Liquefaction

CIVL7137/8137

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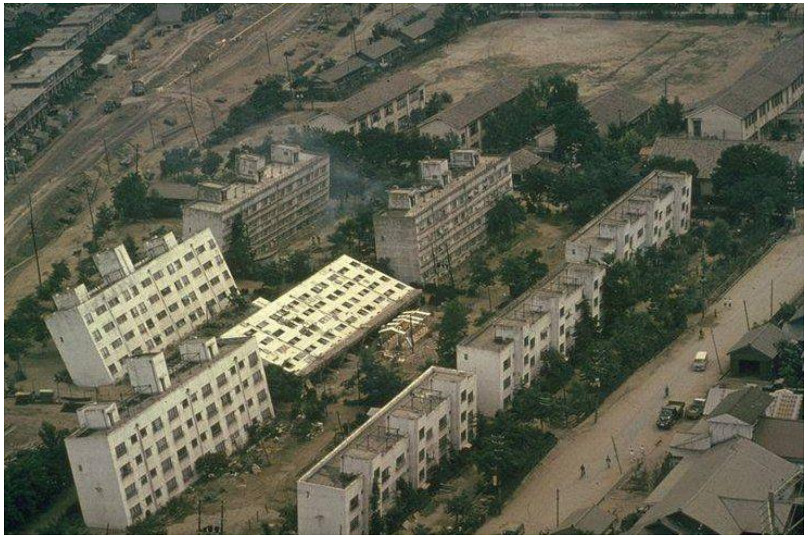
Outline

- Liquefaction-related ground failure
- Liquefaction behavior of soil
- Simplified methods for determining liquefaction potential

Description of Liquefaction

- Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading.
- Liquefaction and related phenomena have been responsible for tremendous amounts of damage in historical earthquakes around the world.
  - Turnagain Heights, Alaska (1964)
  - Niigata, Japan (1964)
  - San Francisco, California (1989)
  - Kobe, Japan (1995)



Liquefaction

- The term liquefaction was introduced by Mogami and Kubo (1953).
- Has historically been used in conjunction with a variety of phenomena that involve soil deformations caused by monotonic, transient, or repeated disturbance of saturated cohesionless soil under undrained conditions.
- The generation of excess pore pressure under undrained loading conditions is a hallmark of liquefaction phenomena.
- The tendency of dry cohesionless soils to densify under both static and cyclic loading is well known.

Liquefaction

- When cohesionless soil is saturated, rapid loading occurs under undrained conditions, so the tendency for densification causes excess pore pressure to increase and effective stresses to decrease

Liquefaction

- In order for liquefaction to occur you must have a layer of soil which is susceptible to liquefaction (a loose sand, gravel, or non-plastic silt) which is saturated.
- In addition, this layer of loose, saturated soil must be bounded or contained by layers which are relatively impermeable with respect to the liquefiable layer.
- Assuming the stress conditions in the soil profile have been constant for a long time (i.e., all consolidation has occurred), the soil layers will be at a state of equilibrium (i.e., no excess pore pressure).
- As a result, the overburden stresses supported by the liquefiable layer (soil and any other loads) will be taken by the grain to grain contact of the soil particles.

Liquefaction

- Given the above, this means the liquefiable layer has a specific height, H. Looking at a column of saturated soil in this layer with a height of H, all the volume is taken up by either soil particles or water (both of which are taken as incompressible) and the height of the soil matrix is equal to the height of the layer.
- As the soil column is subjected to the earthquake motions, the loose soil particles tend to move closer together forming a denser soil matrix (i.e. the height of the soil matrix is no longer equal to the height of the layer).
- As this occurs the overburden stress are transferred to the water (i.e. the pore pressure increases) because the height of the column cannot change as it is filled with incompressible materials and the water is trapped and cannot flow out of the column.
- So as the soil matrix densifies (soil matrix height reduces) and the overburden pressures are transferred to the pore water, the available shear strength of the column of soil is reduced and the soil is said to liquefy. So the soil particles are not suspended in the water due to a water pressure (force) but are still in contact and are actually densifying.

Liquefaction

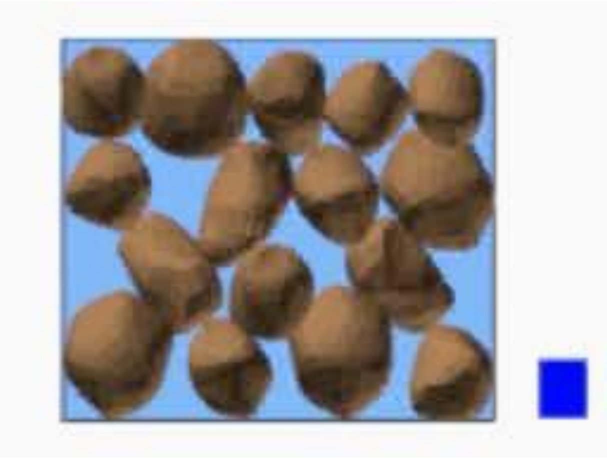
- Because water has no shear strength and the grain to grain shear strength of the soil particles is reduced, any shear strength demand placed on the liquefiable layer to resist a bearing capacity failure (shallow foundation, deep foundation, slope stability) or provide adhesion (skin friction) will be drastically reduced until the pore pressure dissipates.
- Also, the settlements associated with liquefaction result from the densification of the soil matrix and the subsequent dissipation of the pore pressure (outflow of trapped groundwater).

Causes of Soil Liquefaction

- If saturated sand is subjected to ground vibrations, it tends to compact and decrease in volume, if drainage is unable to occur, the tendency to decrease in volume results in an increase in pore water pressure, and if the pore water pressure builds up to the point at which it is equal to overburden pressure, the effective stress becomes zero, the sand loses its strength completely and it develops a liquefied state.

Causes of Soil Liquefaction

- Liquefaction is induced by cyclic, undrained loading of loose, saturated coarse-grained soil, which causes excess pore water pressure to develop and effective stresses to reduce.

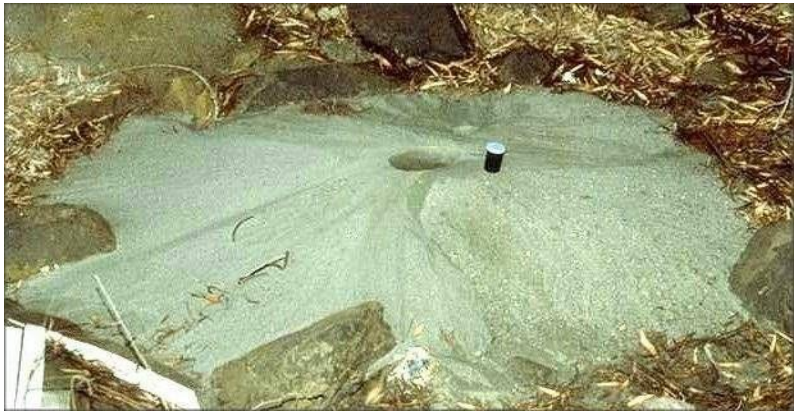


Schematic behavior of sand grains in a soil deposit during liquefaction.
The blue column represents the pore water pressure.

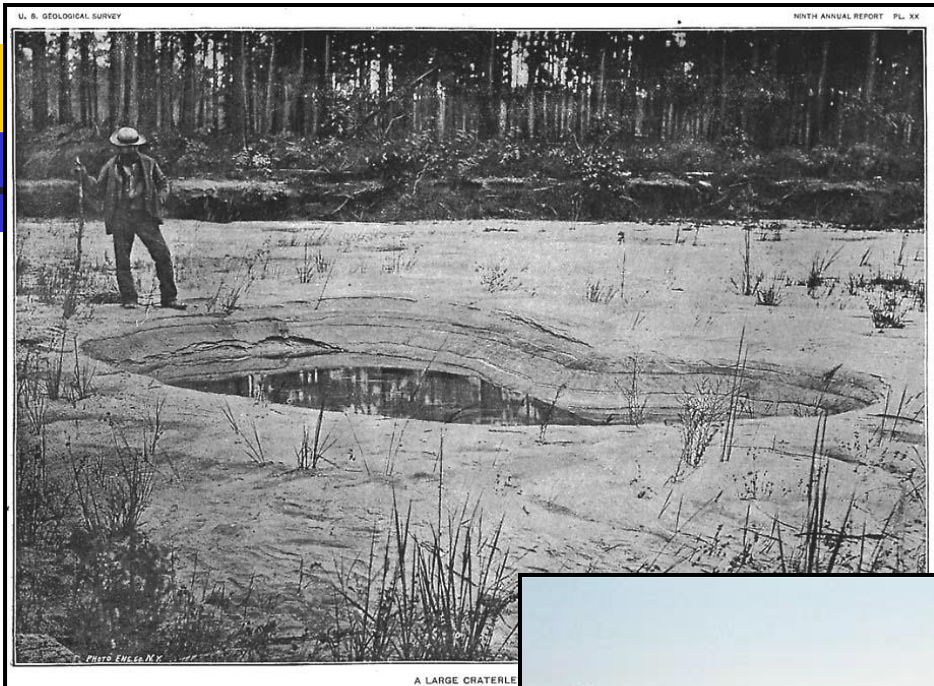
Liquefaction

Liquefaction-Related Ground Failures

- Sand boils – upward venting of excess pore pressure

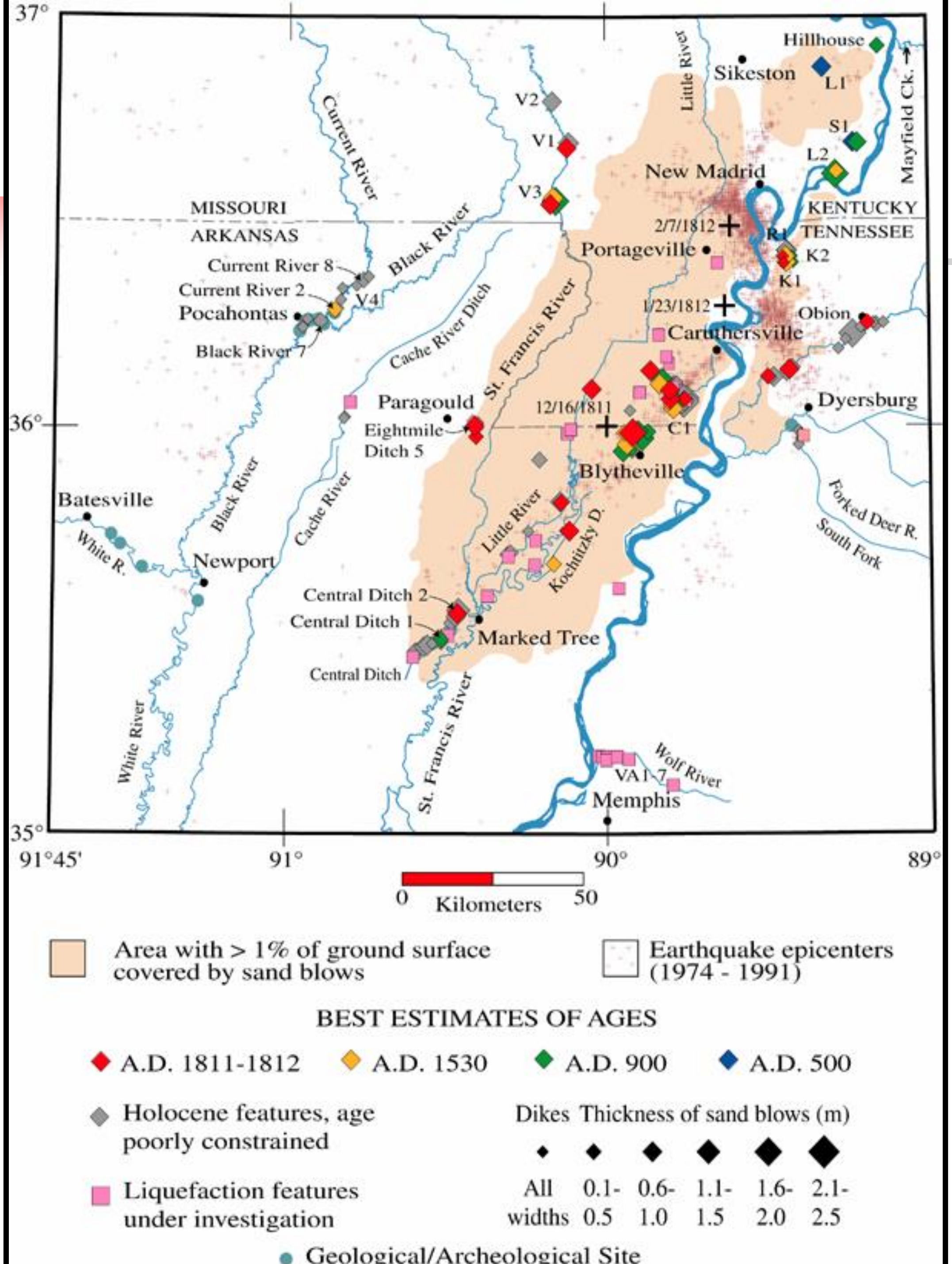


Liquefaction near Umedpar



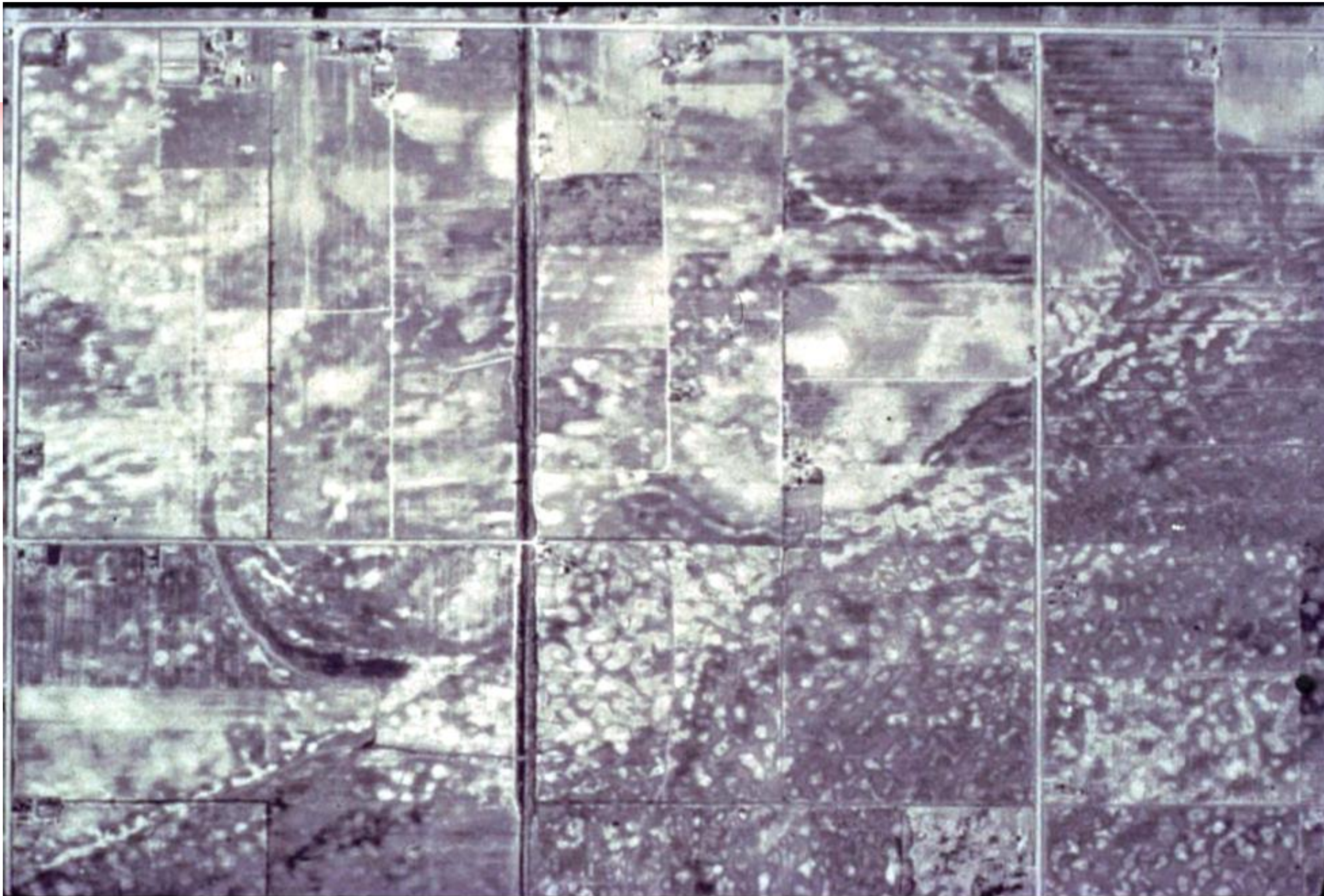
Liquefaction crater near Charleston, SC from the Dutton report (USGS)

Liquefaction crater near India Bridge
Photo by Arch Johnston



Paleoliquefaction sites in the New Madrid seismic zone

Source: Eugene Schwieg USGS



Overprinting of sand blow events near New Madrid, MO

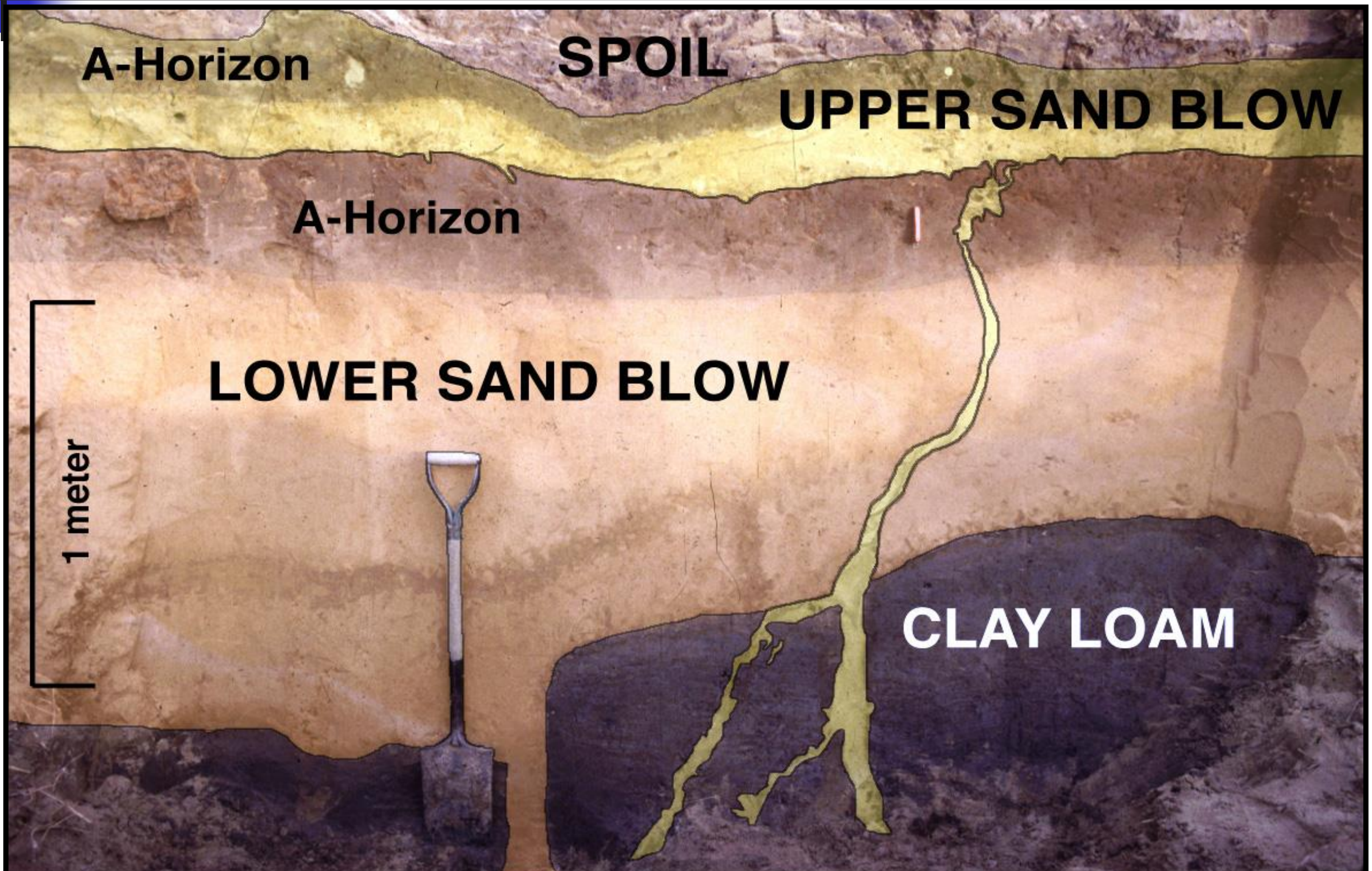


Photo courtesy of E.S. Schweig

Liquefaction-Related Ground Failures

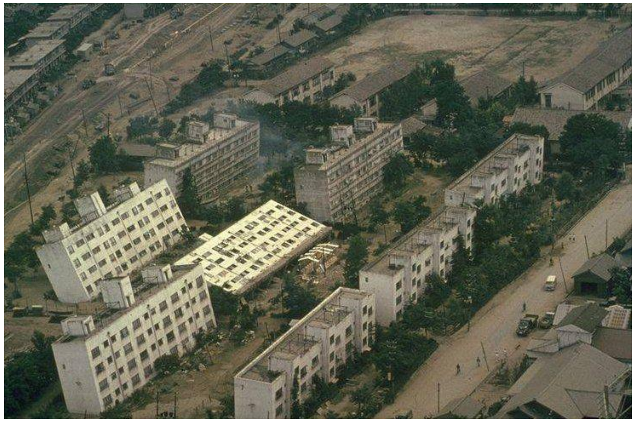
- Lateral Spreads – permanent ground deformations occurring in slopes < 3°
- Liquefied soil exerts higher pressure on retaining walls, which can cause them to tilt or slide.
- This movement can cause settlement of the retained soil and destruction of structures on the ground surface.

Liquefaction-Related Ground Failures

- Flow Failure – Gravity-driven failure in slopes > 3° caused by reduced shear strength in liquefied layer.
- Increased water pressure can also trigger landslides and cause the collapse of dams.
- Lower San Fernando dam suffered an underwater slide during the San Fernando earthquake, 1971.
- Fortunately, the dam barely avoided collapse, thereby preventing a potential disaster of flooding of the heavily populated areas below the dam.

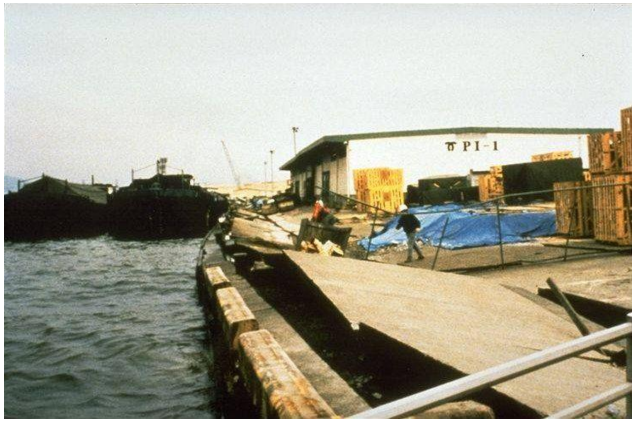
Liquefaction-Related Ground Failures

- Loss of bearing capacity due to reduced shear strength



Liquefaction-Related Ground Failures

- Port and wharf facilities are often located in areas susceptible to liquefaction, and many have been damaged by liquefaction in past earthquakes.
- Most ports and wharves have major retaining structures, or quay walls, to allow large ships to move adjacent to flat cargo handling areas.
- When the soil behind and/or beneath such a wall liquefies, the pressure it exerts on the wall can increase greatly - enough to cause the wall to slide and/or tilt toward the water.
- As illustrated, liquefaction caused major damage to port facilities in Kobe, Japan in the 1995 Hyogo-ken Nanbu earthquake.

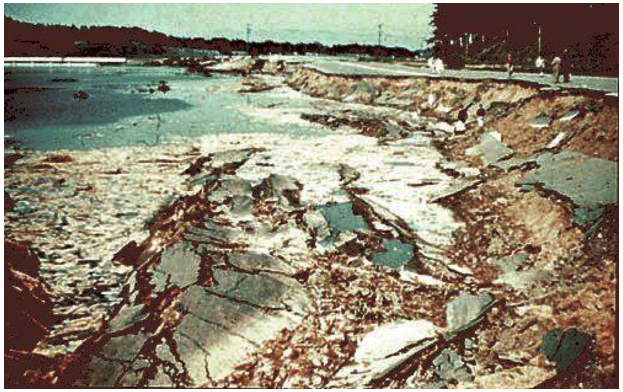


Liquefaction-Related Ground Failures

- Liquefaction also frequently causes damage to bridges that cross rivers and other bodies of water.
- Such damage can have drastic consequences, impeding emergency response and rescue operations in the short term and causing significant economic loss from business disruption in the longer term.
- Liquefaction-induced soil movements can push foundations out of place to the point where bridge spans loose support or are compressed to the point of buckling.

Liquefaction-Related Ground Failures

- Because liquefaction only occurs in saturated soil, its effects are most commonly observed in low-lying areas near bodies of water such as rivers, lakes, bays, and oceans.
- The effects of liquefaction may include major sliding of soil toward the body slumping of water, as in the case of the 1957 Lake Merced slide shown above, or more modest movements that produce tension cracks such as those on the banks of the Motagua River following the 1976 Guatemala Earthquake



Simplified Procedures: 1966 Workshop

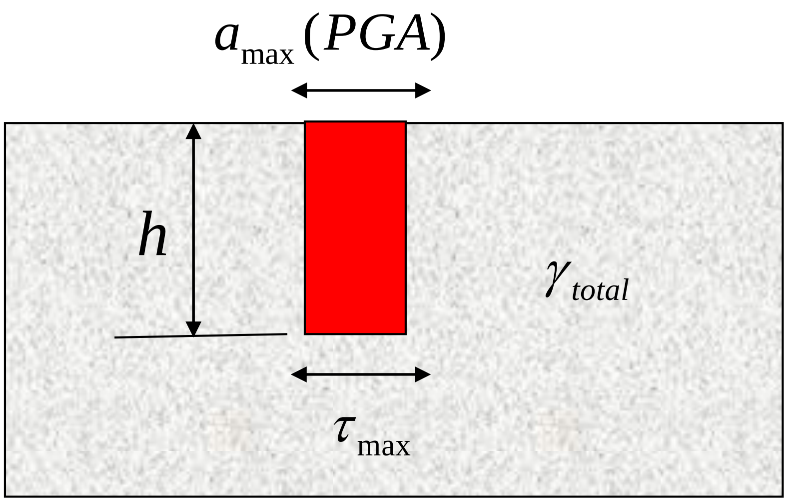
- 1996 National Center for Earthquake Engineering Research (NCEER):
  - Use of the standard and cone penetration tests for evaluation of liquefaction resistance (CPT)
  - Use of shear wave velocity (Vs)
  - Use of Becker Penetration Test (BPT)
  - Magnitude scaling factor
  - Correction factors $K\alpha$ and $K\sigma$
  - Evaluation of seismic factors required for the evaluation procedure

CSR and CRR

- To evaluate liquefaction resistance of soils we need to estimate or calculate:
  - Seismic demand placed on soil
    - Cyclic stress ratio (CSR) and
  - Capacity of the soil to resist liquefaction
    - Cyclic resistance ratio (CRR) – also referred to as liquefaction resistance ratio

Seed and Idriss (1971)

- Assume a rigid soil column and vertically propagating shear waves



Source Dr. Rix

$$\tau_{\max,rigid} = m a_{\max}$$

$$= \left(\frac{\gamma_{total} h}{g}\right) a_{\max} = \sigma_0 \frac{a_{\max}}{g}$$

$$\tau_{\max,\ flexible} = \tau_{\max,rigid}\, r_d$$

$$\tau_{average,\ flexible} = 0.65 \tau_{\max,rigid}\, r_d = 0.65 \sigma_0 \frac{a_{\max}}{g} r_d$$

Seed and Idriss (1971)

- Normalize the shear stress to the effective overburden pressure to calculate the Cyclic Stress Ratio (**CSR**):

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

- The CSR is a simplified measure of liquefaction opportunity (**demand**).

Seed and Idriss (1971)

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

$a_{\max}$ = the peak horizontal acceleration at ground surface

σ_{vo} = total vertical overburden stresses

σ'_{vo} = total effective overburden effective stresses

r_d = Stress reduction coefficient

$r_d = 1.0 - 0.00765z$ for $z \le 9.15m$

$r_d = 1.174 - 0.0267z$ for $9.15 \le z \le 23m$

$r_d = 0.744 - 0.008z$ for $23 \le z \le 30m$

$r_d = 0.50$ for $z > 30m$

z = depth below ground surface in meters

Seed and Idriss (1971)

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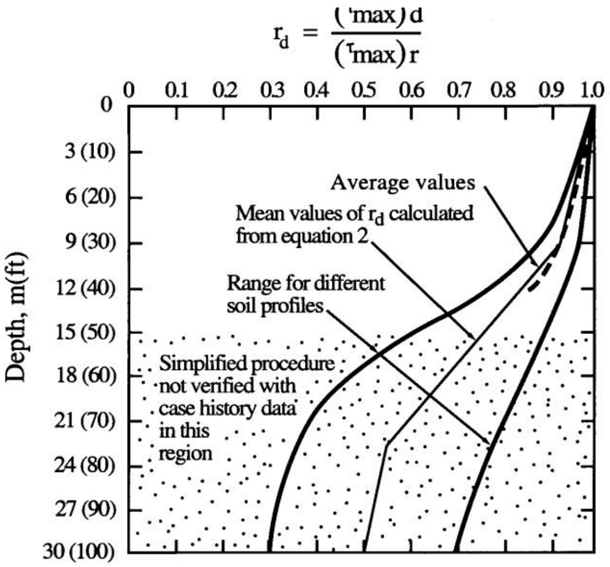


FIGURE 1 r_d **Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines from Equation 2**

Seed and Idriss (1971)

- In the original Seed and Idriss (1971) method, the capacity of soil to resist liquefaction was determined using laboratory cyclic triaxial tests.

- Because of difficulty of obtaining undisturbed specimen of loose, coarse-grained soils below the water table, in situ tests have been replaced laboratory tests:

  - Standard penetration tests (SPT)
  - Cone penetration tests (CPT)
  - Use of shear wave velocity (Vs)
  - Use of Becker Penetration Test (BPT)

Standard Penetration Tests (SPT)

- It is important to correct the raw blow count (N) for various effects such as:
  - Energy ratio
  - Overburden ratio
  - Borehole diameter
  - Rod length
  - Sampling method

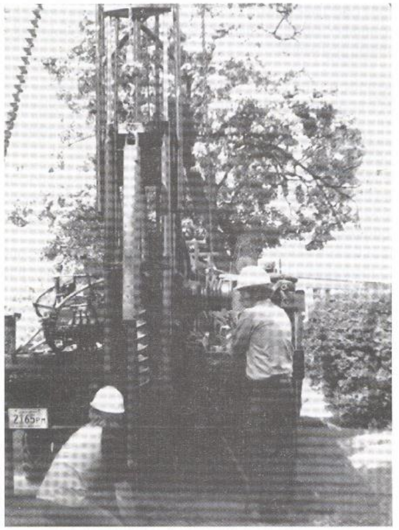


Figure 3-8 Drilling operator performing the SPT test using a safety hammer with rope-cathead lift. Rope is coming off the bottom of cathead and operator is observing for height mark on hammer guide rod. Helper in foreground is taking count and observing penetration.

Standard Penetration Tests (SPT)

- It is common to normalize the standard penetration resistance to vertical effective stress of 1 atmosphere (see figure 6.25 of Cramer's book).

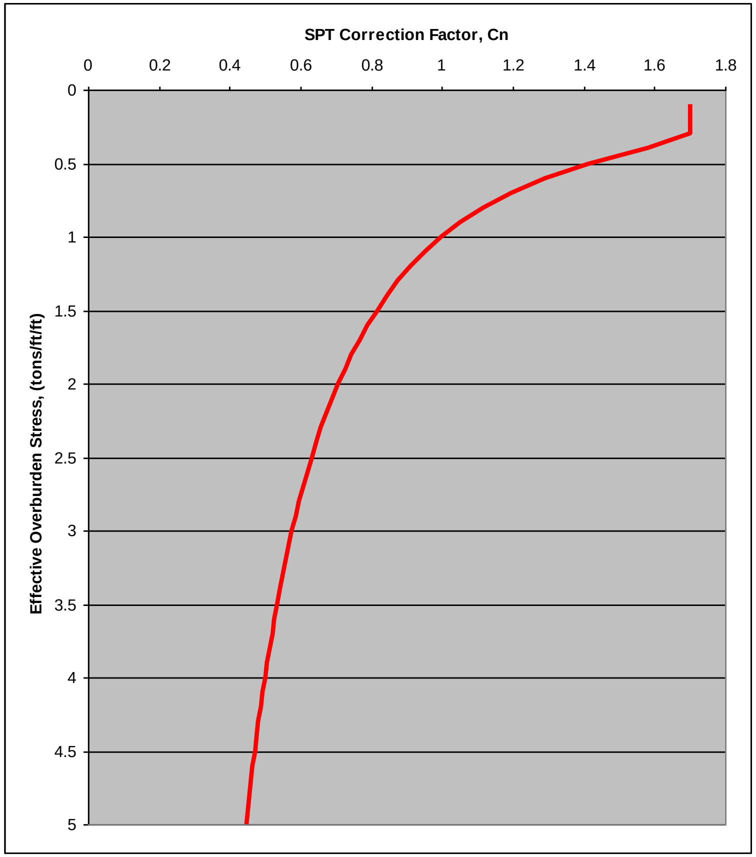
$$C_N = \left(\frac{P_a}{\sigma'_{V0}} \right)^{0.5} \leq 1.7$$

P_a = atmospheric pressure in same units as σ'_{V0}

$P_a = 100 kPa$

or

$P_a = 2000\, psf$



SPT Corrections

- An energy ratio, ER, of 60% has generally been accepted as the reference value.
- Approximate correction factor C_E = ER/60% to modify the SPT results to a 60% energy ratio for various type of hammers and anvils are listed below.

Factor	Equipment Variable	Value
Energy Ratio C_E	Donut Hammer Safety Hammer Automatic-Trip Donut Type Hammer	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3

Other SPT Corrections

$$\left(N_1\right)_{60} = C_N \times C_E \times C_B \times C_S \times C_R \times N$$

Factor	Equipment Variable	Value
Borehole Diameter C_B	65-115 mm 150 mm 200 mm	1.00 1.05 1.15
Sampling Method C_S	Standard Sampler Sampler without liner	1.0 1.2
Rod Length C_R	3-4 m 4-6 m 6-10 m >10	0.75 0.85 0.95 1.00

SPT Correction for Fines Content

- Because the presence of fines in the soil influences the liquefaction resistance and the SPT blow count differently, the SPT blow count is further corrected to an equivalent clean sand value.
- Idriss and Seed recommendation for correcting SPT determined for silty sands to an equivalent clean SPT:

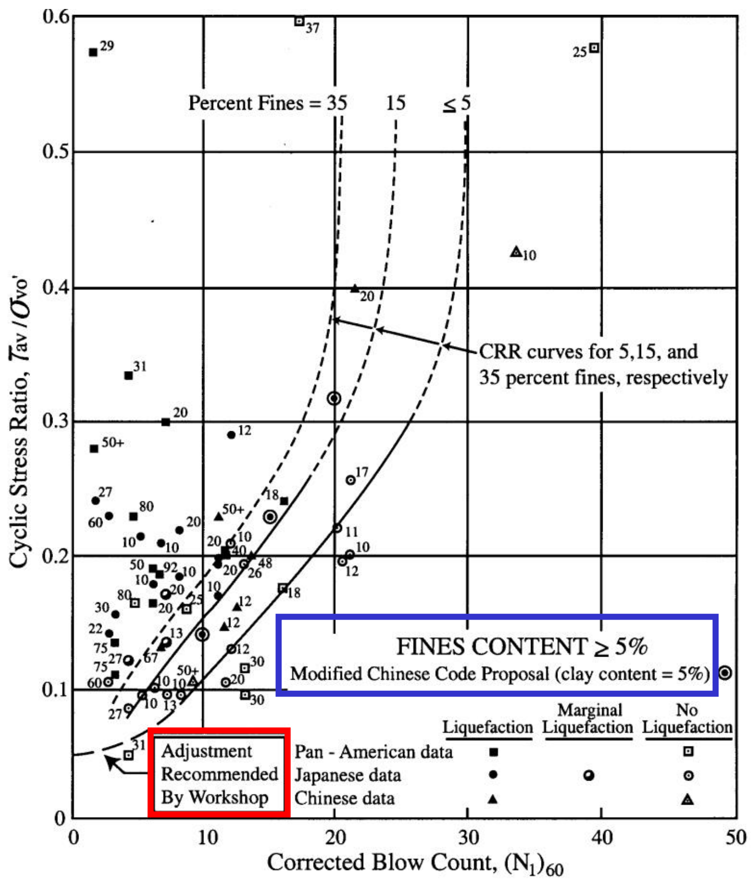
$$\left(N_1\right)_{60CS} = \alpha + \beta\left(N_1\right)_{60}$$

$$
\begin{aligned}
a &= 0.0 && \text{for FC} \le 5\% \\
&= \exp\left[1.76 - \left(\frac{190}{\mathrm{FC}^2}\right)\right] && \text{for } 5\% \le \text{FC} \le 35\% \\
&= 5.0 && \text{for FC} \ge 35\% \\
b &= 1.0 && \text{for FC} \le 5\% \\
&= \left[0.99 + \left(\frac{\mathrm{FC}^{1.5}}{1000}\right)\right] && \text{for } 5\% \le \text{FC} \le 35\% \\
&= 1.2 && \text{for FC} \ge 35\%
\end{aligned}
$$

FC is the fines content measured from laboratory gradation tests on retrieved soil samples.

Standard Penetration Tests (SPT)

Figure 2 Simplified Base Curve Recommended for Calculation of CRR from SPT Data along with Empirical Liquefaction Data (modified From Seed et al., 1985)



The SPT Clean sand base curve for CRR for Mw = 7.5 and must be adjusted for other magnitudes

Standard Penetration Tests (SPT)

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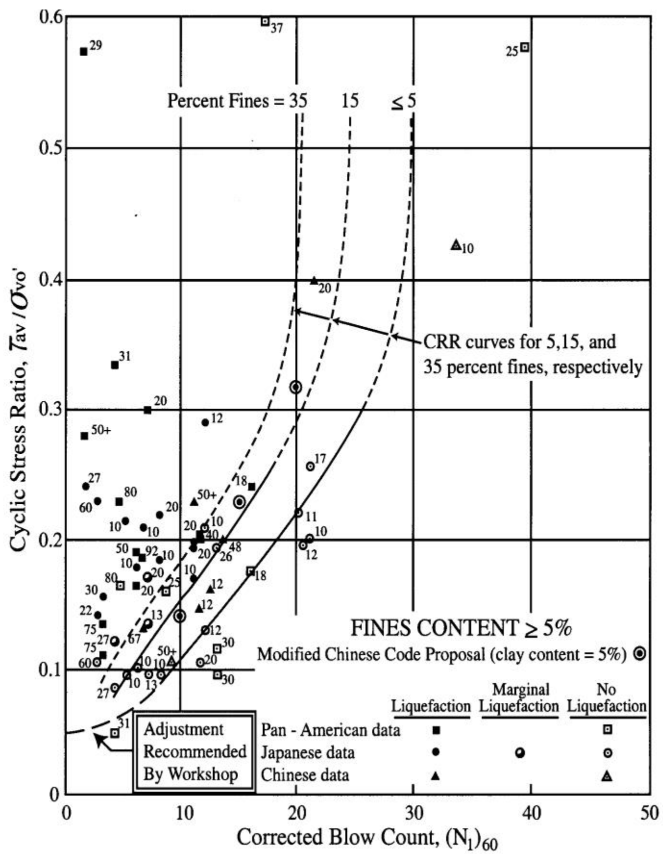


$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4}$$

$$x = (N_1)_{60}$$

$$a = 0.048$$

$$b = -0.1248$$

$$c = -.004721$$

$$d = 0.009578$$

$$e = 0.0006136$$

$$f = -0.0003285$$

$$g = -1.673e - 5$$

$$h = 3.714E - 06$$

Magnitude Scaling Factor

- MSF is used to adjust the simplified curve to magnitudes smaller or larger than 7.5. MSFs are a surrogate for duration of strong-ground motion.

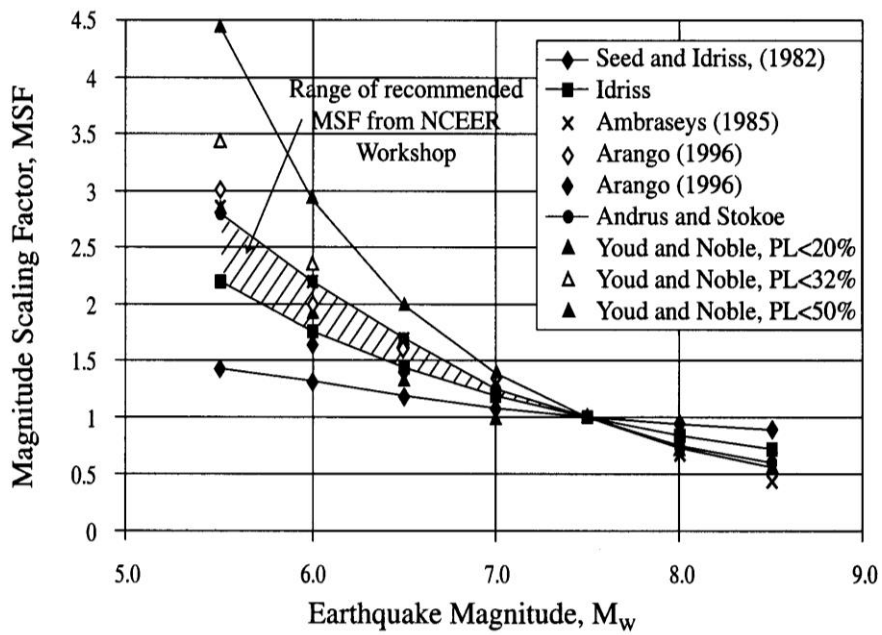


Figure 12 Magnitude Scaling Factors Derived by Various Investigators (After Youd and Noble, Magnitude Scaling Factors, this report)

Magnitude Scaling Factor by various investigators

- Magnitude scaling factors (MSF) are a surrogate for duration of strong motion.

Magnitude, M (1)	Seed and Idriss (1982) (2)	Idriss (3)	Ambraseys (1988) (4)	Arango (1996) (5)	Arango (1996) (6)	Andrus and Stokoe (in press) (7)	Youd and Noble (this report) $P_L<20\%$ (8)	Youd and Noble (this report) $P_L<32\%$ (9)	Youd and Noble (this report) $P_L<50\%$ (10)
5.5	1.43	2.20	2.86	3.00	2.20	2.8	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.1	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.6	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	1.00			1.00
8.0	0.94	0.84	0.67	0.75	0.85	0.8?			0.73?
8.5	0.89	0.72	0.44			0.65?			0.56?

Magnitude Scaling Factor

- To consider the influence of MSF, the equation for factor of safety, FS, against liquefaction which is the ratio of the soil's capacity to resist liquefaction (represented by CRR adjusted for magnitude) divided by the demand placed on the soil (represented by the CSR):

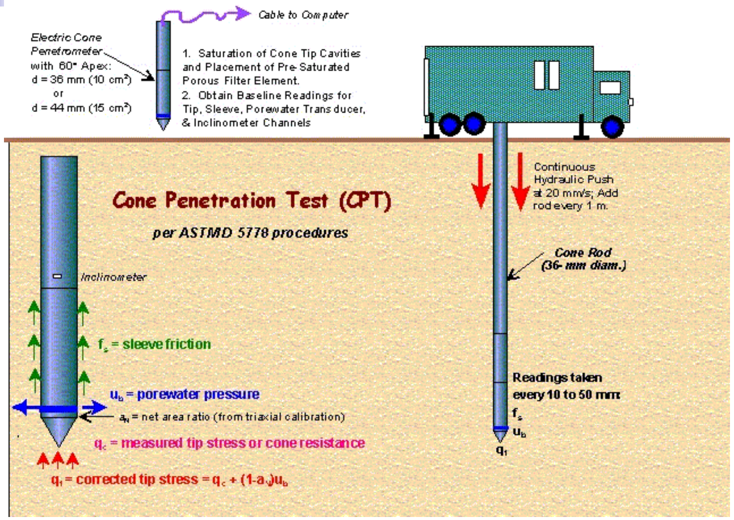
$$FS = \left(\frac{CRR_{7.5}}{CSR} \right) MSF$$

Correction of High Overburden Pressures

- If necessary, the FS can also account for high overburden stresses and initial static shear stresses.

$$FS = \left(\frac{CRR_{7.5}}{CSR} \right) MSF \; K_{\sigma} K_{\alpha}$$

Cone Penetration Test (CPT)

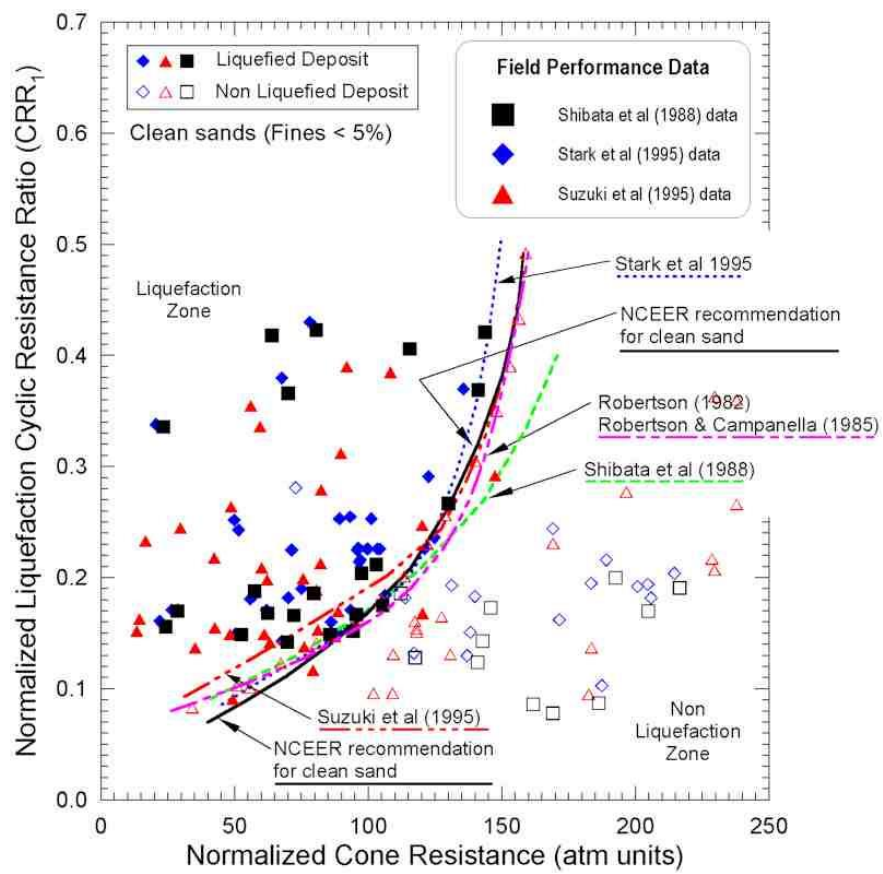


- The CPT provides a continuous vertical profile of tip and sleeve resistance
- The CPT test is less operator dependent than SPT and more robust interpretation of test results
- No samples are obtained

Cone penertratoin Test (CPT)

- The CPT clean sand base curve is used to determine $CRR_{7.5}$

- The database of CPT-based case studies is growing rapidly, which contributes to the reliability of CPT.



Shear Wave Velocity (Vs)

- Vs is a fundamental dynamic soil parameter
- Vs Measurements may be possible in difficutl-to-penetrate soils such as gravelly soils
- Vs is a small-strain parameter, but liquefaction occurs at intermediate-to-large strains.

Shear Wave Velocity (Vs)

- Vs measurements do not, per se, provide samples for classification
- Some types of Vs measurements may miss loose, low-velocity layers prone to liquefaction
- Use of Vs is consistent with site reponse calculations
- The database of Vs-based case studies is growing.

Liquefaction Potential Index (LPI)

- The Liquefaction Potential Index (Iwasaki, 1982) may be used to "integrate" the factor of safety in the upper 20m to correlate the initiation of liquefaction with ground failure.

$$LPI = \sum_i w_i S_i H_i$$

$$where$$

$$w(z) = 10 - 0.5z \qquad z \quad in\ meters$$

$$S = 1 - FS \qquad for\ 0 \leq FS \leq 1$$

$$= 0 \qquad for\ FS > 1$$

$$H = layer \quad thickness$$

Liquefaction Potential Index (LPI)

LPI	Liquefaction Severity
0	Little to none
0 to 5	Minor
5 to 10	Moderate
Greater than 15	Major

Permanent Ground Deformations

- If liquefaction occurs, it is of interest to estimate the permanent ground deformations resulting from lateral spreading.
- Because of the complexity of these processes, most current methods of estimating permanent ground deformations are strictly emprical.
- Hamada et al. (1986)
  - $D_H = 0.75\ T^{0.5}\ \theta^{0.33}$
  - D_H is the horizontal displacement in meters.,
  - T is thickness of the liquefied layer in meters, and
  - θ is the larger of the slope of the ground surface or base of the liquefied layer in percent

Permanent Ground Deformations

- Youd and Perkins (1987)
  - $\text{Log}(D_H) = -15.787 + 1.179M_w - 0.927\log R$
    $0.013R + 0.429\log S + 0.348\log T_{15}$ -
    $4.4527\log(100 - F_{15}) - 0.922D50_{15}$

R = Joyner-Boore distance in km

S = ground slope in percent

T_{15} = cumulative thickness in meters of saturated coarse-graind soils with N1(60) values less than 15

F_{15}= is the average fines content in percent of soils within T_{15} layers

$D50_{15}$= mean grain size in (mm) of soils within the T_{15} layers

Permanent Ground Deformations

- Bartlett and Youd (1992, 1995)
  - $\log(LSI) = -3.49 - 1.86 \log R + 0.98 M_w$

- LSI is the liquefaction Severity Index
- R is the Joyner and Boore distance in km
- Mw is …