During strong earthquake shaking, a loose saturated sand deposit will have a tendency to compact and, thus, have a decrease in volume. If this deposit cannot drain rapidly, there will be an increase in the pore water pressure. The effective stress in the sand deposit is equal to the difference between the overburden pressure and the pore water pressure. With increasing oscillation, the pore water pressure will increase to the point where the pore water pressure will be equal to the overburden pressure. Since the shear strength of a cohesionless soil is directly proportional to the effective stress, the sand will not have any shear strength and is now in a liquefied state. "Sand boils" appearing at the ground surface during an earthquake is evidence that liquefaction has occurred.

Figure 3-12. Liquefaction-induced tilting of three-story residential structure in Central Taiwan. Photograph by Dr. Farzad Naeim.

Liquefaction can have a significant and sometimes devastating effect on buildings supported on the upper soils without consideration of the consequences of liquefaction. Figures 3-11 and 3-12 present examples of the effects of liquefaction on buildings in the 1999 Kocaeli, Turkey and Chi-Chi, Taiwan earthquakes.

3.4.2 Evaluating the Liquefaction Potential by Standard Penetration Tests

There are a number of different methods by which the potential for liquefaction of a soil can be evaluated. These methods generally compare the cyclic shear resistance of the soil with the cyclic shear stresses and strains caused by an earthquake. Simplified empirical methods have been developed that utilize case histories of past occurrences (or non-occurrences) of liquefaction during significant seismic events. Other methods use analytical techniques that incorporate dynamic analysis and laboratory testing. The most common and traditional method of analysis uses correlations between the liquefaction characteristics of soils and the Standard Penetration Test or N-value as originally described by Seed et al.[3-11] Since the analysis was first introduced, the methodology has been refined and various corrections are applied to account for variability in sampling and performance; a summary of recent concensus opinion on liquefaction evaluation was conducted by NCEER and has been edited by Youd and Idriss[3-12]; those concensus opinions are presented herein. Thus, for analysis, a corrected N-value is used. The value of the corrected N-value, denoted as $(N_1)_{60}$ is found by the formula:

$$(N_1)_{60} = N_m \cdot C_N C_E C_B C_R C_S$$

where N_m is the measured standard penetration resistance, C_N is a correction factor for overburden pressure, C_E is the correction factor for hammer energy ratio, C_B is a correction factor of borehole diameter, C_R is the correction factor for rod length, and C_S is the correction for samplers with or without liners.

The overburden pressure correction factor, C_N, may be calculated from the following formula:

$$C_N = (P_a/\sigma'_{vo})^{0.5}$$

where P_a is 100 kPa or approximately atmospheric pressure (2,089 pounds per square foot) and σ'_{vo} is the effective vertical overburden pressure at the depth of the standard penetration sample. Table 3-12 shows the suggested correction factors for the other corrections.

Table 3-12. Corrections to SPT (Ref. 3-12)

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma'_{vo})^{0.5}$
Energy Ratio	Safety Hammer Donut Hammer	C_E	0.60 to 1.17 0.45 to 1.00
Borehole Diameter	65 to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Rod Length	3 to 4 m 4 to 6 m 6 to 10 m 10 to 30 m >30 m	C_R	0.75 0.85 0.95 1.0 <1.0
Sampling Method	Standard Sampler Sampler without liners	C_S	1.0 1.2

With respect to the energy ratio, ER, it is believed that the approximate historical average SPT energy for North American practice is 60% of the maximum theoretical energy achievable. The ER delivered by any particular SPT setup depends on the type of hammer and anvil in the drilling system and on the method of hammer release. The correction factor, C_E, normalizes the N-value to a 60% ER.

During an earthquake, the soils will be subject to cyclic shear stresses induced by the ground shaking. The average cyclic stress ratio (CSR) during an earthquake may be estimated by the following formula:

$$CSR = \tau_{av} / \sigma'_o = 0.65\ (a_{max} / g) \cdot (\sigma_o / \sigma'_o) \cdot r_d$$

where a_{max} = maximum acceleration at the ground surface

σ_o = total overburden pressure at depth under consideration

σ_o' = effective overburden pressure at depth under consideration

r_d = stress reduction coefficient

The range of values for the stress reduction, r_d, are shown in Figure 3-13.

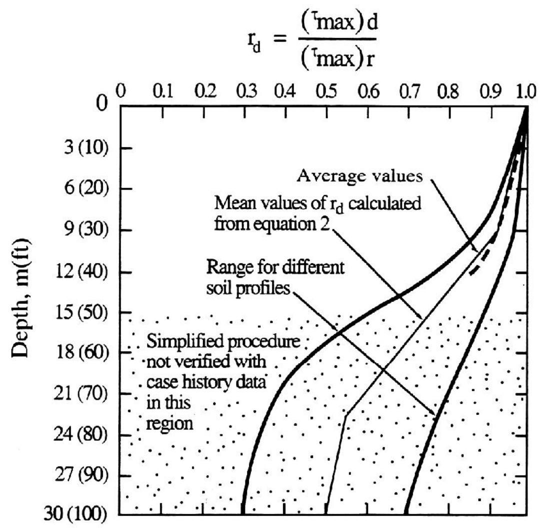

Figure 3-13. Stress Reduction Factor, r_d (Ref. 3-12)

The average value of the stress reduction coefficient, r_d, may be estimated by the following equations:

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

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

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

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

Having estimated the average shear stress ratio, charts similar to Figure 3-14 may be used to determine the potential for liquefaction. Figure 3-14 shows the relationship between the cyclic resistance ratio (CRR) and the corrected standard penetration resistance, N_1, for a magnitude 7.5 earthquake. The CRR is also referred to as the liquefaction resistance or liquefaction resistance ratio. If the CSR (τ_{av} / σ'_o) induced by the earthquake is less than the liquefaction resistance ratio, CRR, as shown on Figure 3-14, liquefaction would not be expected to occur; similarly if the CSR exceeds the CRR, liquefaction would be expected to occur. A factor of safety against liquefaction could be determined by the ratio of the CSR divided by the CRR. For $(N_1)_{60}$ values greater than about 30, no liquefaction would be expected and the factor of safety would be great.

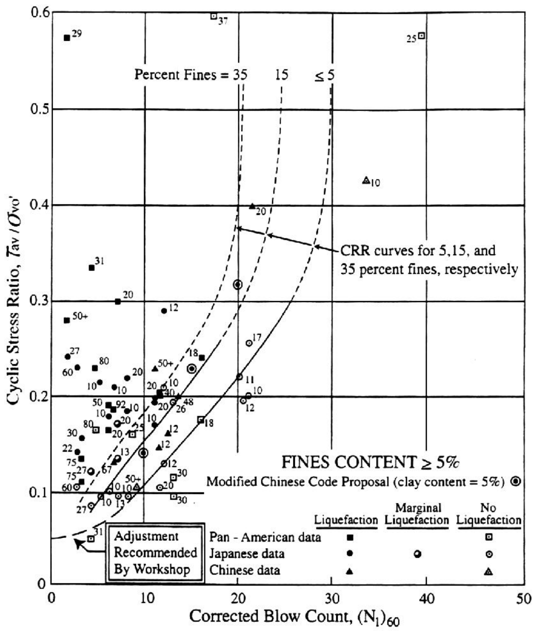

Figure 3-14. Figure 3-14. Curve Recommended for Determining CRR from SPT Data (Ref. 3-12)

The CRR base curve for clean sands (i.e., <5% fines content) may be approximated by the relationship:

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

where:
a = 0.048
b = -0.1248
c = 0.004721
d = 0.009578
e = 0.0006136
f = -0.0003285
g = -1.673×10^{-5}
h = 3.714×10^{-6}
x = $(N_1)_{60}$

This equation is valid for values of $(N_1)_{60}$ less than 30.

Figure 3-14 also shows that the influence of the fines content on the potential for liquefaction in a way that the greater the fines content, the lesser the potential for liquefaction given the same N_1 value. I.M. Idriss and R.B. Seed have developed equations to correct the standard penetration resistance for silty sands, $(N_1)_{60}$, to an equivalent clean sand penetration resistance $(N_1)_{60cs}$. These equations are:

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

where the α and β coefficients are determined by:

$$\alpha = \exp [\, 1.76 - (190/FC^2) \,]$$

$$\beta = [\, 0.99 + (FC^{1.5}/1000) \,]$$

where FC is the fines content measured from laboratory gradation tests on soil samples. These equations essentially represent the CRR curves for different fines contents as shown in Figure 3-12.

As mentioned earlier, Figure 3-14 applies only for a magnitude 7.5 earthquake; to evaluate the potential for liquefaction for other magnitude events; Seed et al. (1983)[(3-13)] originally determined correlation factors that allow the induced stress ratios for other magnitude events to be adjusted to correspond to a magnitude of 7.5 by dividing the stress ratios by the factors given in Table 3-13:

Table 3-13. Seed and Idriss Original Magnitude Scaling Factors (Ref. 3-13)

Earthquake Magnitude	Magnitude Scaling Factor
5.25	1.5
6	1.32
6.75	1.13
7.5	1.0
8.5	0.89

The Seed and Idriss magnitude scaling factors are based on estimates of equivalent cycles of shear stress developed during different magnitude earthquakes. However, it is generally believed now that the original Seed and Idriss magnitude scaling factors are very conservative for moderate-sized earthquakes. Idriss has proposed a new set of magnitude scaling factors after re-evaluating the data. Idriss has proposed that the magnitude scaling factor, MSF, be defined as a function of the moment magnitude, M, as given in the equation:

$$MSF = 10^{2.24} / M^{2.56}$$

Table 3-14. Magnitude Scaling Factors Defined by Various Investigators (Ref. 3-12)

Magnitude M	Seed and Idriss (original)	Idriss	Ambreseys	Arango		Andrus & Stokoe	Youd and Noble		
							P_L<20%	P_L<32%	P_L<50%
5.5	1.43	2.20	2.86	3.00	2.20	2.80	2.86	3.42	4.44
6.0	1.32	1.76	2.20	2.00	1.65	2.10	1.93	2.35	2.92
6.5	1.19	1.44	1.69	1.60	1.40	1.60	1.34	1.66	1.99
7.0	1.08	1.19	1.30	1.25	1.10	1.25	1.00	1.2	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?

Other researchers have also determined magnitude scaling factors; these values are shown in Table 3-14. The table also repeats the original Seed and Idriss MSF factors and also presents the new Idriss MSF factors.

There is not a concensus in the geotechnical community of which of the various sets of magnitude scaling factors to use except is it is generally accepted that the original Seed and Idriss MSF factors are conservative for magnitudes of less than 7.5. It should be noted that Arango has two sets of MSF factors. The first set was based on farthest observed liquefaction effects from the seismic energy source, estimate average peak accelerations at those distant sites, and the absorbed seismic energy requred to cause liquefaction; the second set was developed from energy concepts and the relationship developed by Seed and Idriss between numbers of significant stress cycles and earthquake magnitude. The second Arango MSF factors are similar to the new Idriss MSF factors. The Youd and Noble MSF factors are found in three sets that are a function of P_L, the probability that liquefaction did not occur.

For earthquake magnitudes greater than 7.5, it recommended that the newer Idriss MSF factors be used because it is believed that the original Seed and Idriss MSF factors were not sufficiently conservative in the upper magnitude range.

Thus, the factor of safety (FS) against liquefaction may be written in terms of the CRR, CSR and MSF factors as follows:

$FS = (CRR_{7.5}/CSR)\ MSF$

where $CRR_{7.5}$ is the cyclic resistance ratio for a magnitude 7.5 earthquake from Figure 3-14.

Example

A sand deposit has been identified beneath a site located adjacent to a river. The sand deposit is 10 feet thick and the top of the layer is 10 feet below the ground surface and overlain by a very stiff clay and is underlain by bedrock. The water level has been measured to be at a depth of 10 feet. The standard penetration resistance of the layer has been determined to be 12 blows per foot and a standard sampler was used; a drill rig with a safety hammer with an efficiency of 60% was used. The length of the drill rod is 10 meters and the borehole diameter is 5 inches (127 mm).

The design earthquake has been designated as a moment magnitude 6-3/4 event on a nearby fault and the maximum ground acceleration is expected to be 0.35 g.

The wet unit weight of the clay soils is 125 pounds per cubic foot and the wet unit weight of the sand soils is 130 pounds per cubic foot. The sands has 15% fines content according to a grain size analysis.

Compute the factor of safety against liquefaction of the sand layer.

Solution:

Step 1: Determine the effective overburden pressure at the center of the sand layer:

σ'_o = (125 pcf) (10 ft) + [(130 pcf - 62.4 pcf) (5 ft)]
= 1,588 psf

Step 2: Determine the total overburden pressure at the center of the sand layer:

σ_o = (125 pcf) (10 ft) + (130 pcf) (5 ft)
= 1,900 psf

Step 3: Determine the stress reduction factor, r_d:

z = 15 ft x (1 meter/3.2808 ft)
= 4.572 m

r_d = 1 - 0.00765 z
= 1 - 0.00765 (4.572)
= 0.965

Step 4: Determine the cyclic stress ratio, CSR.

CSR = $\tau_{av} / \sigma'_o = 0.65\ (a_{max} / g)(\sigma_o / \sigma'_o)\ r_d$
= 0.65 (0.35 g/g) (1,900 psf / 1588 psf) (0.965)
= 0.263

Step 5: Determine correction factors to SPT blowcount:

Referring to Table 3-12, the correction factors are

Overburden pressure:
C_N = $(P_a/\sigma'_{vo})^{0.5}$
= $(2{,}089\ \text{psf} / 1{,}588\ \text{psf})^{0.5}$
= 1.15

Energy ratio:
C_E = 1.0, since safety hammer is 60% efficient

Borehole diameter:
C_B = 1.0, since diameter is 5 in. (127 mm)

Rod length:
C_R= 1.0, since rod length is 10 m

Sampling method
C_S = 1.0, since standard sampler used

$(N_1)_{60}$ = $N_m \cdot C_N C_E C_B C_R C_S$
= (12) (1.15) (1.0) (1.0) (1.0) (1.0)
= 13.8

Step 6: Determine correction for fines content:

Since the fines content is greater than 5%, correction is needed.

α = exp [1.76 - $(190/FC^2)$]
= exp [1.76 - $(190/15^2)$]
= 2.50
β = [0.99 + $(FC^{1.5}/1000)$]
= [0.99 + $(15^{1.5}/1000)$]
= 1.05

$(N_1)_{60cs} = \alpha + \beta (N_1)_{60}$
= 2.50 + 1.05 (13.8)
= 17.0

Step 7: Determine the cyclic resistance ratio, $CRR_{7.5}$:

Referring to Figure 3-14, for $(N_1)_{60cs}$ = 17.0, the cyclic resistance ratio is

$CRR_{7.5}$ = 0.185

Step 8: Determine the magnitude scaling factor, MSF, for magnitude 6-3/4:

Use the Idriss magnitude scaling factor,

MSF $= 10^{2.24} / M^{2.56} = 10^{2.24} / (6.75)^{2.56}$
= 1.31

Step 9: Compute the factor of safety against liquefaction:

$$
\begin{aligned}
FS &= (CRR_{7.5}/CSR)\ MSF \\
&= (0.185/0.263)\ 1.31 \\
&= 0.92
\end{aligned}
$$

The factor of safety against liquefaction is less than unity (1.0), therefore, liquefaction would be expected to occur in the event of the design earthquake.

3.4.3 Evaluating the Liquefaction Potential by Cone Penetration Tests

Because of questions regarding the reliability and quality of the standard penetration resistances, and the inability to easily obtain a continuous profile of the resistances, there is more reliance now upon the cone penetration test (CPT). The CPT can provide a nearly continuous profile of penetration resistance and is generally more repeatable and consistent than other forms of penetration testing. One obvious deficiency of the CPT is the lack of a physical sample of the soil tested. A procedure similar to the simplified method for the SPT has been developed and is reported in the NCEER concensus document.[3-12] The chart in Figure 3-15 can be used to determine the cyclic resistance ratio ($CRR_{7.5}$) for clean sands having a fines content of less than or equal to 5% from CPT data. The chart is valid only for a magnitude 7.5 earthquake and shows the calculated cyclic stress ratio (CSR) versus the corrected normalized CPT resistance denoted as q_{c1N}. Like the chart for SPT data, the CPT chart was derived from data from sites where liquefaction effects were or were not observed following past earthquakes. The CRR curve separates the region indicative of liquefaction (above the line) from the region where there was non-liquefaction (below the line).

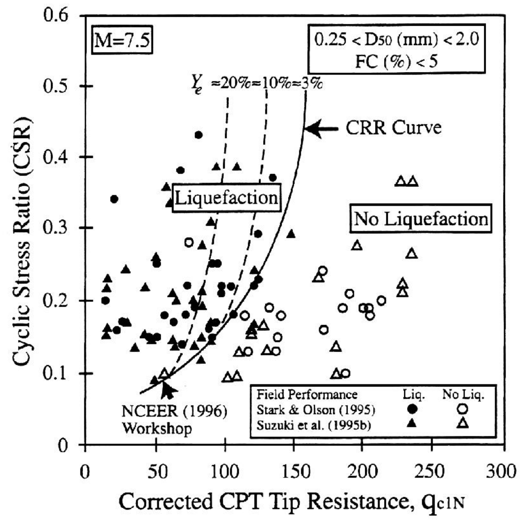

Figure 3-15. Curve Recommended for Determining CRR from CPT Data (Ref. 3-12)

The CRR curve in Figure 3-15 can be approximated by the following set of equations:

If $(q_{c1N})_{cs} < 50$

$$CRR_{7.5} = 0.833\ [(q_{c1N})_{cs} / 1000\ ] + 0.05$$

If $50 \le (q_{c1N})_{cs} < 160$

$$CRR_{7.5} = 93\ [(q_{c1N})_{cs} / 1000\ ]^3 + 0.08$$

where $(q_{c1N})_{cs}$ is the clean sand cone penetration resistance normalized to 100 kPa (approximately one atmosphere of pressure). The truly normalized (i.e., dimensionless) cone penetration resistance corrected for overburden stress (q_{c1N}) is given by:

$$q_{c1N} = C_Q\ (\ q_c / P_a\ ) = q_{c1} / P_a$$

where:

$$C_Q = (\ P_a / \sigma'_o\ )^n$$

C_Q is the normalizing factor for cone penetration resistance; P_a is approximately one atmosphere of pressure given in the same units as the measured field CPT tip resistance, q_c, and calculated overburden pressure, σ'_o. C_Q is limited to a maximum value of 2 at shallow depths. The value of the exponent, n, is dependent on the grain characteristics of the soil. The value of n ranges from 0.5 for clean sands to 1.0 for clays. Discussion on the determination of the exponent n follows.