), and more and more particles start to partially float in the water (this leads to excess pore-pressure buildup). Eventually as shaking continues, the particles float in the water temporarily as they settle downwards and reach a new densified and consolidated state.

Soils Susceptible to liquefaction

Most susceptible would be very loose cohesionless soils. The low permeability of non-plastic silts and sands is a disadvantage.

Higher permeability, higher relative density, and higher cohesion (plasticity) reduce the susceptibility.

Notes:

1) Objectionable deformations might still occur if r_u values are high, even if liquefaction does not occur). Looser soils are more vulnerable.

2) As pore pressure builds-up, stratified soil profiles (particularly with permeability contrasts) may cause water to be temporarily trapped under a relatively impervious layer or seam (e.g., due to alluvial or hydraulic fill construction, or presence of an upper clay stratum), generating a low friction interface and possibly leading to major lateral deformations. This mechanism actually is a driver of what we commonly observe as sand boils where this water escapes upwards through any available high permeability locale (e.g., taking advantage of a crack in the ground, or similar imperfection, …).

Evaluation of Liquefaction Potential and Consequences

I. Is the soil susceptible to liquefaction?

II. If the soil is susceptible, will liquefaction be triggered?

1) Cyclic stress approach (will be further discussed below)

2) Other methods (please see Refs. on page 2): effective-stress response analysis approach, cyclic strain approach, energy dissipation approach, probabilistic approach.

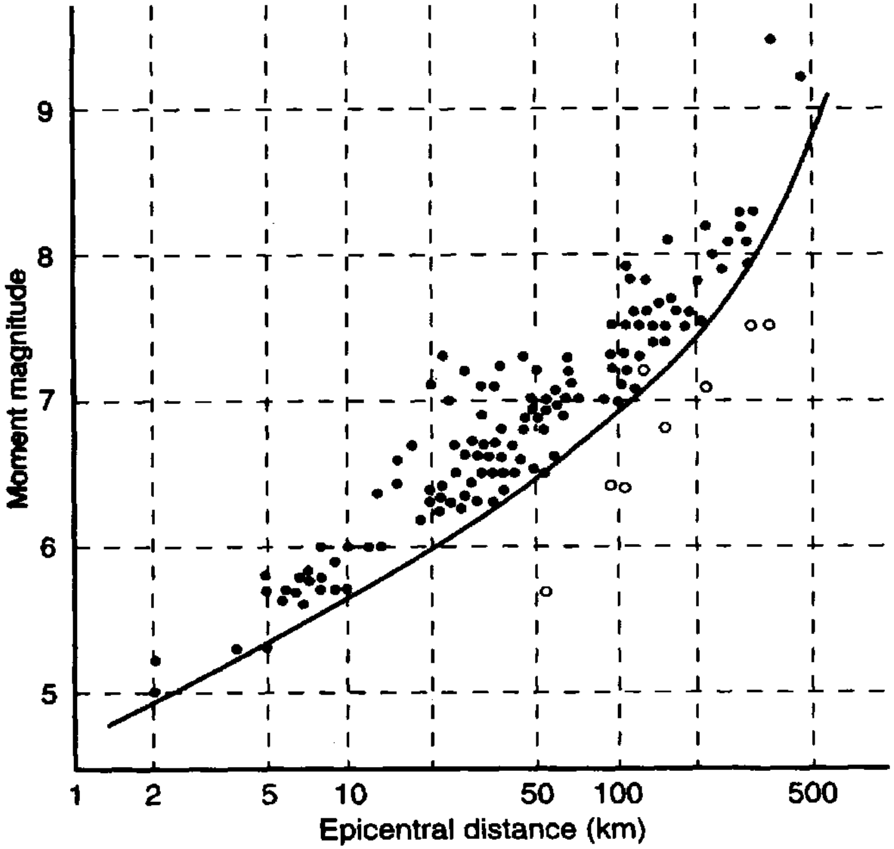
III. If liquefaction is triggered, how much damage will occur?

- Settlements
- Lateral deformations due to cyclic mobility: a) empirical approach, and b) effective-stress response analysis approach
- Flow Failure (see Kramer 1996).

I. Is the soil susceptible to liquefaction?

1. Historical criteria

The epicentral distance to which liquefaction can be expected, increases with increasing earthquake magnitude.



From Kramer (1996)

Figure 1. Relationship between limiting epicentral distance of sites at which liquefaction has been observed and moment magnitude for shallow earthquakes. Deep earthquakes (focal depths > 50 km) have produced liquefaction at greater distances. After Ambraseys (1988).

I. Is the soil susceptible to liquefaction? (cont'd)

2. Geologic criteria

- Depositional environment - Saturated loose fluvial, colluvial, and aeolian deposits are more susceptible to liquefaction.
- Age - Newer soils are more susceptible to liquefaction than older soils.
- Water table - Liquefaction susceptibility decreases with increasing groundwater depth.
- Human-constructed soil strata - Uncompacted soils (e.g., hydraulic fill) are more susceptible to liquefaction than compacted soils.

I. Is the soil susceptible to liquefaction? (cont'd)

3. Compositional criteria

- Grain size and plasticity characteristics - Sands, non-plastic silts, and gravelly soils, under conditions of low permeability, are susceptible to liquefaction.

- Gradation - Well graded soils are less susceptible to liquefaction than poorly graded soils.

- Particle shape - Soils with rounded particles are more susceptible to liquefaction than soils with angular particles.

I. Is the soil susceptible to liquefaction? (cont'd)

4. Initial stress state criterion (for flow liquefaction)

- A loose soil will be susceptible to flow liquefaction only if the static shear stress exceeds its steady state (or residual) strength.
- Residual strength can be estimated as shown in Figure 2.

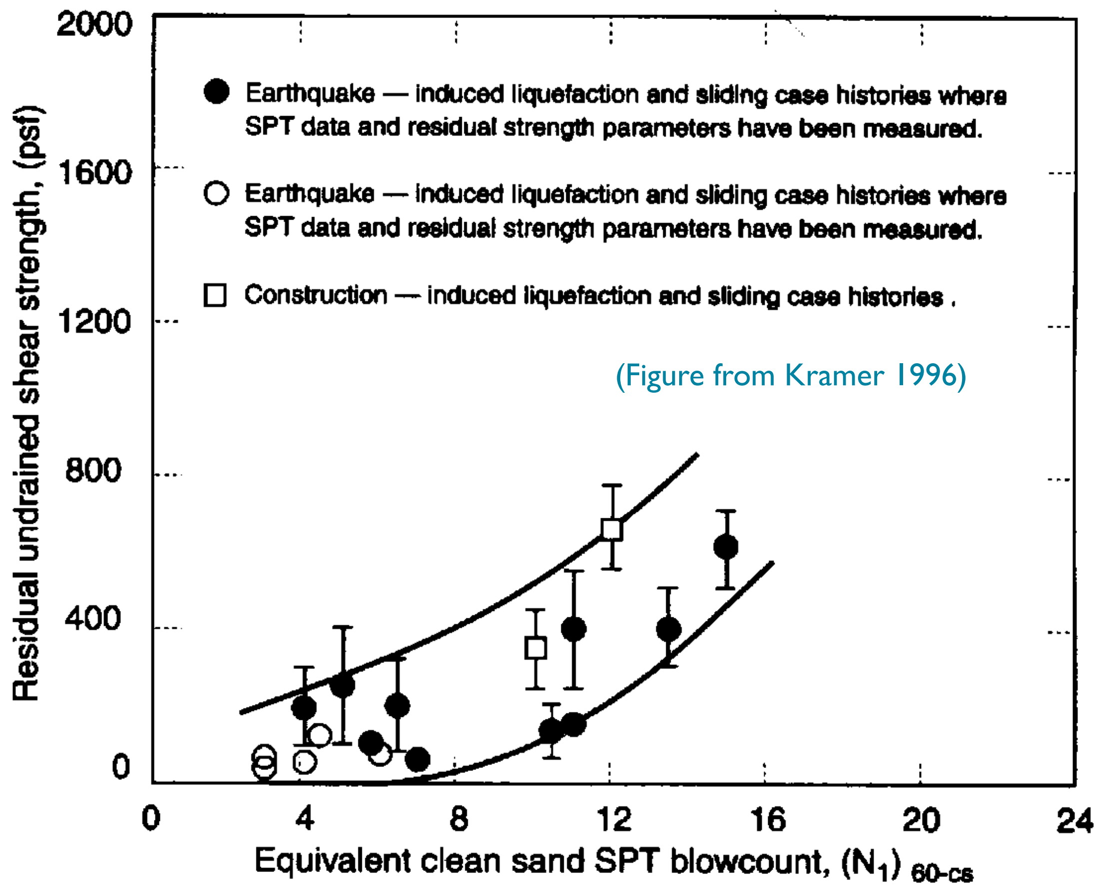


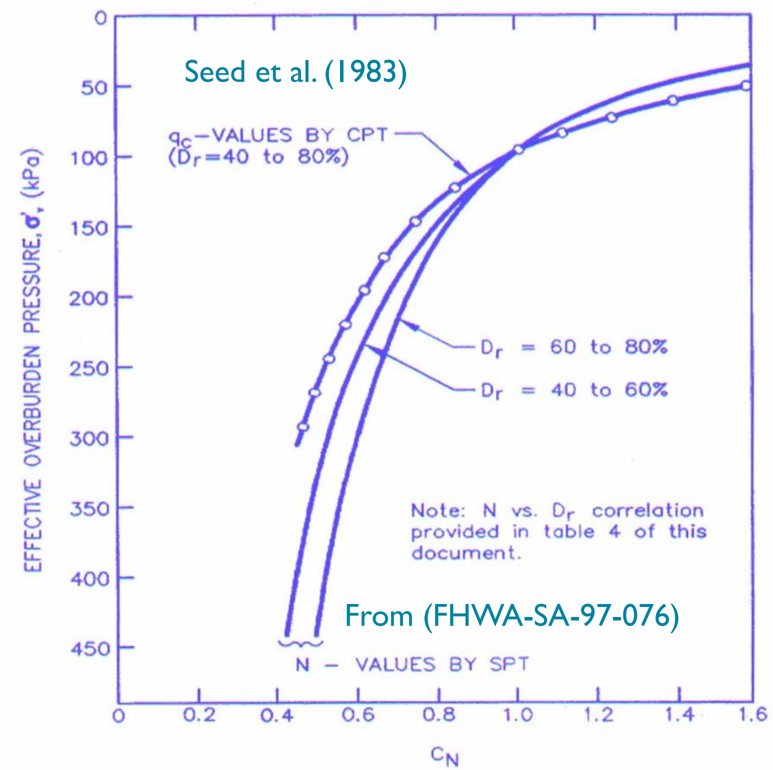
Figure 2. Relationship between residual strength and corrected SPT resistance. (After Seed and Harder, 1990. H. Bolton Seed Memorial Symposium Proceedings, Vol. 2, p. 371. Used by permission of BiTech Publishers, Ltd.)

In Fig. 2 above, $(N_1)_{60\text{-}cs} = (N_1)_{60} + N_{corr}$

where, *cs* stands for "clean sand equivalent, N_{corr} may be obtained from the table below for instance, and $(N_1)_{60}$ is the SPT blow count number normalized to an overburden pressure of 1 ton/ft^2 (96 kPa) and corrected to an energy ratio of 60%.

Note: $(N_1)_{60} = C_N\, N_{60}$ (see below)

$N_{60} = N\, C_{60}$ (see next page)



(Table from Kramer 1996)

Table 1. Recommeded Fines Correction for Estimation of Residual Undrained Strength by Seed–Harder and Stark–Mesri Procedures

	N_{corr} (blows/ft)	
Percent Fines	Seed–Harder	Stark–Mesri
0	0	0
10	1	2.5
15	—	4
20	—	5
25	2	6
30	—	6.5
35	—	7
50	4	7
75	5	7

Comment: All recommendations related to "Fines" continue to be likely to change in the future ..

Correction for	Correction Factor	Reference
Nonstandard Hammer Type (DH = doughnut hammer; ER = energy ratio)	C_{HT}=0.75 for DH with rope and pully C_{HT}=1.33 for DH with trip/auto & ER=80	Seed et al. (1985)
Nonstandard Hammer Weight or Height of Fall (H = height of fall in mm; W = hammer weight in kg)	$C_{HW} = \frac{H \cdot W}{63.5 \cdot 762}$	calculated per Seed et al. (1985)
Nonstandard Sampler Setup (standard samples with room for liners, but used without liners)	C_{SS} = 1.10 for loose sand C_{SS} = 1.20 for dense sand	Seed et al. (1985)
Nonstandard Sampler Setup (standard samples with room for liners, and liners are used)	C_{SS} = 0.90 for loose sand C_{SS} = 0.80 for dense sand	Skempton (1986)
Short Rod Length	C_{RL} = 0.75 for rod length 0-3 m	Seed et al. (1983)
Nonstandard Borehole Diameter	C_{BD} = 1.05 for 150 mm borehole diameter C_{BD} = 1.15 for 200 mm borehole diameter	Skempton (1986)

Notes: N = Uncorrected SPT blow count.

$C_{60} = C_{HT} \cdot C_{HW} \cdot C_{SS} \cdot C_{RL} \cdot C_{BD}$

$N_{60} = N \cdot C_{60}$

C_N = Correction factor for overburden pressure.

$(N_1)_{60} = C_N \cdot N_{60} = C_N \cdot C_{60} \cdot N$

C_{60} from Richardson et al. (1995)

From (FHWA-SA-97-076)

II. If the soil is susceptible, will liquefaction be triggered?

(by cyclic stress approach)

Step 1. Calculate equivalent cyclic shear stress induced by a given earthquake (i.e., the "Demand"). Herein, this is dictated by an expected peak acceleration at the site scaled by a factor of 0.65 based on engineering judgment.

$$\tau_{cyc} = 0.65 \frac{a_{\max}}{g} \sigma_v \, r_d = CSR \, \sigma'_{v0} \qquad (1)$$

where $a_{\max}$ is the peak ground surface acceleration, g the acceleration of gravity, σ_v the total vertical stress, and r_d the value of a stress reduction factor at the depth of interest. r_d may be obtained from Figure 3 below. This equation also defines CSR, the cyclic stress ratio, with σ'_{v0} being the initial vertical effective stress (see also approach based on Arias Intensity (Kayen and Mitchell 1997).

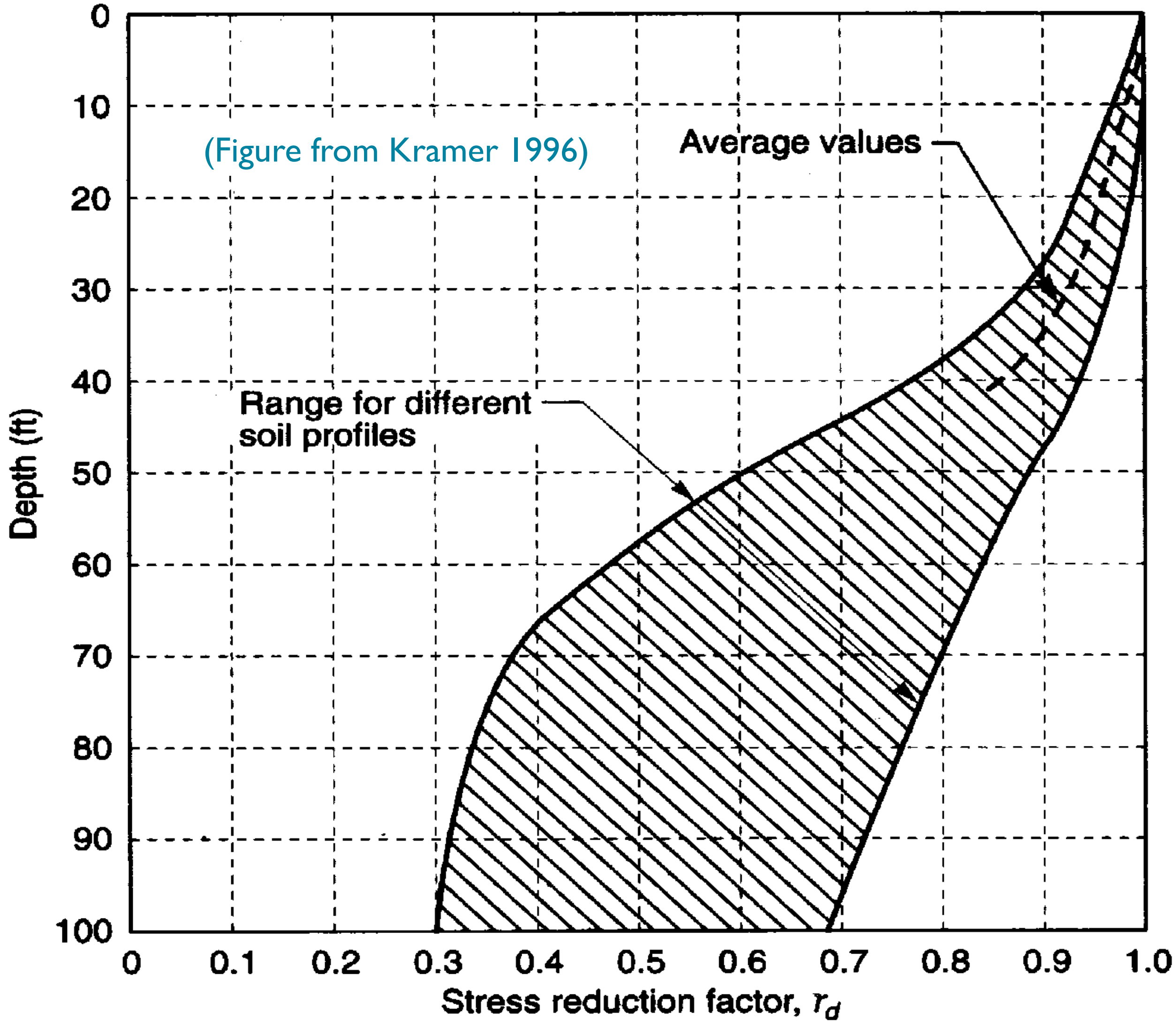


Figure 3. Reduction factor to estimate the variation of cyclic shear stress with depth below level or gently sloping ground surfaces. (After Seed and Idriss, 1971.)

II. If the soil is susceptible, will liquefaction be triggered? (cont'd)

by cyclic stress approach

Step 2. Calculate the cyclic shear stress required to cause liquefaction (i.e., the "Capacity"):

$$\tau_{cyc,L} = CSR_L \, \sigma'_{v0} \qquad (2)$$

where σ'_{v0} is the initial vertical effective stress, CSR_L is the cyclic stress ratio, and may be obtained based on:

- SPT resistance (Fig. 4 for clean sands, Fig. 5 for silty sands).
- CPT resistance (Fig. 8).
- See also references for approaches based on Shear wave velocity (e.g., Andrus and Stokoe 2000).

(Figure from Kramer 1996)

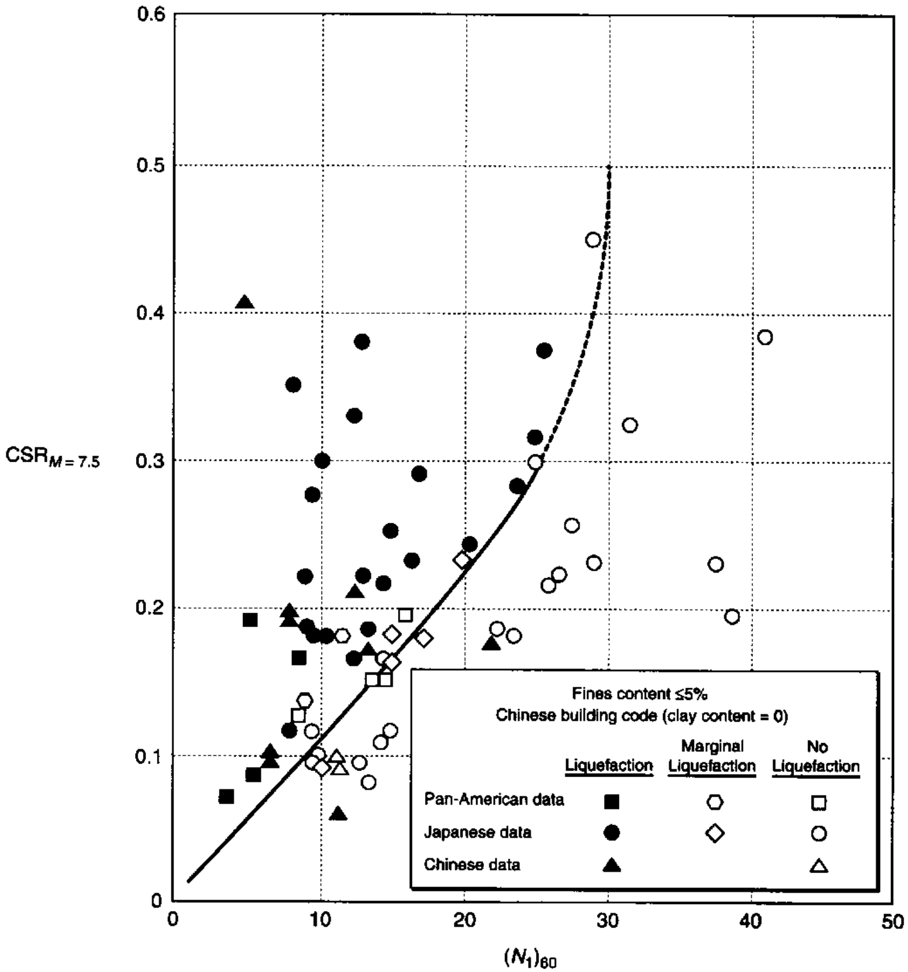


Figure 4. Relationship between cyclic stress ratios causing liquefaction and $(N_1)_{60}$ values for clean sands in $M = 7.5$ earthquakes. (After Seed et al. (1975). Influence of SPT procedures in soil liquefaction resistance evaluations, *Journal of Geotechnical Engineering*, Vol. 111, No. 12. Reprinted by permission of ASCE.)

(Figure from Kramer 1996)

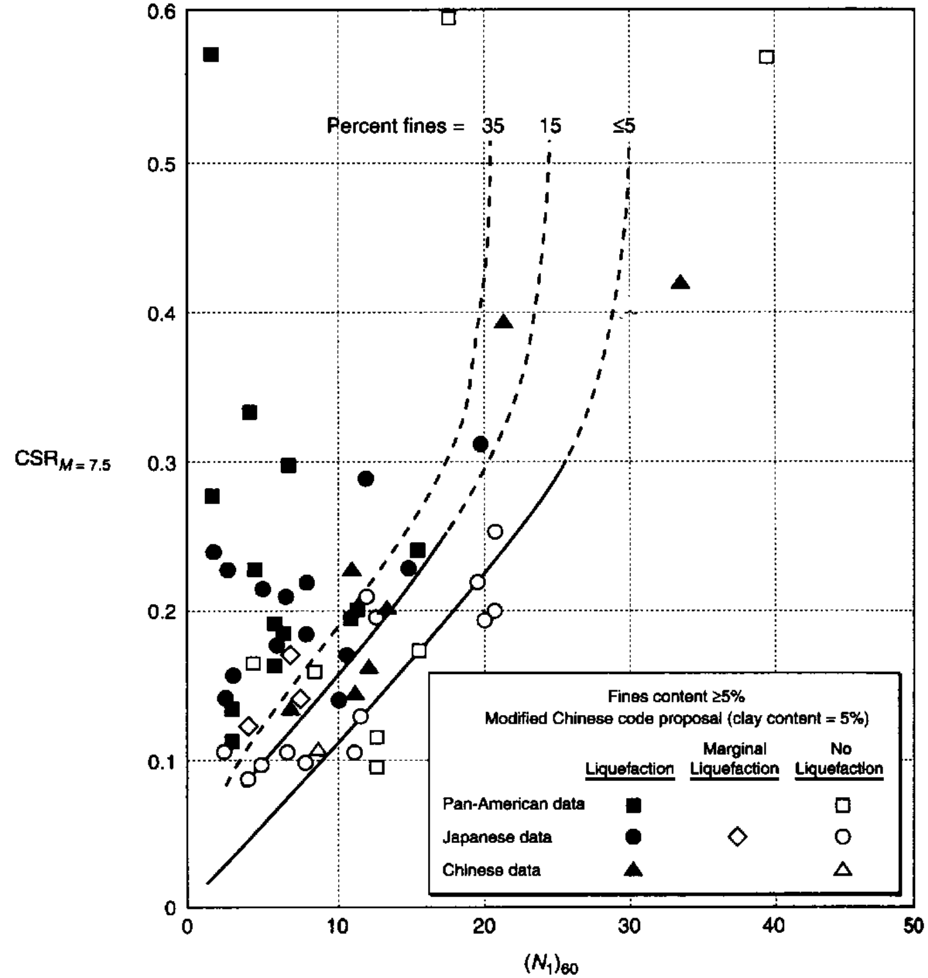


Figure 5. Relationship between cyclic stress ratios causing liquefaction and $(N_1)_{60}$ values for silty sands in $M = 7.5$ earthquakes. (After Seed et al. (1975). Influence of SPT procedures in soil liquefaction resistance evaluations, *Journal of Geotechnical Engineering*, Vol. 111, No. 12. Reprinted by permission of ASCE.)

Notes:

1. Use the following table for earthquake magnitudes other than M=7.5

Table 2. Magnitude Correction Factors for Cyclic Stress Approach

Magnitude, M	$CSR_M/CSR_{M=7.5}$
$5\frac{1}{4}$	1.50
6	1.32
$6\frac{3}{4}$	1.13
$7\frac{1}{2}$	1.00
$8\frac{1}{2}$	0.89

(Table from Kramer 1996)

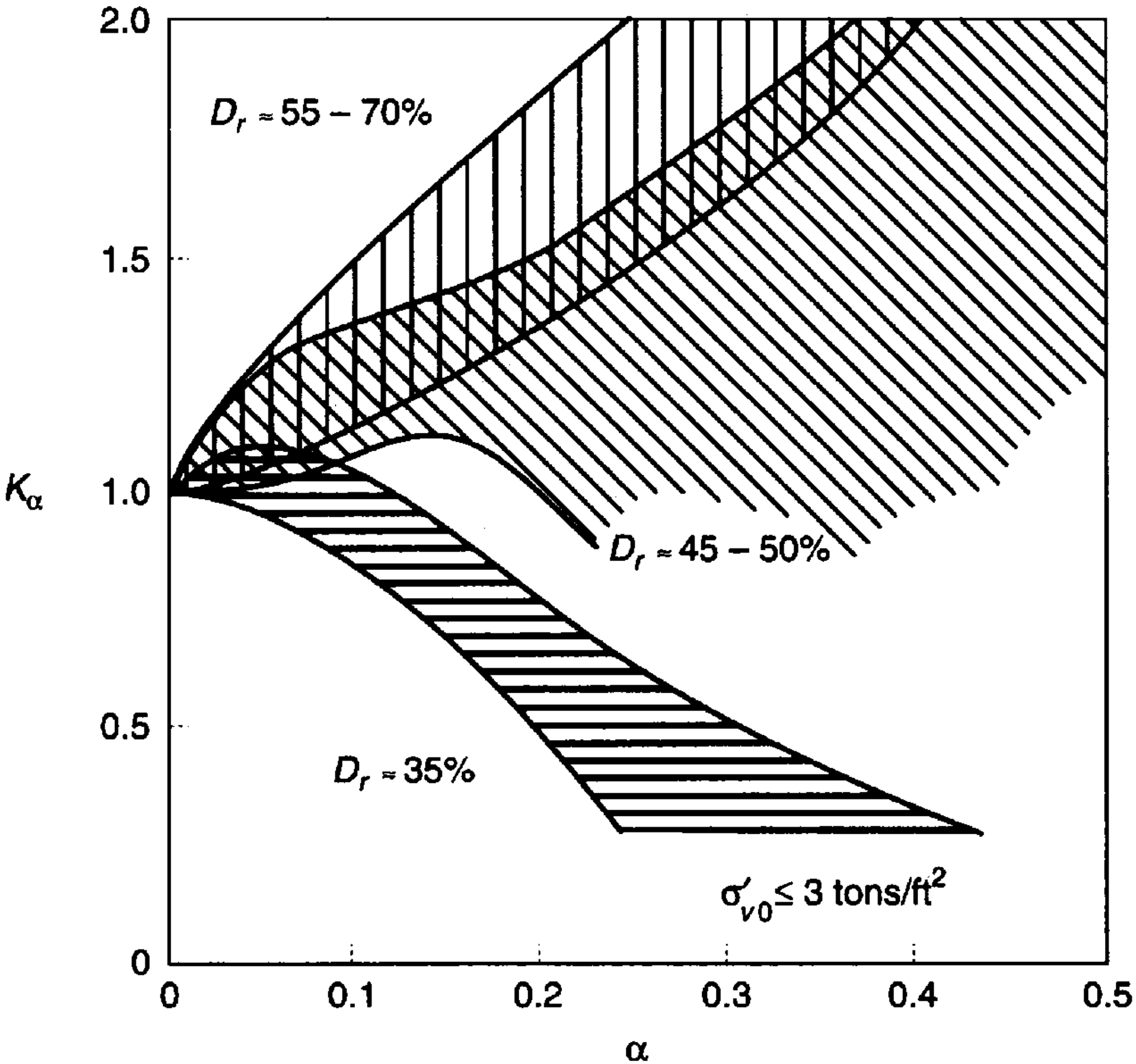
2. The influence of plasticity could be accounted for by multiplying the CSR_L by the factor (Ishihara 1993):

$$F = \begin{cases} 1.0 & \text{PI} \leq 10 \\ 1.0 + 0.022(\text{PI} - 10) & \text{PI} > 10 \end{cases}$$

3. Figs. 4 and 5 are for level-ground sites, and shallow liquefaction. To account for site slope (initial shear stress) and deep liquefaction, modify the CSR_L by:

$$CSR_{\alpha,\sigma} = CSR_L \, K_\alpha \, K_\sigma \qquad (3)$$

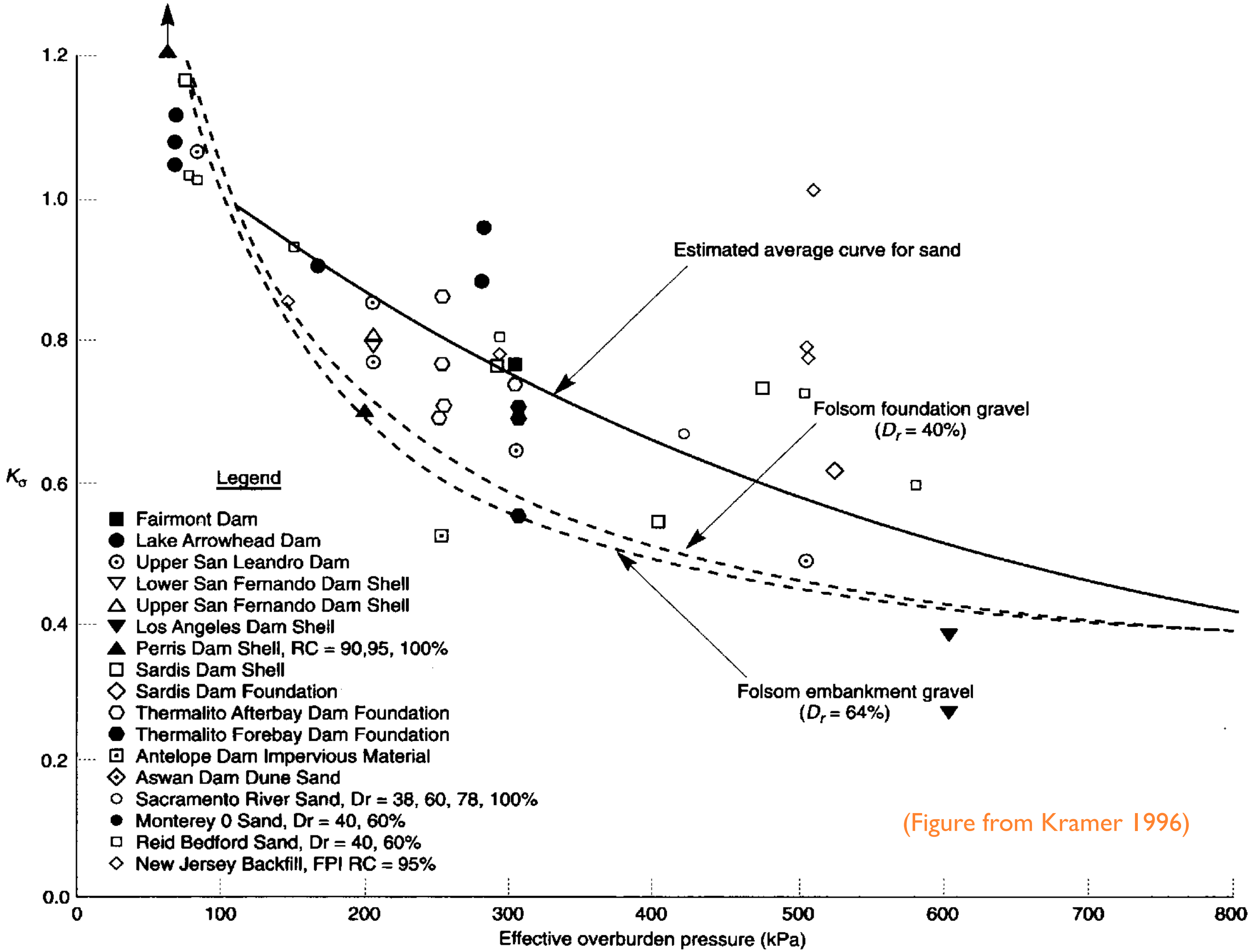
where $\alpha = \tau_{h,static} / \sigma'_{v0}$ and K_α and K_σ are correction factors that may be obtained from Figs. 6 and 7 below. The term $\tau_{h,static}$ is the acting static shear stress (also known as the "Driving Shear Stress").



(Figure from Kramer 1996)

Figure 6. 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.)

Note: The data in this figure is not accepted fully by all experts



(Figure from Kramer 1996)

Figure 7.a Variation of correction factor, K_σ, with effective overburden pressure. (After Marcuson et al., 1990. Used by permission of EERI.)

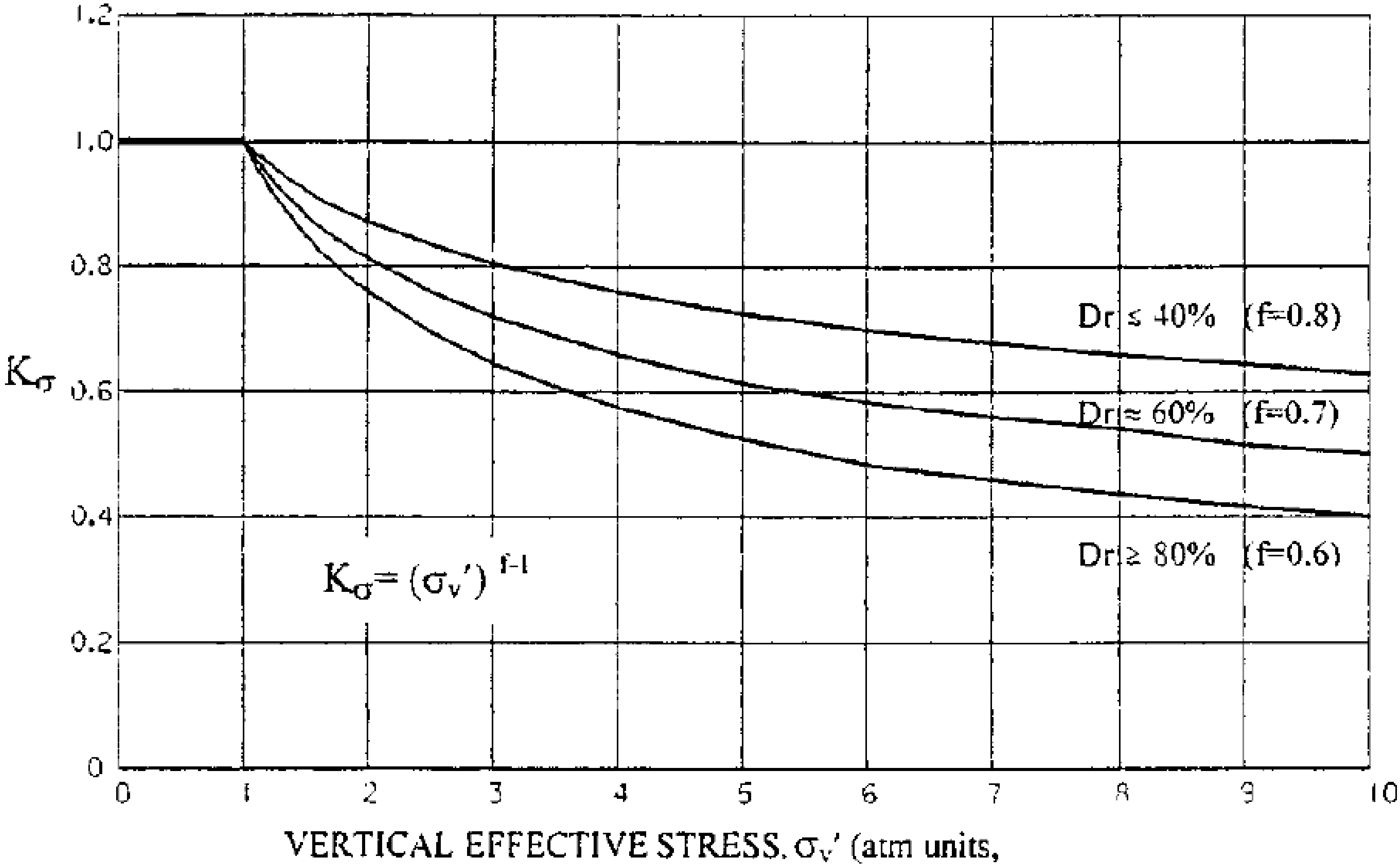


Fig. 7b: Recommended for practice by Youd et al. 2001

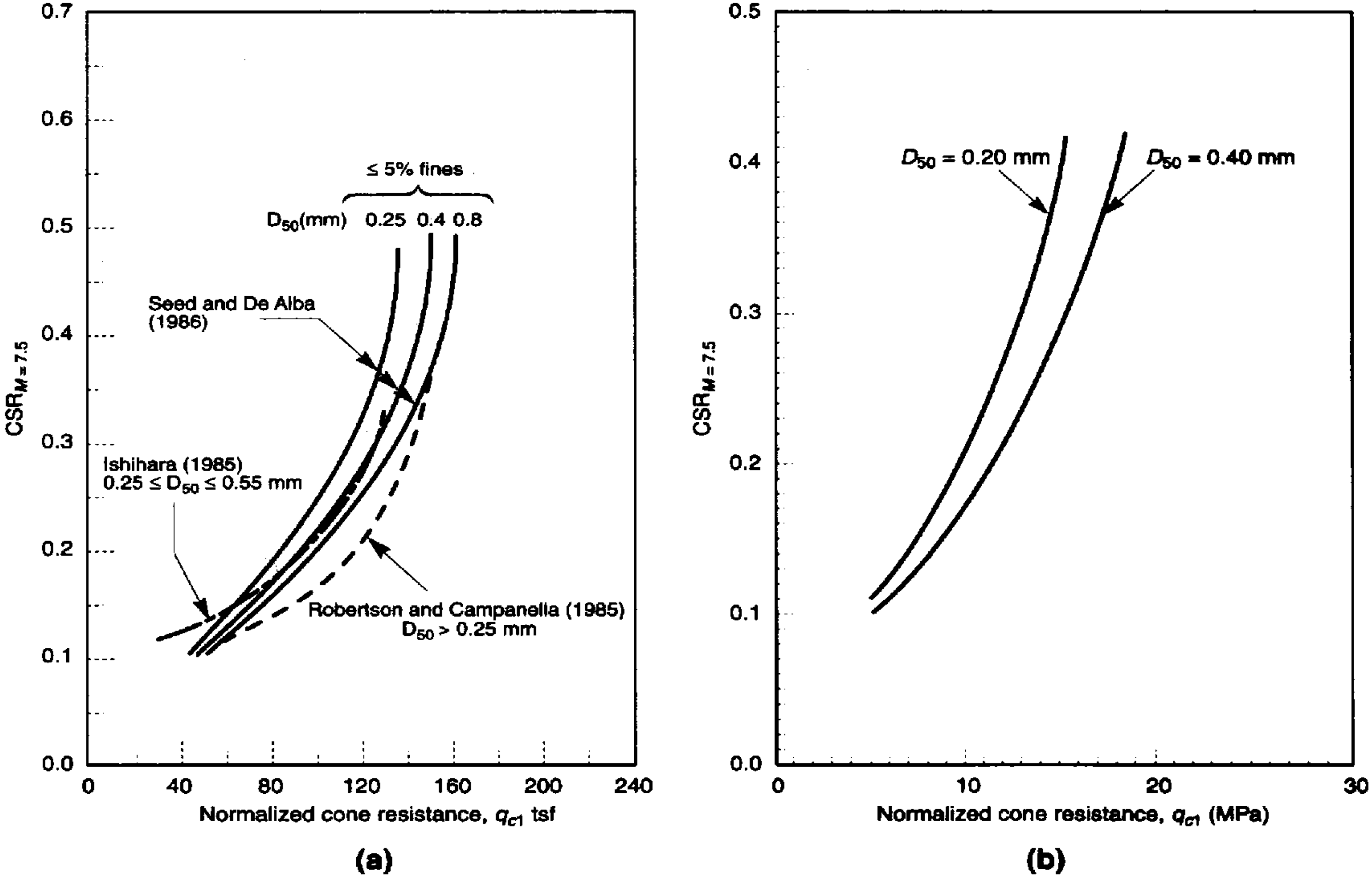


Figure 8.

Figure 9.34 CPT-based liquefaction curves: (a) based on correlations with SPT data; (b) based on theoretical/experimental results. (After Mitchell and Tseng, 1990, H. Bolton Seed Memorial Symposium Proceedings, Vol. 2, p. 347. Used by permission of BiTech Publishers, Ltd.)

(Figure from Kramer 1996)

Note:

1. In Fig. 8, q_{c1} is the tip resistance q_c normalized to a standard effective overburden pressure p_a of 1 ton/ft^2 (96 kPa):

$$q_{c1} = q_c \left(\frac{p_a}{\sigma'_{v0}} \right)^{0.5} \qquad \text{or} \qquad q_{c1} = \frac{1.8}{0.8 + \sigma'_{v0}} q_c$$

where σ'_{v0} is the initial effective overburden pressure.

2. The effects of fines can be accounted for by adding tip resistance increments to the measured tip resistance q_c (Ishihara 1993):

Fines Content (%)	Tip Resistance Increment (tons/ft^2)
≤ 5	0
~ 10	12
~ 15	22
~ 35	40

(Table from Kramer 1996)

3. Use Table 2 for earthquake Magnitudes other than M=7.5

II. If the soil is susceptible, will liquefaction be triggered? (cont'd)

by cyclic stress approach

Step 3. Calculate the safety factor against liquefaction:

$$FS_L = \frac{\tau_{cyc,L}}{\tau_{cyc}} = \frac{CSR_L}{CSR} = \frac{CRR}{CSR} \qquad (4)$$

On this basis, liquefaction will be triggered if $FS_L < 1$.

Note: *CRR* above is Cyclic Resistance Ratio

Another way of Magnitude scaling: To more accurately represent the earthquake shaking energy, Youd et al. (2001) suggested including a Magnitude Scaling Factor of the form:

$MSF = (7.5/M_w)^n$

where M_w is Moment magnitude, and n = 2.56 for M_w = 7.5 or greater, and up to 3.3 for M_w less than 7.5

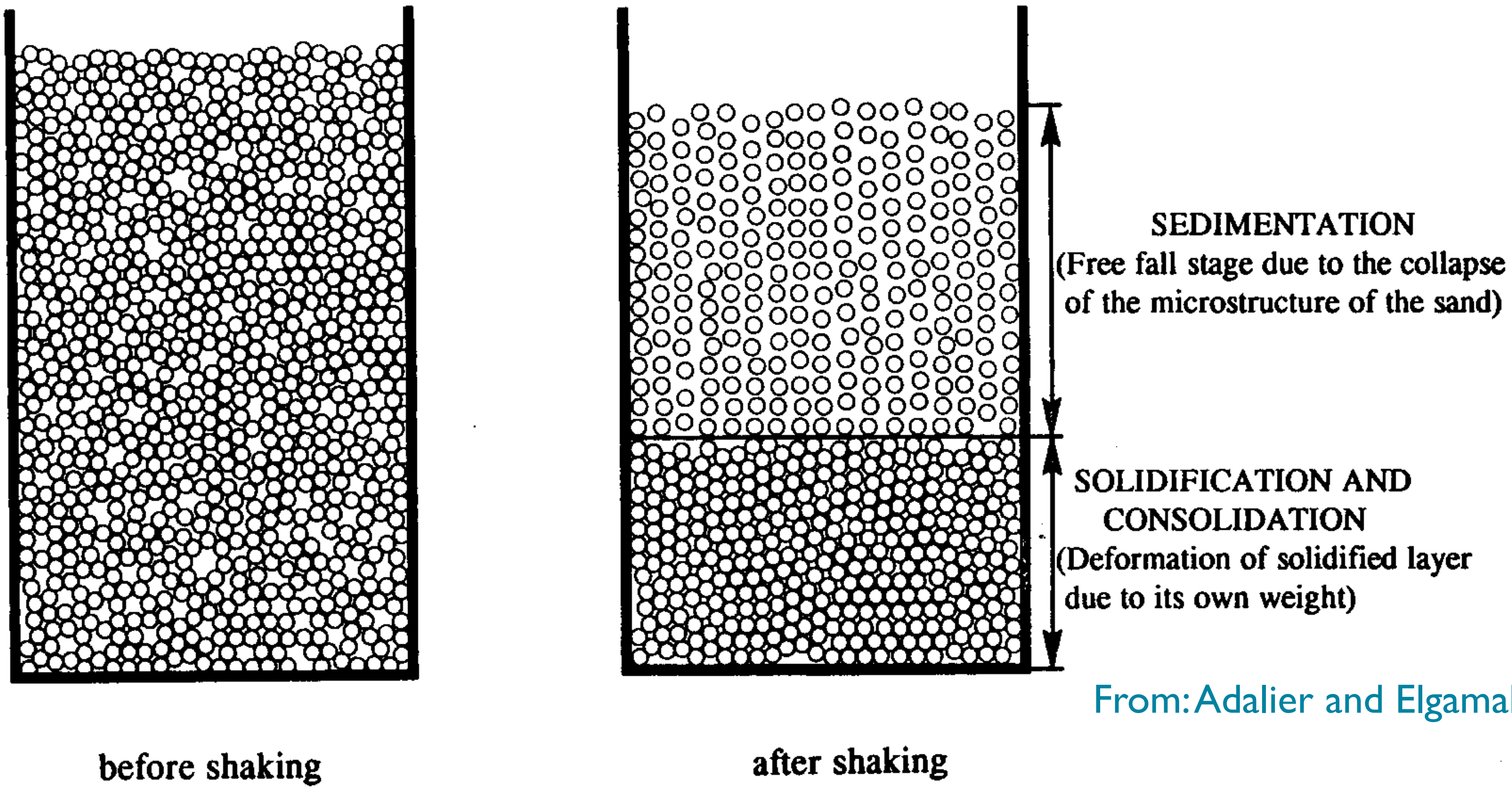
As such, $a_{maxM7.5} = a_{max} / MSF$

and

$$\tau_{cycM\,7.5} = 0.65 \frac{a_{\max M\,7.5}}{g} \sigma_v \, r_d = CSR_{M\,7.5}\, \sigma'_{v0}$$

With this adjustment, both CSR and CSR_L can be compared directly for M=7.5

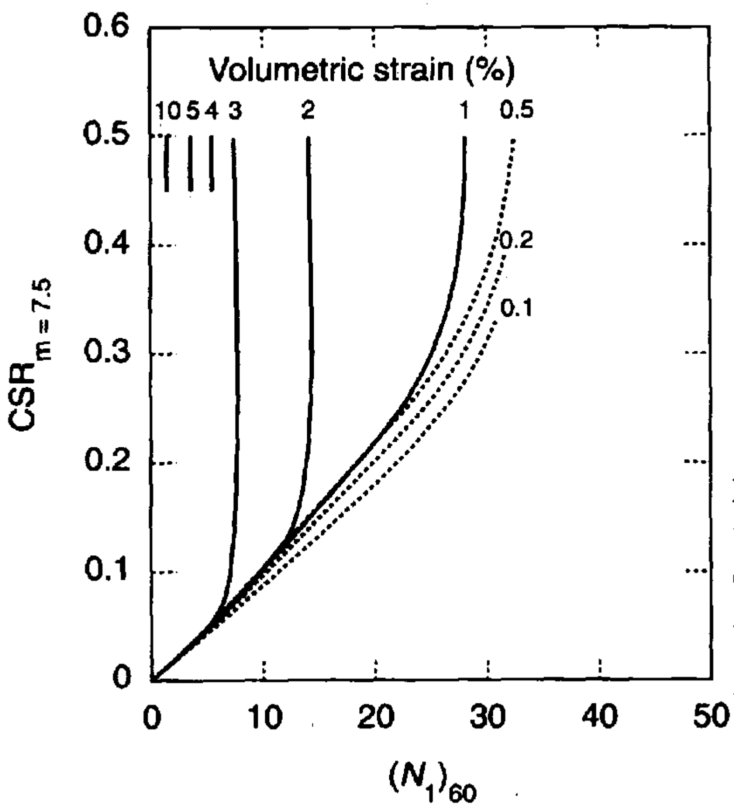
If liquefaction is triggered, how much Settlement occurs?



From: Adalier and Elgamal

III. If liquefaction is triggered, how much settlement occurs?

Settlement by Tokimatsu-Seed method



(Figure from Kramer 1996)

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.)

Table 9-2 Magnitude Correction Factors for Cyclic Stress Approach

Magnitude, M	$CSR_M/CSR_{M=7.5}$
$5\frac{1}{4}$	1.50
6	1.32
$6\frac{3}{4}$	1.13
$7\frac{1}{2}$	1.00
$8\frac{1}{2}$	0.89

(Table from Kramer 1996)

To use Fig. 9.53, the *CSR* can be calculated from Equation (1). For earthquake magnitudes other than 7.5, the *CSR* should be modified according to the Table above.

III. If liquefaction is triggered, how much settlement occurs?

Settlement by Ishihara-Yoshimine method

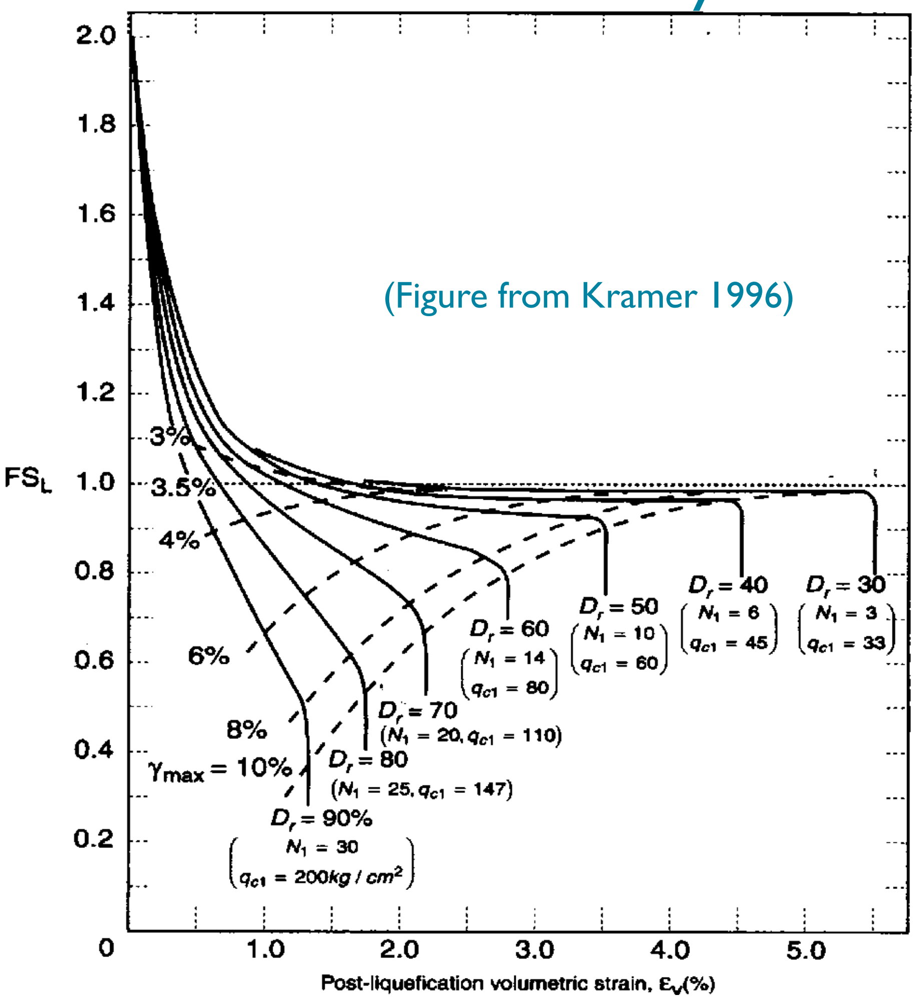


Figure 9.54 Chart for estimating postliquefaction volumetric strain of clean sand as function of factor of safety against liquefaction or maximum shear strain. (After Ishihara and Yoshimine, 1992; used by permission of JSSMFE.)

To use Fig. 9.54, the FS_L can be calculated using Equation (4).

In this Figure, Note $N_1 = 0.833(N_1)_{60}$

III. If liquefaction is triggered, how much lateral deformation occurs?

Estimates can be based on Residual Strength (see Fig. 2). In addition, for the residual shear strength s_r, Olson and Stark (2002) proposed:

$s_r/\sigma'_{vo} = 0.03 + 0.0075\,(N_1)_{60}$ plus or minus 0.03

for $(N_1)_{60}$ less or equal to 12

and

$s_r/\sigma'_{vo} = 0.03 + 0.0143\,(q_{c1})$ plus or minus 0.03

for q_{c1} less than or equal to 6.5 MPa

Earlier, Baziar and Dobry (1995) proposed for loose silty sands:

$s_r = 0.12 - 0.19\,(\sigma'_{vo})$

Summary of SPT-Based Empirical Method
NCEER/NSF Proceedings (Youd et al., 2001)

Step 1 – Discretize boring log into a series of soil layers;

Step 2 – For each soil layer, compute the vertical total stress (σ_{vo}) and vertical effective stresses (σ'_{vo});

Step 3 –Determine Moment Magnitude and Peak Ground Acceleration (a_{max}) for project site;

Step 4 – Compute the shear stress reduction coefficient, r_d ;

Step 5 – Compute the Cyclic Stress Ratio, CSR;

Step 6 – Compute $(N_1)_{60}$ the SPT blow count normalized to overburden pressure of 100 kPa (1ton/sq ft) and hammer energy ratio or hammer efficiency of 60%;

Step 7 – Adjust $(N_1)_{60}$ to account for fines content (FC) by calculating the equivalent clean sand value, $(N_1)_{60CS}$;

Step 8 – Calculate the Cyclic Resistance Ratio for Magnitude 7.5 earthquake, $CRR_{7.5}$;

Step 9 – Calculate the Magnitude Scaling Factor, MSF;

Step 10 – Calculate the Factor of Safety (FS) against liquefaction; and

Step 11 – Calculate the volumetric strain / settlement within each liquefied layer.

See Idriss and Boulanger (2008) EERI Monograph for Additional details (Courtesy of Dr. M. Fraser)

SPT-Based Empirical Method – Idriss & Boulanger, 2008

Step 1 – Discretize boring log into a series of soil layers;

Step 2 – For each soil layer, compute the vertical total stress (σ_{vo}) and vertical effective stresses (σ'_{vo});

Step 3 –Determine Moment Magnitude and Peak Ground Acceleration (a_{max}) for project site;

Step 4 – Determine the shear stress reduction coefficient, r_d ;

Step 5 – Compute the Cyclic Stress Ratio, CSR;

Step 6 – Compute $(N_1)_{60}$ the SPT blow count normalized to overburden pressure of 100 kPa (1ton/sq ft) and hammer energy ratio or hammer efficiency of 60%;

Step 7 – Adjust $(N_1)_{60}$ to account for fines content (FC) by calculating the equivalent clean sand value, $(N_1)_{60CS}$;

Step 8 – Calculate the Cyclic Resistance Ratio for Magnitude 7.5 earthquake, $CRR_{7.5}$;

Step 9 – Calculate the Magnitude Scaling Factor, MSF;

Step 10 – Adjust the Cyclic Resistance Ratio for actual earthquake Magnitude and overburden stress ($CRR_{M,\sigma' vc}$);

Step 11 – Calculate the Factor of Safety (FS) against liquefaction; and

Step 12 – Calculate the volumetric strain / settlement within each liquefied layer.