Summary Report

Workshop Participants

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Abstract

Over the past twenty-five years, a procedure, termed the "simplified procedure," has evolved for evaluating liquefaction resistance of soils. This procedure has become the standard of practice in North America and throughout much of the world. Following disastrous earthquakes in Alaska and in Niigata, Japan in 1964, Professors H.B. Seed and I.M. Idriss developed and published the basic "simplified procedure." The procedure, which is largely empirical, evolved over the decades primarily through summary papers by H.B. Seed and his colleagues. In 1985, Professor Robert V. Whitman convened a workshop on behalf of the National Research Council (NRC) in which thirty-six experts reviewed the state-of-knowledge and the state-of-the-art for assessing liquefaction hazard. No general review or update of the simplified procedures has occurred since that time. The purpose of the 1996 workshop, sponsored by the National Center for Earthquake Engineering Research (NCEER), was to convene a group of experts to review developments and gain consensus for further augmentations to the procedure. To keep the workshop focused and the content tractable, the scope was limited to evaluation of liquefaction resistance. Post-liquefaction phenomena, such as soil deformation and ground failure, although equally or more important, were beyond the scope of this workshop. The participants developed consensus recommendations on the following topics: (1) use of the standard and cone penetration tests for evaluation of liquefaction resistance, (2) use of shear wave velocity measurements for evaluation of liquefaction resistance, (3) use of the Becker penetration test for gravelly soils, (4) magnitude scaling factors, (5) correction factors K_α and K_σ, and (6) evaluation of seismic factors required for the evaluation procedure. Probabilistic analysis and seismic energy considerations were also reviewed. Seismic energy concepts were judged to be insufficiently developed to make recommendations for engineering practice. Probabilistic methods have been used in some risk analyses, but are still outside the mainstream of standard practice.

Introduction

Over the past twenty-five years, a procedure, termed the "simplified procedure," has evolved for evaluating the seismic liquefaction resistance of soils. This procedure has become the standard of practice in North America and throughout much of the world. Following disastrous earthquakes in Alaska and Niigata, Japan in 1964, Seed and Idriss (1971) developed and published the basic "simplified procedure." The procedure has been corrected and augmented periodically since that time with landmark papers by Seed (1979), Seed and Idriss (1982), and Seed et al. (1985). In 1985, Professor Robert V. Whitman from the Massachusetts Institute of Technology convened a workshop on behalf of the National Research Council (NRC) in which thirty-six experts and observers thoroughly reviewed the state-of-knowledge and the state-of-the-art for assessing liquefaction hazard. That workshop produced a report (NRC, 1985) that has become a widely used standard and reference for liquefaction hazard assessment. No general review or update of the simplified procedures has occurred since that time.

The purpose of the 1996 workshop, sponsored by the National Center for Earthquake Engineering Research (NCEER), was to convene a group of experts to review recent developments and gain consensus on further corrections and augmentations to the procedure. Emphasis was placed on new developments since the NRC review. To keep the workshop focused and the content tractable, the scope was limited to procedures for evaluating liquefaction resistance of soils under level to gently sloping ground. In this context, liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent severe softening of granular soils. Post-liquefaction phenomena, such as soil deformation and ground failure, although equally or more important than triggering, were beyond the scope of the workshop.

The simplified procedure was developed from evaluations of field observations and field and laboratory test data. Field evidence of liquefaction generally consisted of observed sand boils, ground fissures or lateral spreads. Data were collected mostly from sites on level to gently sloping terrain underlain by Holocene alluvial or fluvial sediment at shallow depths (less than 15 m). The original procedure was verified for and is applicable only to these site conditions. The primary focus of the workshop was to review and update procedures for evaluating soil liquefaction resistance for these general site conditions. Limited attention was given to liquefaction resistance evaluation for sediment layers at greater depths (high overburden pressures) and beneath steeply sloping terrain or embankments.

Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR)

Calculation or estimation of two variables is required for evaluation of liquefaction resistance of soils. These variables are the seismic demand placed on a soil layer, expressed in terms of cyclic stress ratio (CSR), and the capacity of the soil to resist liquefaction, expressed in terms of cyclic resistance ratio (CRR), hereafter referred to as liquefaction resistance or liquefaction resistance ratio. CRR is a symbol proposed by Robertson and Wride that was endorsed by the workshop. Previously, this factor had been called the cyclic stress ratio required to generate liquefaction, or the cyclic strength ratio, and had been given different symbols by different writers. For example, Seed and Harder (1990) used the symbol CSR*l*, Youd (1993) used the symbol CSRL, and Kramer (1996) used the symbol CSR_L to denote this ratio. The workshop participants agreed that CRR conveys an appropriate meaning and generates less confusion than the use of CSR with or without a subscript to signify liquefaction resistance.

Seed and Idriss (1971) formulated the following equation for calculation of CSR:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65\ (a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d \qquad (1)$$

where a_{max} is the peak horizontal acceleration at ground surface generated by the earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective vertical overburden stresses, respectively, and r_d is a stress reduction coefficient. The latter coefficient provides an approximated correction for flexibility of the soil profile. The workshop participants recommend the following minor modification to the procedure for calculation of CSR. For noncritical projects, the following equations may be used to estimate average values of r_d.

$r_d = 1.0 - 0.00765\ z$	for $z \leq 9.15$ m	(2a)
$r_d = 1.174 - 0.0267\ z$	for $9.15\ m < z \leq 23$ m	(2b)
$r_d = 0.744 - 0.008\ z$	for $23 < z \leq 30$ m	(2c)
$r_d = 0.50$	For $z > 30$ m	(2d)

where z is depth below ground surface in meters. Parts a and b of this equation were proposed by Liao and Whitman (1986b), part c was added by Robertson and Wride (this report), and part d was suggested by William F. Marcuson (US Army Engineers, oral commun.) in post-workshop discussions. Mean values of r_d calculated from Equation 2 are plotted on Figure 1 along with the mean and range of values proposed by Seed and Idriss (1971). The workshop participants agreed that for convenience in programming spreadsheets and other electronic aids, and to be consistent with past practice, r_d values determined from Equation 2 are suitable for use in routine engineering practice. The user should understand, however, and take into account that r_d values calculated from Equations 2 or 3 give only the mean value from a range of possible r_d values and that the range of r_d values increases with depth. Thus the certainty with which CSR can be calculated decreases with depth when mean r_d values are used to simplify calculations. In addition to the uncertainty in r_d, the simplified procedure is not well verified for depths greater than about 15 m, as indicated on Figure 1. Thus the user should understand that results developed from the simplified procedure are quite uncertain at depths greater than 15 m.

As an alternative to Equation 2, Thomas F. Blake (Fugro-West, Inc., Ventura, Calif., written commun.) approximated the mean curve plotted on Figure 1 by the following equation:

$$r_d = \frac{(1.000 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1.000 + 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001210z^2)} \qquad (3)$$

where z is depth beneath ground surface in meters. Equation 3 yields essentially the same values for r_d as Equations 2a-d, but is easier to program for many applications and may be used in routine engineering practice.

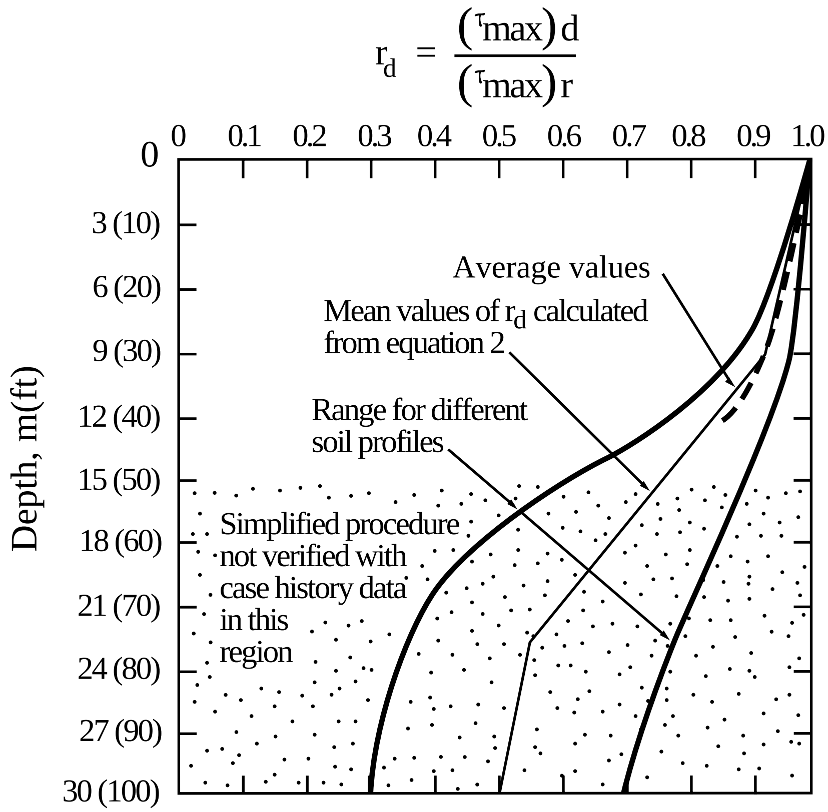


FIGURE 1 r_d Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines from Equation 2

The primary focus of the workshop was to improve procedures for evaluating liquefaction resistance of soils, CRR. A plausible method for evaluating CRR is retrieving undisturbed soil specimens from field sites and testing those specimens in the laboratory using cyclic tests to model seismic loading conditions. Unfortunately, specimens of granular soils retrieved with typical drilling and sampling techniques are generally too disturbed to yield meaningful laboratory tests results. Only through use of specialized sampling techniques, such as ground freezing, can sufficiently undisturbed specimens be obtained. The cost of such procedures is generally prohibitive for all but the most critical projects. To avoid the difficulties associated with undisturbed sampling and testing, field tests have become the state-of-the-practice for routine liquefaction investigations.

Several field tests have gained common usage for evaluation of liquefaction resistance, including the cone penetration test (CPT), the standard penetration test (SPT), shear-wave velocity measurements (V_s), and the Becker penetration test (BPT). These tests were discussed at the workshop along with associated criteria for evaluating liquefaction resistance. Possible improvements to the state-of-the-art were reviewed and consensus recommendations developed for engineering practice. A conscientious attempt was made to correlate liquefaction resistance criteria from the various field tests to provide generally consistent results no matter which test is employed. Thus the choice of test should depend on availability of equipment, site conditions, cost, and preference. Primary advantages and disadvantages of each test are listed in Table 1.

Table 1. Comparison of Advantages and Disadvantages of Various Field Tests for Assessment of Liquefaction Resistance

Feature	Test Type			
	SPT	CPT	V_s	BPT
Number of test measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behavior influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good	Very good	Fair	Fair
Soil types in which test is recommended	Non-gravel	Non-gravel	All	Primarily gravel
Test provides sample of soil	Yes	No	No	No
Test measures index or engineering property	Index	Index	Engineering property	Index

Standard Penetration Test (SPT)

Criteria for evaluation of liquefaction resistance based on standard penetration test (SPT) blow counts have been rather robust over the years. Those criteria are largely embodied in the CSR versus $(N_1)_{60}$ plot reproduced in Figure 2. That plot shows calculated CSR and $(N_1)_{60}$ data from sites where liquefaction effects were or were not observed following past earthquakes along with CRR curves separating data indicative of liquefaction from data indicative of nonliquefaction for various fines contents. The CRR curve for a fines content less than five percent is the basic penetration criterion for the simplified procedure and is referred to hereafter as the "simplified base curve." The CRR curves in Figure 2 are valid only for magnitude 7.5 earthquakes.

Clean Sand Base Curve

Several changes to the SPT criteria were endorsed by workshop participants. The first change is to curve the trajectory of the simplified base curve at low $(N_1)_{60}$ to a projected CRR intercept of about 0.05 (Figure 2). This adjustment reshapes the base curve to achieve consistency with CRR curves

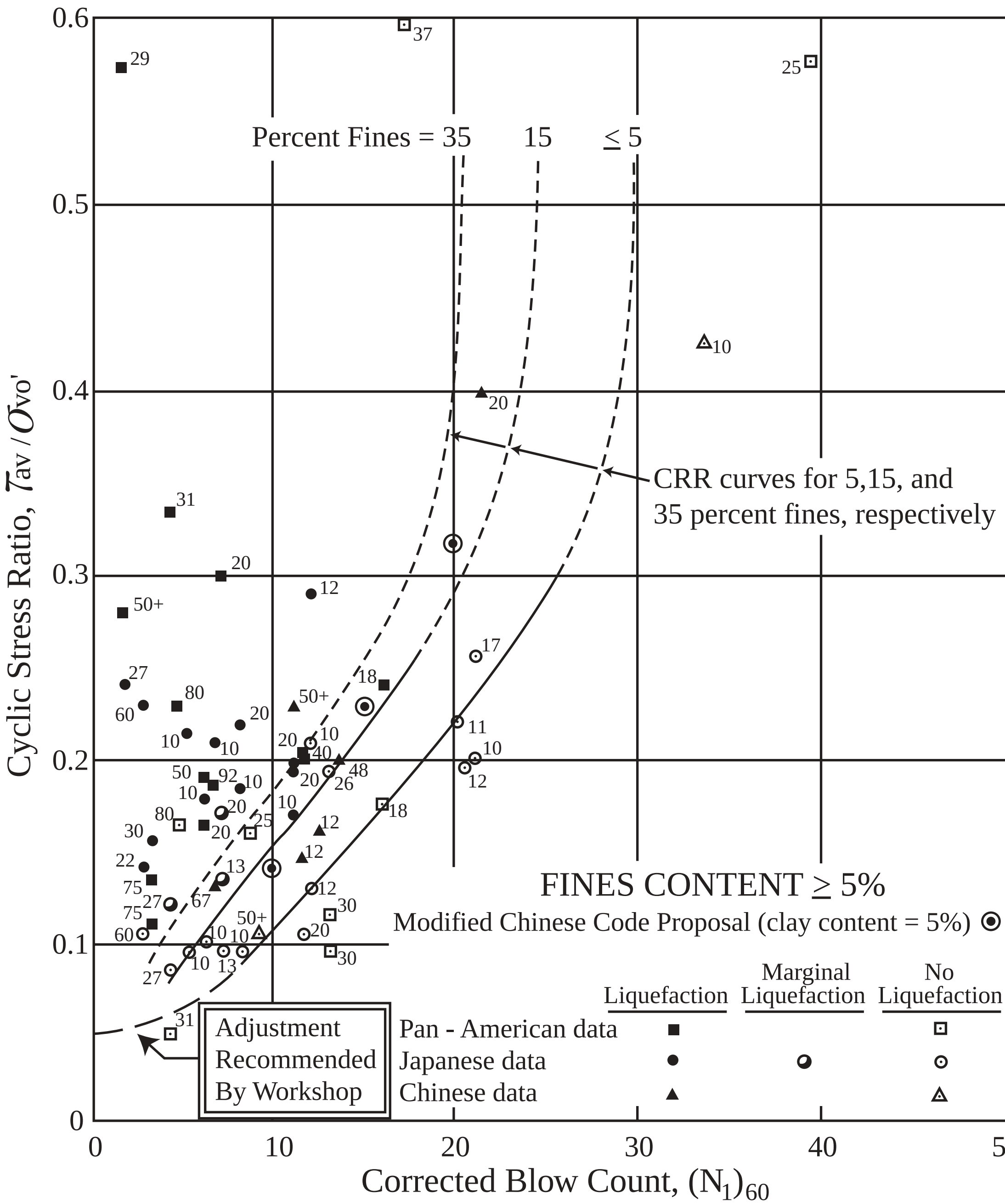


Figure 2 Simplified Base Curve Recommended for Calculation of CRR from SPT Data along with Empirical Liquefaction Data (modified From Seed et al., 1985)

developed from CPT data and probabilistic analyses by Liao et al. (1988) and Youd and Noble (Statistical and Probabilistic Analyses, this report). Seed and Idriss (1982) originally projected that curve through the origin, but there were few data to constrain the curve in the lower part of the plot. A better fit to the present empirical data is to bow the lower end of the base curve as indicated in Figure 2.

Thomas F. Blake (Fugro-West, Inc., Ventura, Calif., written commun.) approximated the simplified base curve plotted on Figure 2 by the following equation:

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

where $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes; $x = (N_1)_{60}$; $a = 0.048$; $b = -0.1248$; $c = -0.004721$; $d = 0.009578$; $e = 0.0006136$; $f = -0.0003285$; $g = -1.673E-05$; and $h = 3.714E-06$. This equation is valid for $(N_1)_{60}$ less than 30 and may be used in spreadsheets and other analytical techniques to approximate the simplified base curve for engineering calculations. Robertson and Wride (this report) indicate that Equation 4 is not applicable for $(N_1)_{60}$ less than three, but the general consensus of workshop participants is that the curve defined by Equation 4 should be extended to intersect the intercept at a CRR value of about 0.05.

Correlations for Fines Content and Soil Plasticity

Another change was the quantification of the fines content correction to better fit the empirical data and to support computations with spreadsheets and other electronic computational aids. In the original development, Seed et al. (1985) found that for a given $(N_1)_{60}$, CRR increases with increased fines content. It is not clear, however, whether the CRR increase is because of greater liquefaction resistance or smaller penetration resistance as a consequence of the general increase of compressibility and decrease of permeability with increased fines content. Based on the empirical data available, Seed et al. developed CRR curves for various fines contents as shown on Figure 2.

After a lengthy review by the workshop participants, consensus was gained that the correction for fines content should be a function of penetration resistance as well as fines content. The participants also agreed that other grain characteristics, such as soil plasticity may affect liquefaction resistance; hence any correlation based solely on penetration resistance and fines content should be used with engineering judgement and caution. The following equations, developed by I.M. Idriss with assistance from R.B. Seed are recommended for correcting standard penetration resistance determined for silty sands to an equivalent clean sand penetration resistance:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \qquad (5)$$

where α and β are coefficients determined from the following equations:

$$\alpha = 0 \qquad \text{for } FC \le 5\% \qquad (6a)$$
$$\alpha = \exp[1.76 - (190/FC^2)] \qquad \text{for } 5\% < FC < 35\% \qquad (6b)$$
$$\alpha = 5.0 \qquad \text{for } FC \ge 35\% \qquad (6c)$$

$$\beta = 1.0 \qquad \text{for } FC \le 5\% \qquad (7a)$$
$$\beta = [0.99 + (FC^{1.5}/1000)] \qquad \text{for } 5\% < FC < 35\% \qquad (7b)$$
$$\beta = 1.2 \qquad \text{for } FC \ge 35\% \qquad (7c)$$

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

These equations may be used for routine liquefaction resistance calculations. Back calculation of CRR curves as a function of fines content and $(N_1)_{60}$ for magnitude 7.5 earthquakes using Equations 5-7 yield curves that are essentially identical to the curves plotted on Figure 2.

Several workshop participants suggested that liquefaction resistance should also increase with soil plasticity. Agreement could not be reached, however, on formulation of a correction for plasticity. There is very little empirical data from which such a correction could be developed. Nevertheless, some practitioners have been increasing CRR by about 10 percent for soils with fines and plasticity indices greater than 15 percent. This increase seemed appropriate to several participants, but consensus was not attained. The participants did agree, however, that plasticity should be measured as part of liquefaction investigations with the goal of better defining the influence of plasticity on liquefaction resistance.

Although not endorsed by workshop participants, Robertson and Wride (this report) reviewed fines content data as part of their workshop assignment to review liquefaction resistance criteria based on SPT measurements. They suggest correcting the calculated $(N_1)_{60}$ to an equivalent $(N_1)_{60cs}$ using a correction factor, K_s, which is solely a factor of fines content as noted below:

$$(N_1)_{60cs} = K_s\,(N_1)_{60} \tag{8a}$$

where

$$K_s = 1 + [(0.75/30)(FC - 5)] \tag{8b}$$

This recommendation is for soils with nonplastic fines (PI ≤ 5 percent). For soil with plastic fines, the correction factor, K_s, would likely be larger, but the available empirical data are insufficient at present to define a plasticity adjustment. For fines contents less than about 15 percent, the CRR curves are not greatly different than the curves of Seed et al. (1985). However, for fines contents greater than 15 percent, the Robertson and Wride CRR curves are significantly less conservative and plot to the left of the curves of Seed and others. Although there are little empirical data to control the positioning of the curves for fines contents greater than 15 percent and $(N_1)_{60}$ greater than 10, the general consensus of workshop participants was that the CRR curves should not be shifted to a less conservative position, as proposed by Robertson and Wride, without additional supporting data.

Other Corrections

In addition to grain characteristics, several other factors affect SPT results. One of the more important of these factors is the energy delivered to the SPT sampler. An energy ratio, ER, of 60% has generally been accepted as the reference value. The ER delivered by a particular SPT setup depends primarily on the type of hammer and anvil in the drilling system and on the method of hammer release. Approximate correction factors (C_E = ER/60%) to modify the SPT results to a 60% energy ratio for various types of hammers and anvils are listed in Table 2. Because of variations in drilling and testing equipment and differences in procedures used, a rather wide range in the energy correction factor, C_E, has been observed as noted in the table. Even when procedures are carefully monitored to conform to established standards, such as ASTM D-1686, considerable variation in C_E may occur because of minor variations in equipment and procedures. Even within a given borehole, variations in energy ratio between hammer blows or between tests typically may vary by as much

Table 2. Corrections to SPT (Modified from Skempton, 1986)
as Listed by Robertson and Wride (this report)

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$(P_a/\sigma'_{vo})^{0.5}$ $C_N \leq 2$
Energy ratio	Donut Hammer Safety Hammer Automatic-Trip Donut-Type Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole diameter	65 mm to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Rod length	3 m to 4 m 4 m to 6 m 6m 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 1.1 to 1.3

as ten percent. Thus, the recommended practice is to measure the energy ratio frequently at each site where the SPT is used. Where measurements can not be made, careful observation and notation of the equipment and procedures is required to estimate a C_E value for use in liquefaction resistance calculations. Use of good-quality testing equipment and carefully controlled testing procedures conforming to ASTM D-1686 will generally yield more consistent energy ratios and C_E values from the upper parts of the ranges listed in Table 2.

Additional correction factors are required for rod lengths less than 10 m and greater than 30 m, borehole diameters outside the recommended interval (65 mm to 125 mm), and sampling tubes without liners. Ranges of correction values for each of these variables are listed in Table 2. Careful documentation of drilling equipment and procedures, including measurement of ER, is required to select the most appropriate values for these correction factors. Even so, some uncertainty remains in the actual factors that should apply for any field operation.

Because the SPT N-value also varies with effective overburden stress, an overburden stress correction factor is also applied. This factor has commonly been calculated from the following equation (Liao and Whitman, 1968a):

$$C_N = (P_a/\sigma\_{vo})^{0.5} \qquad (9)$$

where C_N is a factor to correct measured penetration resistance for overburden pressure and P_a equals 100 kPa or approximately one atmosphere of pressure in the same units used for σ'_{vo}. The effective overburden pressure, σ'_{vo}, applied in this equation should be the overburden pressure that was effective at the time the SPT test was conducted. Even though the ground water level may have changed and a different water table level applied in the calculation of CSR (Equation 1), the correction of blow count requires use of the effective pressures that were effective at the time of drilling and testing.

The SPT N-value corrected for each of the above variables is given by the following equation:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \tag{10}$$

where N_m is the measured standard penetration resistance, C_E is the correction for hammer energy ratio (ER), C_B is a correction factor for borehole diameter, C_R is the correction factor for rod length, and C_S is the correction for samplers with or without liners. Suggested ranges of values for each of these correction factors are listed in Table 2. Selection of appropriate factors from within these ranges requires specific information on equipment and drilling procedures and engineering judgement. The engineer should become familiar with details of the SPT procedure to avoid or at least minimize possible errors associated with SPT testing and to gain expertise in selecting appropriate correction factors.

A final change recommended by workshop participants is the use of revised magnitude scaling factors rather than the original Seed and Idriss (1982) factors to adjust $CRR_{7.5}$ to CRR for other earthquake magnitudes. Magnitude scaling factors are addressed later in this report.

Cone Penetration Test (CPT)

The workshop participants were unable to reach consensus on CPT criteria for evaluating liquefaction resistance. Robertson and Wride (this report) developed the techniques presented below with input from workshop attendees. Robertson and Wride verified these criteria against SPT and other data from sites they had investigated. T.L. Youd and his students compared liquefaction resistances calculated from CPT criteria against field performance at nineteen sites where surface effects of liquefaction were or were not observed. The CPT criteria yielded apparently correct prediction of liquefaction or nonliquefaction with greater than 90 percent reliability. Youd and his students also compared liquefaction resistances from CPT criteria with results from SPT criteria at 50 sites with parallel CPT soundings and SPT borings, with a conclusion that the CPT criteria listed below yield consistent and reasonably conservative results. G.R. Martin (oral commun., February 1998) and several colleagues from southern California also compared results developed from parallel CPT soundings and SPT boreholes. They determined that liquefaction resistances estimated from the CPT procedure are on average slightly smaller, and thus more conservative, than liquefaction resistances developed from the parallel SPT tests. The above investigators endorse the CPT criteria listed below, but strongly recommend that at least one parallel borehole near a CPT sounding be drilled at each site to verify soil types and liquefaction resistances estimated from the CPT. I.M. Idriss, on the other hand, reviewed the CPT criteria and concluded that inadequate development and verification has been made to presently recommend these criteria to the geotechnical profession. In particular, Professor Idriss indicated that the correction for grain characteristics using I_c needs further consideration and verification. R.S. Olsen reviewed the CPT criteria listed below and concluded that the criteria are incorrectly developed and formulated. He recommends the criteria he has developed and presents in a paper submitted to the workshop (Olsen, this report).

A primary advantage of the CPT is that a nearly continuous profile of penetration resistance is developed for stratigraphic interpretation. The CPT results are generally more consistent and repeatable than results from other penetration tests listed in Table 1. The continuous profile also allows a more detailed interpretation of soil layers and soil types than the other tools listed in the Table. This stratigraphic capability makes the CPT particularly advantageous for reconnaissance investigations. In addition, CPT data can be used to estimate liquefaction resistance of penetrated soil layers. Thus the CPT can be used to develop preliminary soil and liquefaction resistance profiles for site investigations. These preliminary profiles should then be verified by other techniques, such as drilling and SPT testing.

In recent years, increased field performance data have become available at liquefaction sites investigated with CPT (Robertson and Wride, this report). These data have facilitated the development of CPT-based liquefaction resistance correlations. These correlations allow direct calculation of CRR, rather than through conversion of CPT measurements to equivalent SPT blow counts and then applying SPT criteria, a technique that was commonly applied in the past.

Figure 3 shows a chart developed by Robertson and Wride (this report) for determining cyclic resistance ratio ($CRR_{7.5}$) for clean sands (fines content, FC $\leq$ 5%) from CPT data. The chart, which is valid only for magnitude 7.5 earthquakes, shows calculated CRR plotted as a function of corrected and normalized CPT resistance, q_{c1N}, from sites where liquefaction effects were or were not observed following past earthquakes. A CRR curve separates regions of the plot with data indicative of liquefaction from regions indicative of nonliquefaction. Dashed curves showing approximate cyclic shear strain potential, γ_l, as a function of q_{c1N} are drawn on Figure 3 to emphasize that cyclic shear strain and ground deformation potential of liquefied soils decreases as penetration resistance increases.

The CRR curve in Figure 3 is approximated by the following simplified equation:

$$\text{If } (q_{c1N})_{cs} < 50 \qquad CRR_{7.5} = 0.833[(q_{c1N})_{cs}/1000] + 0.05 \qquad (11a)$$

$$\text{If } 50 \leq (q_{c1N})_{cs} < 160 \qquad CRR_{7.5} = 93\ [(q_{c1N})_{cs}/1000]^3 + 0.08 \qquad (11b)$$

where $(q_{c1N})_{cs}$ is the clean sand cone penetration resistance normalized to 100 kPa (approximately one atmosphere of pressure).

Normalization of Cone Penetration Resistance

Although cone penetration resistance is commonly corrected only for overburden stress, resulting in the term q_{c1}, truly normalized (i.e., dimensionless) cone penetration resistance corrected for overburden stress (q_{c1N}) is given by:

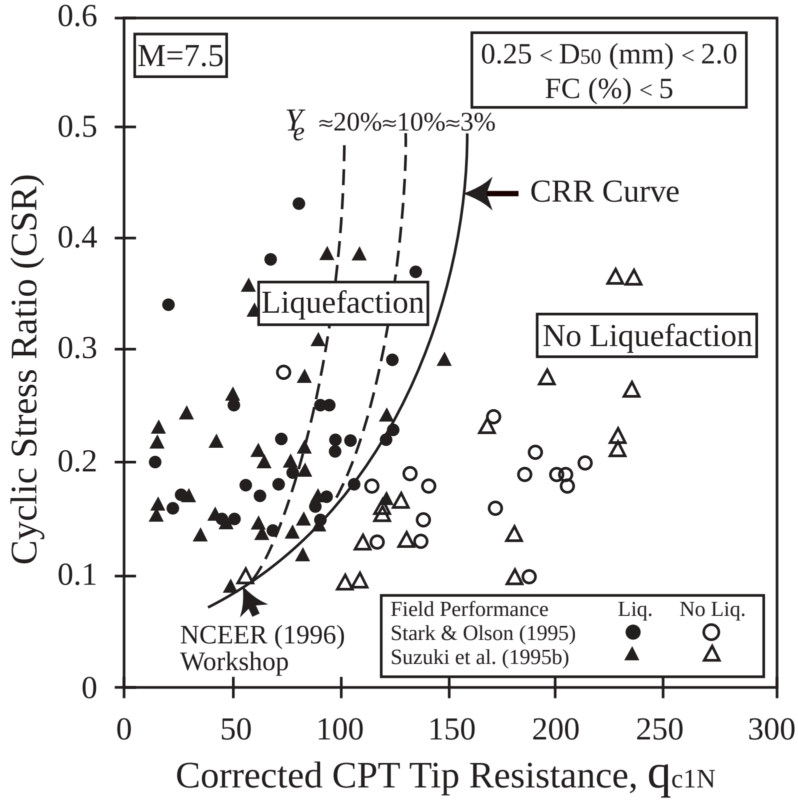


Figure 3 Curve Recommended for Calculation of CRR from CPT Data along with Empirical Liquefaction Data (After Robertson and Wride, this report)

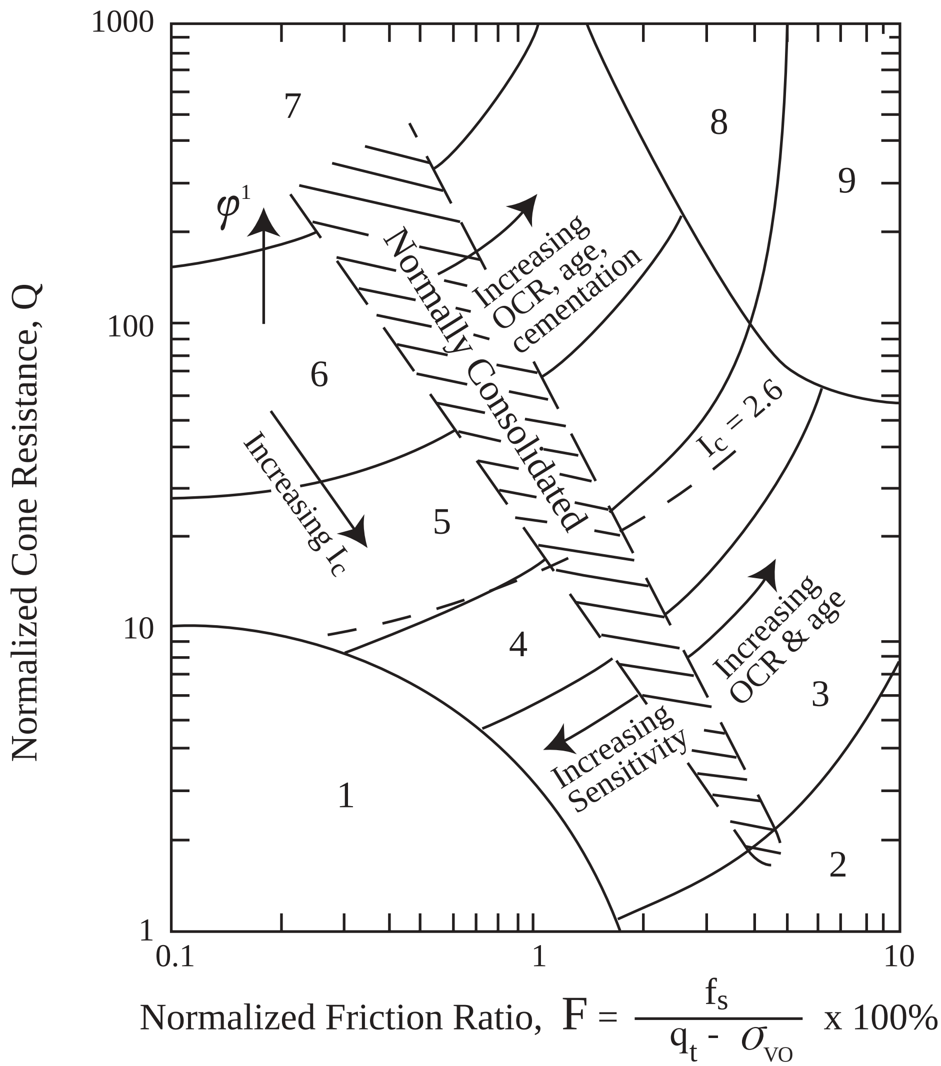
$$q_{c1N} = C_Q(q_c / P_a) \tag{12}$$

where

$$C_Q = (P_a/\sigma'_{vo})^n \tag{13}$$

C_Q is a normalizing factor for cone penetration resistance, P_a is 100 kPa or approximately one atmosphere of pressure in the same units used for σ_{vo}, and q_c is field cone penetration resistance measured at the tip. A maximum C_Q value of 2.0 is generally applied to CPT data at shallow depths. The value of the exponent, n, is dependent on grain characteristics of the soil and ranges from 0.5 for clean sands to 1.0 for clays (Olsen, this report). Selection of the value for use in liquefaction resistance calculations is discussed in the following paragraphs.

The CPT friction ratio (sleeve resistance, f_s, divided by cone tip resistance, q_c) generally increases with increasing fines content and soil plasticity. Robertson and Wride (this report) suggest that appropriate grain characteristics, such as approximate soil type and a rough estimate of fines content, termed apparent fines content herein, can be estimated directly from CPT data for sandy soils. Relationships recommended by Robertson and Wride are reproduced in Figures 4 and 5. The boundaries between soil types 2 through 7 on Figure 4 can be approximated as concentric circles (Jeffries and Davies, 1993). The radius of each circle, referred to as the soil behavior type index, I_c, is calculated from the following equation:



1. Sensitive, fine grained
2. Organic soils - peats
3. Clays - silty clay to clay
4. Silt mixtures - clayey silt to silty clay
5. Sand mixtures - silty sand to sandy silt
6. Sands - clean sand to silty sand
7. Gravelly sand to dense sand
8. Very stiff sand to clayey sand*
9. Very stiff, fine grained*

**Heavily overconsilidated or cemented*

Figure 4 CPT-Based Soil Behavior Type Chart Proposed by Robertson (1990)

$$I_c = [(3.47 - \log Q)^2 + (1.22 + \text{Log } F)^2]^{0.5} \quad (14)$$

where

$$Q = [(q_c - \sigma_{vo})/P_a][(P_a/\sigma'_{vo})^n] \quad (15)$$

and

$$F = [f_s/(q_c - \sigma_{vo})] \times 100\% \quad (16)$$

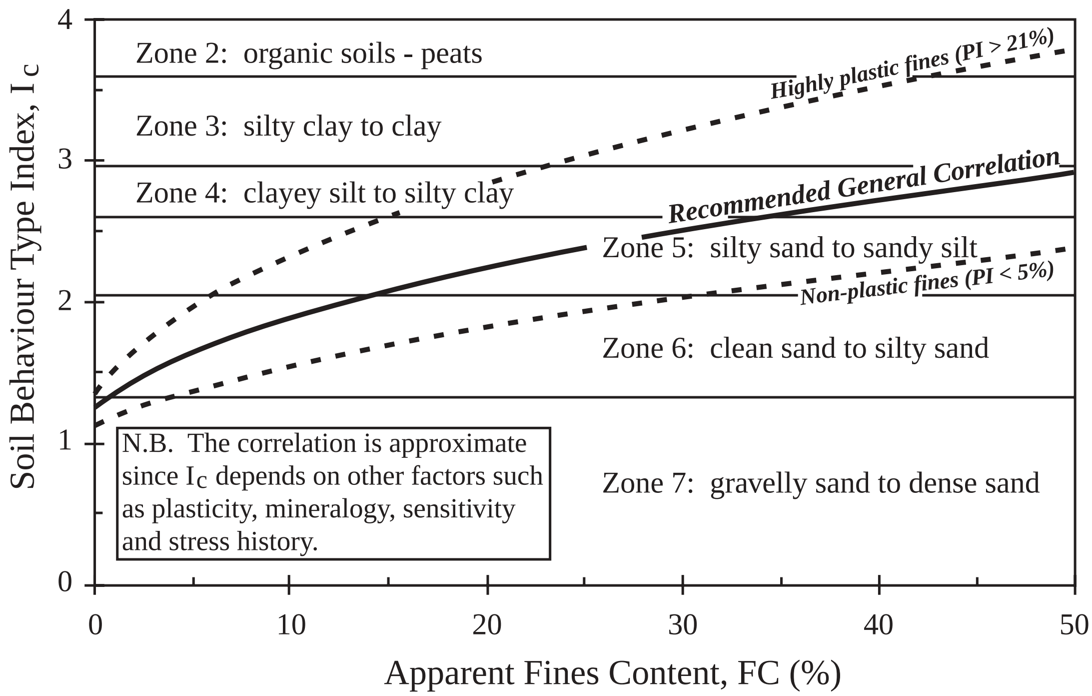


Figure 5 CPT Soil Behavior Type Index, I_c, Versus Apparent Fines Content for Normally Consolidated Soils (After Robertson and Wride, this report)

The soil behavior chart in Figure 4 was developed using an exponent, n, of 1.0, which is the appropriate value for clayey-type soils. For clean sands, however, an exponent value of 0.5 is more appropriate, and a value intermediate between 0.5 and 1.0 would be appropriate for silts and silty sands. Robertson and Wride recommend the following procedure for selecting an exponent and calculating the soil behavior type index, I_c.

The first step is to differentiate soil types characterized as clays from soil types characterized as sands and silts using Figure 4. This differentiation is performed by assuming an exponent, n, of 1.0 (characteristic of clays) and performing the following calculations. For clays, the dimensionless normalized CPT penetration resistance, Q, is defined as:

$$Q = [(q_c - \sigma_{vo})/P_a][P_a/\sigma\_{vo}]^{1.0} = [(q_c - \sigma_{vo})/\sigma\_{vo}] \quad (17)$$

If the calculated I_c calculated with an exponent of 1.0 is greater than 2.6, the soil is classed as clayey and is considered too clay-rich to liquefy. Samples should be taken and tested, however, to confirm the soil type and liquefaction resistance. Criteria, such as the Chinese criteria, might be applied to confirm that the soil is nonliquefiable. The so-called Chinese criteria, as defined by Seed and Idriss (1982), stipulate that liquefaction can only occur if all three of the following conditions are met:

(1) The clay content (particles smaller than 5μ) is less than 15 percent, by weight.
(2) The liquid limit is less than 35% percent.
(3) The natural moisture content is less than 0.9 times the liquid limit.

If the calculated I_c is less than 2.6, the soil is most likely granular in nature and Q should be recalculated using an exponent, n, of 0.5. For this calculation, C_Q should also be calculated with an exponent, n, of 0.5 (Equation 13), and q_{c1N} (calculated from Equation 12) substituted for Q in Equation 14. I_c should then be recalculated using Equation 14. If the recalculated I_c is less than 2.6, the soil can be classed as nonplastic and granular, and this I_c can be used to estimate liquefaction resistance as noted below. If the recalculated I_c is greater than 2.6, however, the soil is likely to be very silty and possibly plastic. In this instance, q_{c1N} should be recalculated from Equation 12 using an intermediate exponent, n, of 0.7 in Equation 13 and I_c recalculated from Equation 14 using the recalculated value for q_{c1N}. This intermediate I_c is then used to calculate liquefaction resistance. In this instance, a soil sample should be retrieved and tested to verify the soil type and whether the soil is liquefiable by other criteria, such as the Chinese criteria.

Because the relationship between I_c and soil type is rather approximate, the consensus of the workshop was that all soils characterized by an I_c of 2.4 or greater should be sampled and tested to confirm the soil type and to test the liquefiability with other criteria. Also, soil layers characterized by an I_c greater than 2.6, but with a normalized friction ratio, F, less than 1.0 percent (Region 1 of Figure 4) can be very sensitive, and hence should also be sampled and tested. Although perhaps not technically liquefiable according to the Chinese criteria, such sensitive soils may suffer severe softening and even strength loss under earthquake loading conditions.

Calculation of Clean Sand Equivalent Normalized Cone Penetration Resistance, $(q_{c1N})_{cs}$

To correct the normalized penetration resistance, (q_{c1N}), of sands with fines to an equivalent clean sand value,$(q_{c1N})_{cs}$, for use in the calculation of liquefaction resistance, CRR, the following relationships are applied:

$$(q_{c1N})_{cs} = K_c q_{c1N} \qquad (18)$$

where the CPT correction factor for grain characteristics, K_c, is defined by the following equations (Robertson and Wride, this report):

$$\text{For } I_c \leq 1.64 \quad K_c = 1.0 \qquad (19a)$$

$$\text{For } I_c > 1.64 \quad K_c = -0.403\, I_c^4 + 5.581\, I_c^3 - 21.63\, I_c^2 + 33.75\, I_c - 17.88 \qquad (19b)$$

Although the measured fines content could be substituted for the apparent fines content in Figure 5 to determine an I_c, such a substitution will likely yield erroneous results and should not be done. As noted above, I_c is a function of plasticity and other factors as well as fines content. Thus when using CPT data, I_c must be calculated from Equation 14 rather than estimated from the measured fines content.

The K_c versus I_c curve defined by Equations 19a and 19b is plotted on Figure 6. For I_c greater than 2.6, the curve is shown as a dashed line, indicating that the soils are most likely too clay rich or plastic to liquefy.

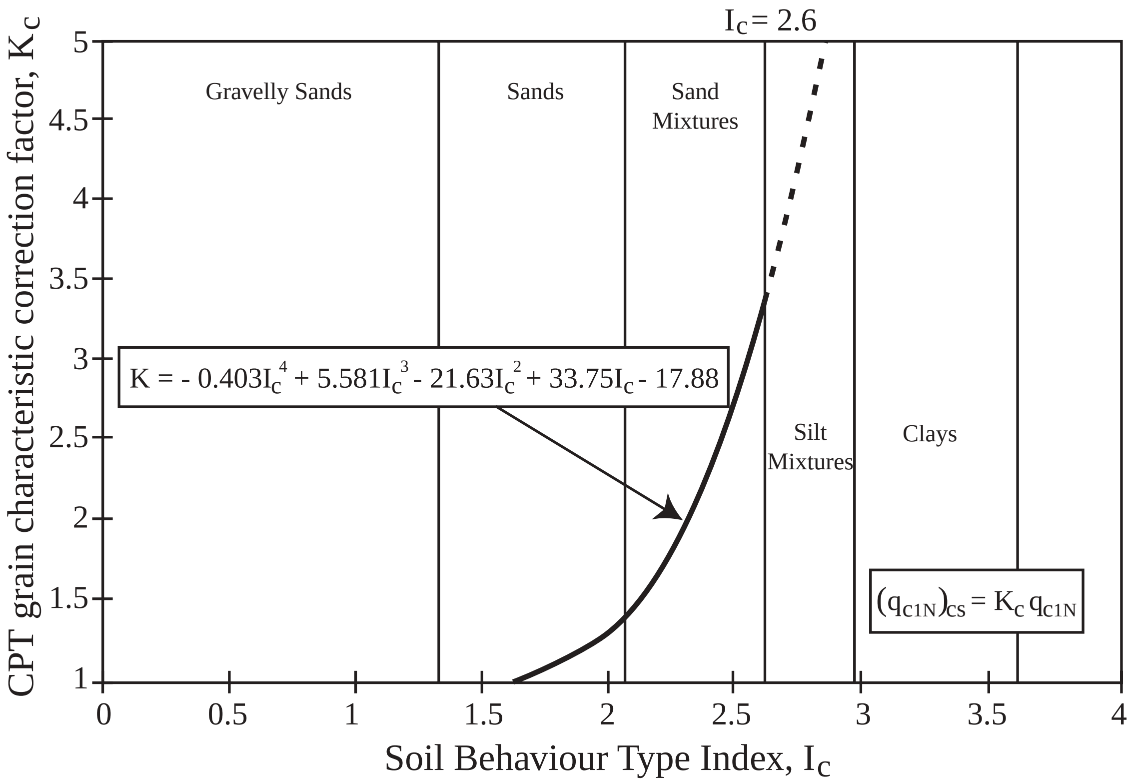


Figure 6 Grain-Characteristic Correction Factor, K_c, for Determination of Clean-Sand Equivalent CPT Resistance (After Robertson and Wride, this report)

With an appropriate I_c and K_c, Equations 11 and 19 can be used to calculate $CRR_{7.5}$. To adjust CRR to magnitudes smaller or larger than 7.5, the calculated $CRR_{7.5}$ is multiplied by an appropriate magnitude scaling factor. The same magnitude scaling factors are used with CPT data as with SPT or shear wave velocity data. Magnitude scaling factors are discussed in a later section of this report.

Although approached by a somewhat different route, the procedure for calculation of liquefaction resistance, CRR, given above is generally consistent and will generally give compatible results with the procedure proposed by Olsen (this report) for most level to gently sloping site conditions. As noted by Olsen, almost any CPT normalization technique, such as the procedure noted above, will give results consistent with his normalization procedure for shallow soil layers. For deep sites (σ'_{vo} > 150 kPa or depths greater than about 15 m), significant differences in results may develop between the two procedures. Those depths are deeper than most documented occurrences of liquefaction at natural sites and thus are deeper than the verified depth for the simplified procedure.

Correction of Cone Penetration Resistance for Thin Soil Layers

Theoretical as well as laboratory studies indicate that cone resistance is influenced by softer or stiffer soil layers above or below the cone tip. As a result, the CPT will not usually measure the full penetration resistance in thin sand layers sandwiched between layers of softer soils. The distance to which cone tip resistance is influenced by an approaching interface increases with stiffness of the

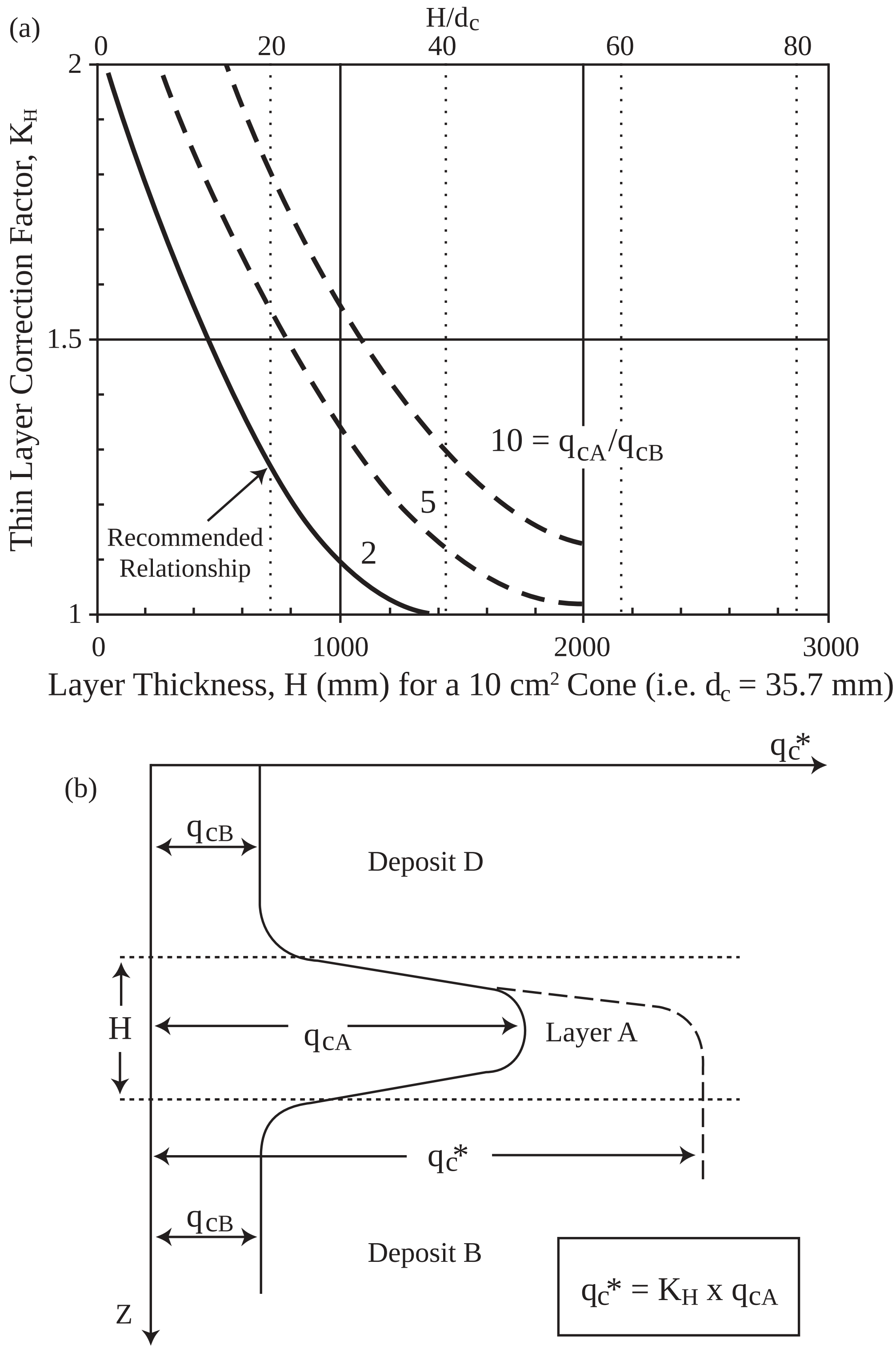


Figure 7 Thin-Layer Correction Factor, K_H, for Determination of Equivalent Thick-Layer CPT Resistance (After Robertson and Fear, 1995)

stiff layer. In soft clays or loose sands, the distance of influence can be as small as 2 to 3 cone diameters. In stiff clays or dense sands, the distance of influence may be as large as 20 cone diameters. (The diameter of the standard 10 cm^2 cone is 36 mm.) Thus care should be taken when interpreting cone resistance of sand layers sandwiched between silt or clay layers with lower penetration resistances. Based on a simplified elastic solution, Vreugdenhil et al. (1994) developed a procedure for estimating the full cone penetration resistance of thin stiff layers contained within softer strata. Based on this model, Robertson and Fear (1995) suggest a correction factor for cone resistance, K_H, as a function of layer thickness as shown in Figure 7. The correction applies only to thin stiff layers embedded within thick soft layers. Because the corrections have a reasonable trend, but appear rather large, Robertson and Fear (1995) recommend conservative corrections corresponding to $q_{cA}/q_{cB} = 2$ as shown on Figure 7. The equation for evaluating the correction factor, K_H, is

$$K_H = 0.5\,[(H/1{,}000) - 1.45]^2 + 1.0 \qquad (20)$$

where H is the thickness of the interbedded layer in mm, and q_{cA} and q_{cB} are cone resistances of the stiff and soft layers, respectively.

Shear Wave Velocity

During the past decade, several simplified procedures have been proposed for the use of field measurements of small-strain shear wave velocity, V_S, to assess liquefaction resistance of granular soils (Stokoe et al., 1988; Tokimatsu et al., 1991; Robertson et al., 1992; Kayen et al., 1992; Andrus, 1994; Lodge, 1994). The use of V_S as a field index of liquefaction resistance is justified because both V_S and CRR are similarly influenced by void ratio, effective confining stresses, stress history, and geologic age. The advantages of using V_S include the following: (1) V_S can be accurately measured in situ using a number of techniques such as crosshole and downhole seismic tests, the seismic cone penetration test, or spectral analysis of surface waves; (2) V_S measurements are possible in soils that are difficult to penetrate with CPT and SPT or to extract undisturbed samples, such as gravelly soils, and at sites where borings or soundings may not be permitted; (3) measurements can be performed in small laboratory specimens, allowing direct comparisons between measured laboratory and field behavior; and (4) V_S is directly related to small-strain shear modulus, a parameter required in analytical procedures for estimating dynamic soil response at small and intermediate shear strains.

Two significant limitations of using V_S in liquefaction hazard evaluations are that (1) seismic wave velocity measurements are made at small strains, whereas liquefaction is a large strain phenomenon; and (2) seismic testing does not provide samples for classification of soils and identification of nonliquefiable soft clay-rich soils. To compensate for the latter limitation, a limited number of borings should be drilled and samples taken to identify nonliquefiable clay-rich soils that might classify as liquefiable by V_S criteria and also to identify weakly cemented soils that might be liquefiable but classify as nonliquefiable because of their characteristically high V_S values.

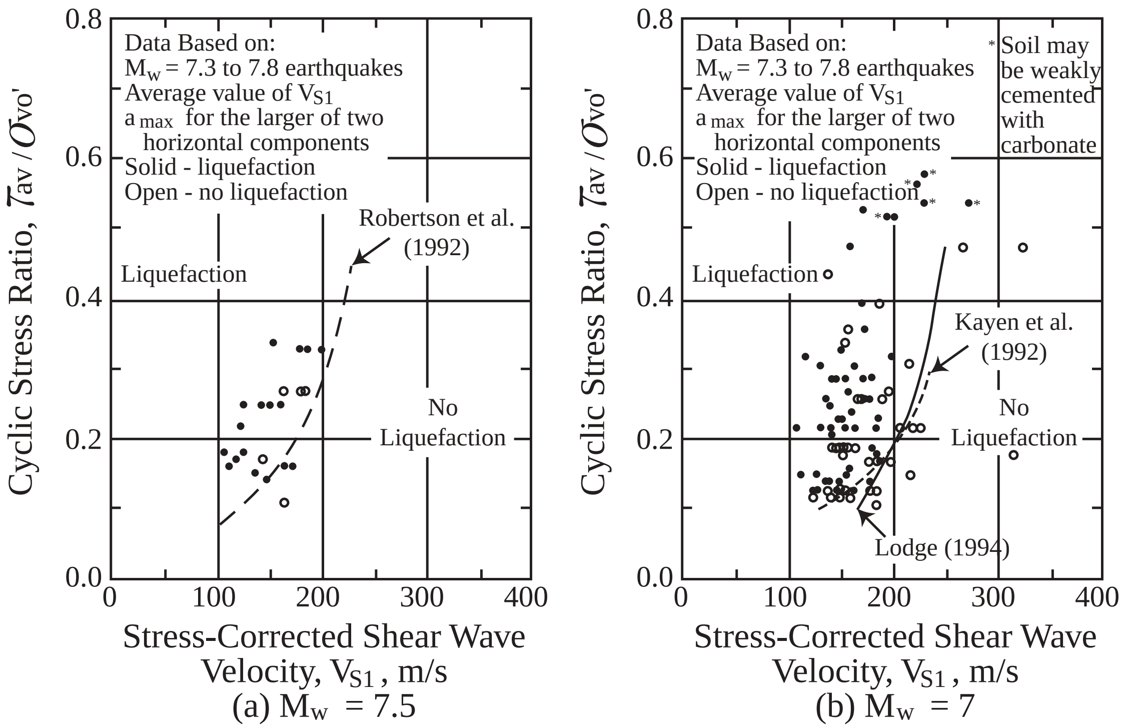


Figure 8 CSR Charts Based on Corrected Shear-Wave Velocities Suggested by (a) Robertson et al. (1992), and (b) Kayen et al. (1992) and Lodge (1994) (After Andrus and Stokoe, this report)

Criteria for Evaluating Liquefaction Resistance

Robertson et al. (1992) proposed a stress-based liquefaction assessment procedure using field performance data from sites in the Imperial Valley, California. These investigators normalized V_S by:

$$V_{S1} = V_S(P_a/\sigma'_{vo})^{0.25} \tag{21}$$

where P_a is a reference stress of 100 kPa, approximately atmospheric pressure, and σ'_{vo} is effective overburden pressure in kPa. Robertson et al. chose to modify V_S in terms of σ'_{vo} to follow the traditional procedures for modifying standard and cone penetration test resistances. The liquefaction resistance bound (CRR curve) determined by these investigators for magnitude 7.5 earthquakes is plotted on Figure 8a along with data calculated from several field sites where liquefaction did or did not occur. The cyclic stress ratios were calculated using estimates of a_{max} for the larger of two horizontal components of ground acceleration that would have occurred at the site in the absence of liquefaction.

Subsequent liquefaction resistance boundaries proposed by Kayen et al. (1992) and Lodge (1994) for magnitude 7 earthquakes are shown on Figure 8b. These curves are based on field performance

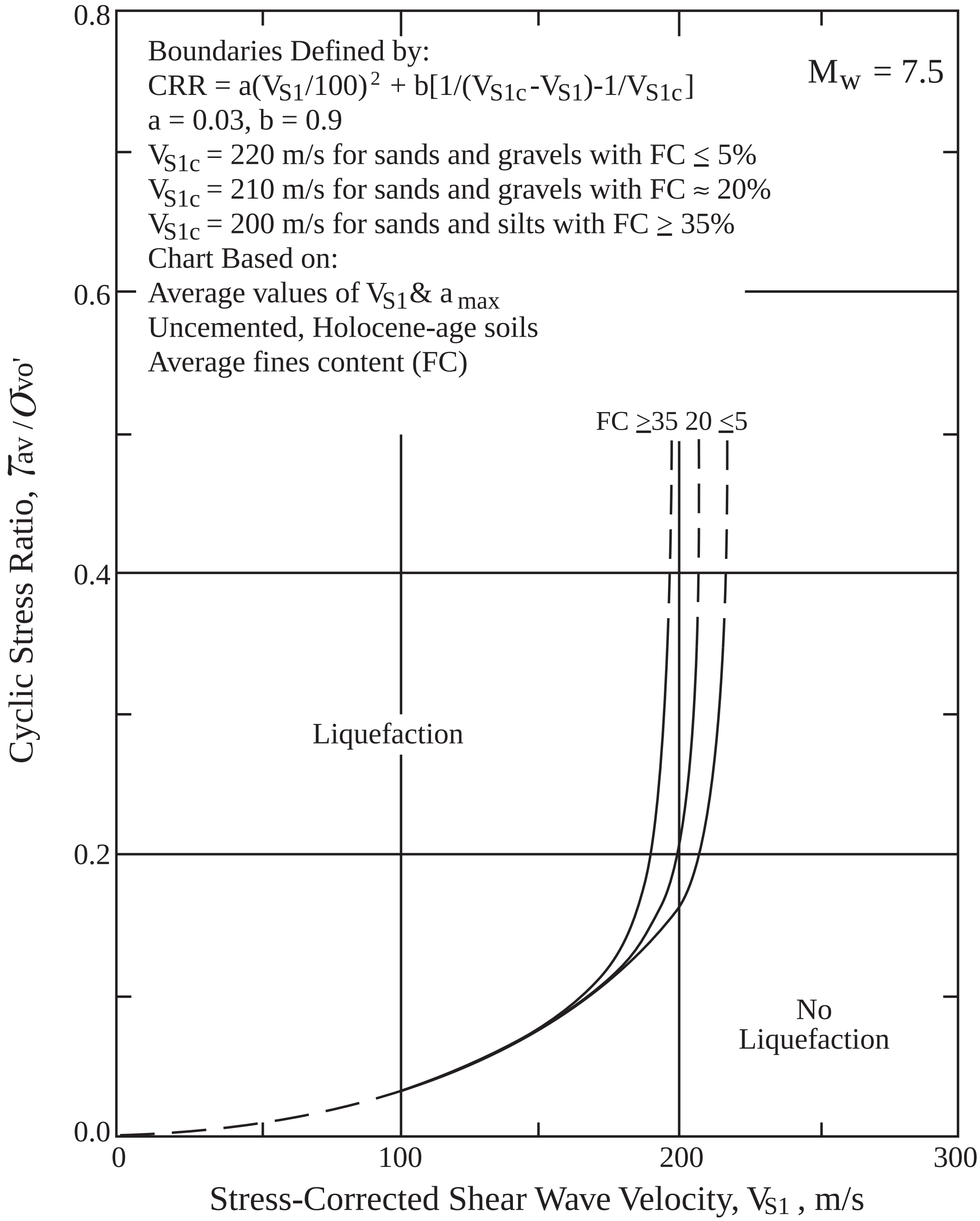


Figure 9 Curves Recommended by Workshop for Calculation of CRR from Corrected Shear Wave Velocity (After Andrus and Stokoe, this report)

data from the 1989 Loma Prieta earthquake. With few exceptions, the liquefaction case histories are bounded by the relationships proposed by these suggested bounds. The relationship proposed by Lodge (1994) provides a conservative lower boundary for liquefaction case histories with V_{S1} less than about 200 m/s. The relationship by Robertson et al. (1992) is the least conservative of the three.

Professor Ricardo Dobry suggested a relationship between cyclic resistance ratio and V_{S1} for constant average cyclic shear strain, γ_{av}, of the form:

$$CRR = \tau_{av}/\sigma'_{vo} = f(\gamma_{av})V_{S1}^2 \qquad (22)$$

where γ_{av} is constant average shear strain. This formula supports a CRR bound passing through the origin and provides a rational approach for extrapolating beyond the limits of the available field performance data, at least for lower values of V_{S1} ($V_{S1} \leq 125$ m/s).

For higher values of V_{S1}, Andrus and Stokoe (this report) reason that the CRR bound should become asymptotic to some limiting V_{S1} value. This limit is caused by the tendency of dense granular soils to exhibit dilative behavior at large strains. Thus Equation 22 is modified to:

$$\tau_{av}/\sigma'_{vo} = CRR = a(V_{S1}/100)^2 + b/(V_{S1c} - V_{S1}) - b/V_{S1c} \qquad (23)$$

where V_{S1c} is the critical value of V_{S1}, which separates contractive and dilative behavior, and a and b are curve fitting parameters.

Using the relationship between V_{S1} and CRR expressed by Equation 23, Andrus and Stokoe drew curves to separate data from sites where liquefaction effects were and were not observed. Best fit values for the constants a and b were 0.03 and 0.9, respectively, for magnitude 7.5 earthquakes. Andrus and Stokoe also determined the following best-fit values for V_{S1c}:

V_{S1c} = 220 m/s for sands and gravels with fines contents less than 5%
V_{S1c} = 210 m/s for sands and gravels with fines contents of about 20 %
V_{S1c} = 200 m/s for sands and gravels with fines contents greater than 35%

Figure 9 presents CRR boundaries recommended by Andrus and Stokoe for magnitude 7.5 earthquakes and uncemented Holocene-age soils with various fines contents. Although these boundaries pass through the origin, natural alluvial sandy soils with shallow water tables rarely have corrected shear wave velocities less than 100 m/s, even near ground surface. For a V_{S1} of 100 m/s and a magnitude 7.5 earthquake, the calculated CRR is 0.03. This minimal CRR value is generally consistent with intercept CRR values for the CPT and SPT procedures.

Equation 23 can be scaled to other magnitude values through use of magnitude scaling factors. These factors are discussed in a later section of this paper.

Becker Penetration Tests

Liquefaction resistance of non-gravelly soils has been evaluated primarily through CPT and SPT, with occasional V_S measurements. CPT and SPT measurements, however, are not generally reliable in gravelly soils. Large gravel particles may interfere with the normal deformation of soil materials around the penetrometer increasing penetration resistance. In an attempt to surmount these difficulties, several investigators have employed large-diameter penetrometers. The Becker penetration test (BPT) has become one of the more effective and widely used of these type of tools.

The BPT was developed in Canada in the late 1950s and consists of a 3-meter-long double-walled casing driven into the ground with a double-acting diesel-driven pile hammer. The hammer impacts are applied at the top of the casing and penetration is continuous. The Becker penetration resistance is defined as the number of blows required to drive the casing through an increment of 300 mm.

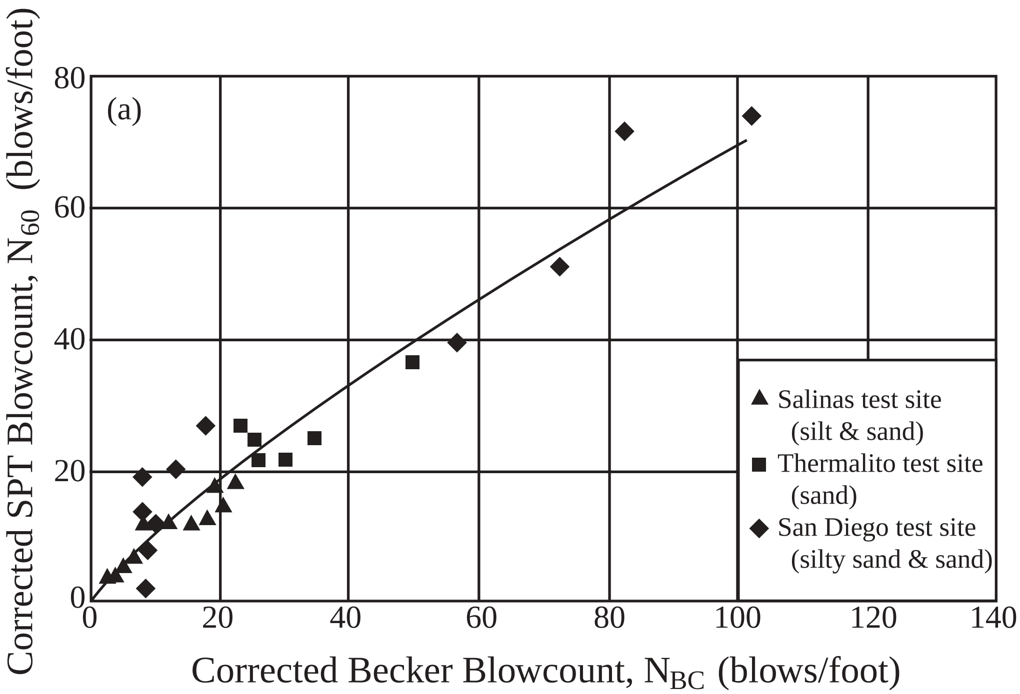
The BPT has not been standardized and several different types of equipment and procedures have been used. Also, only a few BPT blow counts have been measured at sites where liquefaction has occurred. Thus the BPT is not correlated directly with liquefaction resistance, but is used to estimate equivalent SPT blow counts though empirical correlation. The equivalent SPT blow count is then used to estimate liquefaction resistance.

To provide uniformity, Harder and Seed (1986) recommend employment of newer AP-1000 drill rigs equipped with supercharged diesel hammers, 168-mm O.D. casing , and a plugged bit. From several sites where both BPT and SPT tests were conducted in parallel soundings, Harder and Seed (1986) developed a preliminary correlation between Becker and standard penetration resistance (Figure 10a). Additional comparative data compiled since 1986 are plotted on Figure 10b. The original Harder and Seed correlation curve (solid line) is drawn on Figure 10b along with dashed curves representing 20% over- and under-predictions of SPT blow counts. These plots indicate that SPT blow counts can be roughly estimated from BPT measurements.

A major source of variation in BPT blow counts is deviations in hammer energy. Rather than measuring hammer energy directly, Harder and Seed (1986) monitored bounce-chamber pressures and found that uniform combustion conditions (e.g., full throttle with a supercharger) correlated rather well with variations in Becker blow count. From this information, Harder and Seed (this report) developed an energy correction procedure based on measured bounce-chamber pressure.

Direct measurement of transmitted hammer energy could provide a more theoretically rigorous correction for Becker hammer efficiency. Sy and Campanella (1994) and Sy et al. (1995) instrumented a small length of Becker casing with strain gages and accelerometers in an attempt to measure transferred energy. They analyzed the recorded data with a pile-driving analyzer to determine strain, force, acceleration, and velocity. The transferred energy was determined by time integration of force times velocity. They were able to verify many of the variations in hammer energy previously identified by Harder and Seed (1986), including effects of variable throttle settings and energy transmission efficiencies of various drill rigs. However, they were not able to reduce the scatter or uncertainty in converting BPT blow counts to SPT blow counts. Because the Sy and Campanella procedure requires considerably more effort than monitoring of bounce-chamber pressure without producing greatly improved results, the workshop participants agreed that the bounce-chamber technique appears adequate for routine practice.

Friction along the driven casing also influences penetration resistance. Harder and Seed (1986) did not evaluate the effect of casing friction; hence the correlation in Figure 10b intrinsically incorporates casing friction. Casing friction, however, remains a concern for depths greater than 30 m and for measurement of penetration resistance in soft soils underlying thick deposits of dense soil. Either of these circumstances could lead to greater casing friction than is intrinsically incorporated in the Seed and Harder correlation.



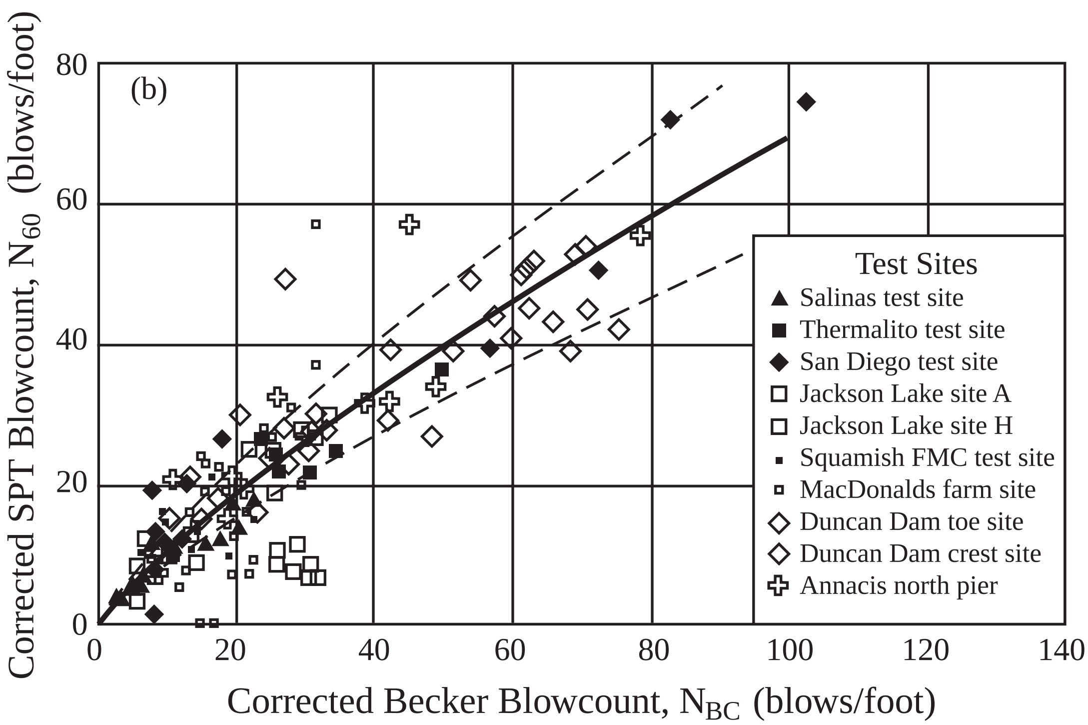


Figure 10 Correlation Between Corrected Becker Penetration Resistance, NBC, and Corrected SPT Resistance, N_{60}, (a) Harder and Seed (1986); (b) Data from Additional Sites (After Harder, this report)

The following procedures are recommended for routine practice: (1) The BPT should be conducted with newer AP-1000 drill rigs equipped with supercharged diesel hammers used to drive plugged 168-mm O.D. casing. (2) Bounce-chamber pressures should be used to adjust measured BPT blow counts to N_{bc} to account for variations in diesel hammer combustion efficiency. For most routine applications, correlations developed by Harder and Seed (1986) may be used for these adjustments. (3) The influence of casing friction is intrinsically accounted for in the Harder and Seed BPT-SPT correlation. This correlation, however, has not been verified and should not be used for depths greater than 30 meters or for sites with thick dense deposits overlying loose sands or gravels. For these conditions, mudded boreholes may be needed to reduce casing friction, or sophisticated wave-equation analyses may be applied to quantify frictional effects.

Magnitude Scaling Factors

In developing the simplified procedure, Seed and Idriss (1982) compiled a sizable data base from sites where liquefaction did or did not occur during earthquakes with magnitudes near 7.5. Analyses were made of these data to calculate cyclic stress ratios (CSR) and $(N_1)_{60}$ values for various sites where surface effects of liquefaction were or were not observed. Results from clean sand sites (fines content ≤ 5 percent) were plotted on a CSR versus $(N_1)_{60}$ plot. An updated version of that plot (Seed et al., 1985) is reproduced in Figure 2. A deterministic curve was drawn through the plot to separate regions with data indicative of liquefaction (solid symbols) from regions with data indicative of nonliquefaction (open symbols). Where there was a mixture of data, the curve was conservatively placed to ensure that data indicative of liquefaction plot above or to the left of the bounding curve. This curve, termed the simplified base curve or $CRR_{7.5}$ curve, is relatively well constrained by empirical data between CSR of 0.08 and 0.35 and is logically extrapolated to higher and lower values beyond that range. As shown in Figure 2, the workshop participants recommend bowing the lower part of the simplified base curve to intersect the ordinate of the plot at a CRR of about 0.05.

To adjust the simplified base curve to magnitudes smaller or larger than 7.5, Seed and Idriss (1982) introduced correction factors called "magnitude scaling factors." These factors are used to scale the simplified base curve upward or downward on the CSR versus $(N_1)_{60}$ plot. Conversely, magnitude weighting factors, which are the inverse of magnitude scaling factors, may be applied to correct CSR for magnitude. Either correcting CRR via magnitude scaling factors, or correcting CSR via magnitude weighting factors, leads to the same final result. Because the original papers by Seed and Idriss were written in terms of magnitude scaling factors, the use of magnitude scaling factors is continued in this report.

To illustrate the influence of magnitude scaling factors on calculated hazard, the equation for factor of safety (FS) against liquefaction can be written in terms of CRR, CSR, and MSF as follows:

$$FS = (CRR_{7.5}/CSR)MSF \qquad (24)$$

where $CRR_{7.5}$ is the cyclic resistance ratio determined for magnitude 7.5 earthquakes using Figure 2 or Equation 4 for SPT data, Figure 3 or Equation 11 for CPT data, or Figure 9 or Equation 23 for V_{S1} data. Equation 24 demonstrates that the factor of safety against development of liquefaction at a site is directly proportional to the magnitude scaling factor selected.

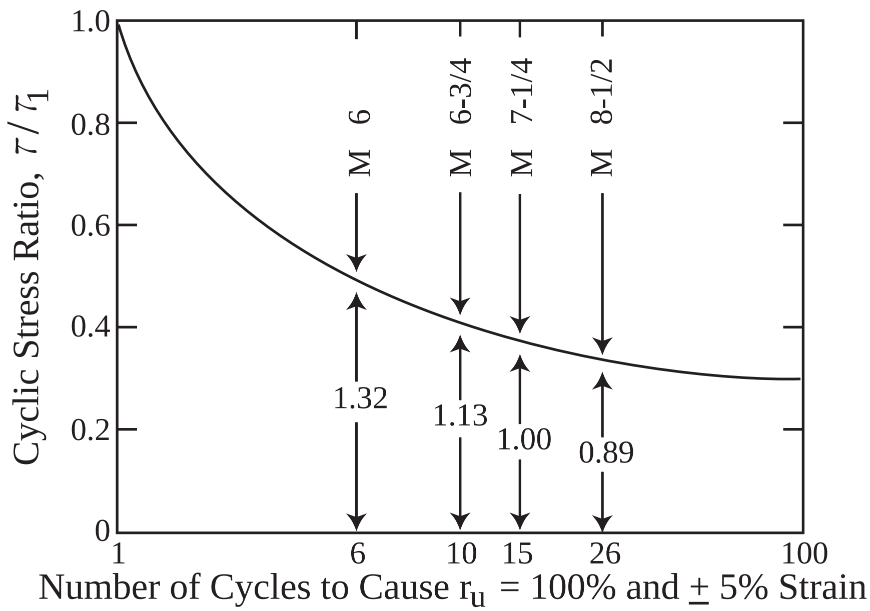


Figure 11 Representative Relationship Between CSR and Number of Cycles To Cause Liquefaction and (After Seed and Idriss, 1982)

Seed and Idriss (1982) Scaling Factors

Because of the limited empirical data available in the 1970s, Seed and Idriss (1982) were unable to narrowly constrain bounds between liquefaction and nonliquefaction regions on CRR plots for magnitudes other than 7.5. Consequently, they based their scaling factors on representative loading cycles and laboratory test results. From a study of strong-motion accelerograms, the number of representative loading cycles generated by an earthquake was correlated with earthquake magnitude. For example, magnitude 7.5 earthquakes were characterized by 15 loading cycles, whereas, magnitude 8.5 earthquakes were characterized by 26 loading cycles, and magnitude 6.5 earthquakes by 10 loading cycles. Second, laboratory tests were conducted to measure the number of loading cycles required to generate liquefaction and five percent cyclic strain. Laboratory tests were conducted using a variety of clean sands, void ratios, and ambient stress conditions. From these tests, a single representative curve was developed that relates cyclic stress ratio to the number of loading cycles required to generate liquefaction (Figure 11). By dividing CSR values from this curve for various numbers of cycles, representative of various earthquake magnitudes, by the CSR for 15 cycles (magnitude 7.5), the initial set of magnitude scaling factors was derived. These scaling factors are listed in Column 2 of Table 3 and are plotted on Figure 12. These magnitude scaling factors have been routinely applied in engineering practice since their introduction in 1982.

Idriss Scaling Factors

In preparing his H.B. Seed memorial lecture, I.M. Idriss reevaluated the data that he and the late Professor Seed had used to calculate the original (1982) magnitude scaling factors. In so doing, Idriss re-plotted the data on a log-log plot and found that the data plotted as a straight line. He further noted that one outlier point had strongly influenced the original analysis, causing the original

Table 3. Magnitude Scaling Factor Values Defined by Various Investigators (from Youd and Noble, Magnitude Scaling Factors, this report)

Mag-nitude, M (1)	Seed and Idriss (1982) (2)	Idriss (3)	Ambraseys (1988) (4)	Arango (1996) (5)	(6)	Andrus and Stokoe (in press) (7)	Youd and Noble (this report) P_L<20% (8)	P_L<32% (9)	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?

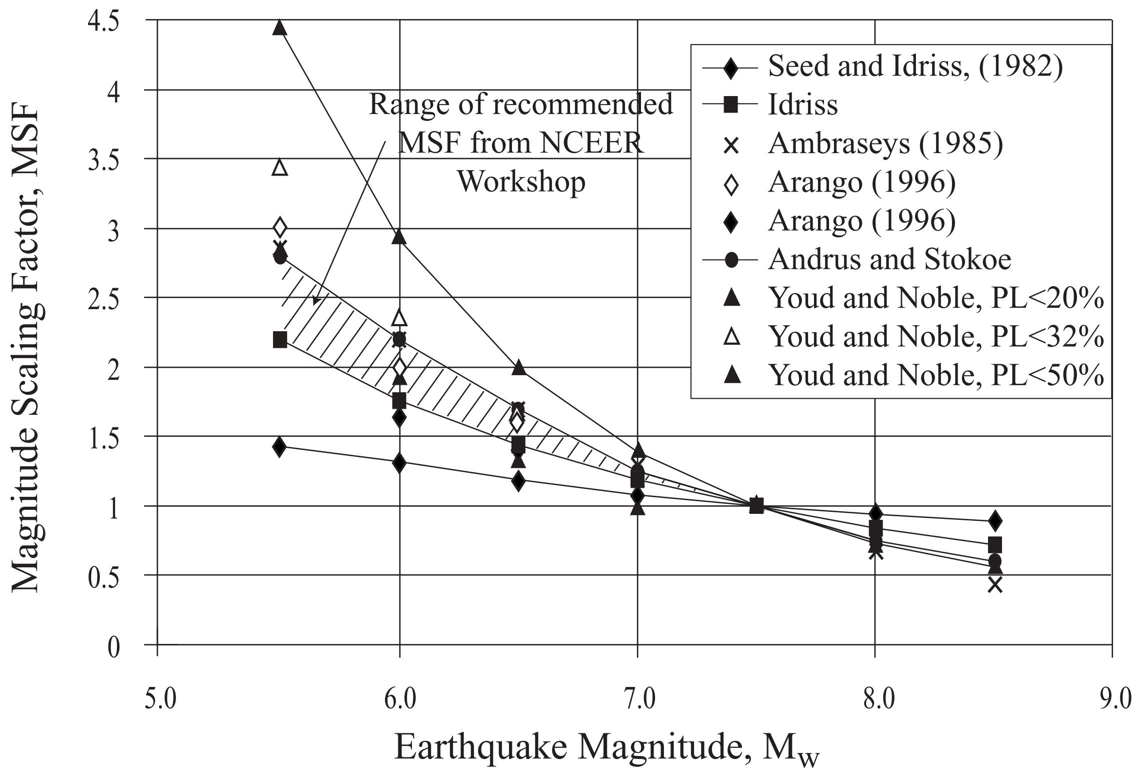


Figure 12 Magnitude Scaling Factors Derived by Various Investigators (After Youd and Noble, Magnitude Scaling Factors, this report)

plot to be nonlinear and characterized by unduly low values for magnitudes less than 7.5. Based on this reevaluation, Idriss defined a new set of magnitude scaling factors. These factors are listed in Column 3 of Table 3, plotted on Figure 12, and are defined by the following equation:

$$MSF = 10^{2.24}/M^{2.56} \tag{25}$$

Idriss recommends these revised scaling factors for use in engineering practice in place of the original factors.

The revised scaling factors are significantly larger than the original scaling factors for magnitudes less than 7.5 and somewhat smaller than the original factors for magnitudes greater than 7.5. Relative to the original scaling factors, the revised factors lead to a reduced calculated liquefaction hazard for magnitudes less than 7.5 and increased calculated hazard for magnitudes greater than 7.5.

Ambraseys Scaling Factors

Field performance data collected since the 1970s for magnitudes less than 7.5 indicate that the original Seed and Idriss (1982) scaling factors may be overly conservative. For example, Ambraseys (1988) analyzed liquefaction data compiled through the mid-1980s and plotted calculated cyclic stress ratios for sites that did or did not liquefy on CSR versus $(N_1)_{60}$ plots. From these plots, Ambraseys developed empirical exponential equations that define CRR as a function of $(N_1)_{60}$ and moment magnitude, M_w. By holding the value of $(N_1)_{60}$ constant in the equations and taking the ratio of CRR determined for various magnitudes of earthquakes to the CRR for a magnitude 7.5 earthquakes, Ambraseys derived the magnitude scaling factors listed in Column 4 of Table 3. These factors are also plotted on Figure 12. For magnitudes less than 7.5, the MSF suggested by Ambraseys are significantly greater than both the original factors developed by Seed and Idriss (Column 2, Table 3) and the revised factors by Idriss (Column 3). Because they are based on observational data, these factors have validity for estimating liquefaction hazard; however, they have not been widely used in engineering practice. Conversely, for magnitudes greater than 7.5, Ambraseys factors are significantly lower than the original (Seed and Idriss, 1982) and Idriss's revised scaling factors. Because there are little data to constrain Ambraseys' scaling factors for magnitudes greater than 7.5, these factors are uncertain, are likely overly conservative, and are not recommended for engineering practice.

Arango Scaling Factors

Arango (1996) developed two sets of magnitude scaling factors. The first set (Column 5, Table 3) is based on farthest observed liquefaction effects from the seismic energy source, the estimated average peak accelerations at those distant sites, and the absorbed seismic energy required to cause liquefaction. The second set (Column 6, Table 3) was developed from energy concepts and the relationship derived by Seed and Idriss (1982) between numbers of significant stress cycles and earthquake magnitude. The MSF listed in Column 5 are similar in value (within about 10%) to the MSF of Ambraseys (Column 4), and the MSF listed in Column 6 are similar in value (within 6%) to the revised MSF proposed by Idriss (Column 3).

Andrus and Stokoe Scaling Factors

From their studies of liquefaction resistance as a function of shear wave velocity, V_s, Andrus and Stokoe (this report) developed Equation 23 for calculating CRR from V_s for magnitude 7.5 earthquakes. Using this equation, Andrus and Stokoe drew curves on graphs with plotted values of CSR as a function of V_{S1} from sites where surface effects of liquefaction were or were not observed. Graphs were plotted for sites shaken by magnitude 6, 6.5, 7, and 7.5 earthquakes. The positions of the CRR curves were visually adjusted on each graph until a best fit bound was obtained. Magnitude scaling factors were then estimated by taking the ratio of CRR for a given magnitude to the CRR for magnitude 7.5 earthquakes. These MSF were then fitted to the following exponential function

$$MSF = (M_w/7.5)^{-3.3} \qquad (26)$$

Values for magnitudes less than 6 and greater than 7.5 were extrapolated from this equation. MSF values from this analysis are listed in Column 7, Table 3, and plotted on Figure 12. For magnitudes less than 7.5, the MSF proposed by Andrus and Stokoe are rather close in value (within about 5 percent) to the MSF proposed by Ambraseys. For magnitudes greater than 7.5, the Andrus and Stokoe MSF are slightly smaller than the revised MSF proposed by Idriss.

Youd and Noble Scaling Factors

Youd and Noble (Magnitude Scaling Factors, this report) used a logistic analysis to analyze case history data from sites where effects of liquefaction were or were not reported following past earthquakes. This analysis yielded the following probabilistic equation:

$$\text{Logit}(P_L) = \ln(P_L/(1-P_L)) = -7.633 + 2.256\,M_w - 0.258\,(N_1)_{60cs} + 3.095\ln CRR \qquad (27)$$

where P_L is the probability that liquefaction occurred, $1-P_L$ is the probability that liquefaction did not occur, and $(N_1)_{60cs}$ is the corrected blow count, including the correction for fines content. Youd and Noble recommend direct application of this equation to calculate the CRR for a given probability of liquefaction occurrence. In lieu of direct application, Youd and Noble define MSF for use with the simplified procedure. These MSF were developed by rotating the simplified base curve to near tangency with the probabilistic curves for P_L of 50%, 32%, and 20% and various earthquake magnitudes. These MSF are defined as the ratio of the ordinate of the rotated base curve at the point of near tangency to the ordinate of the unrotated simplified base curve at the same $(N_1)_{60cs}$. Because the rotated simplified base curves lie entirely below the given probability curve, CRR calculated with these MSF are characterized by smaller probability of liquefaction occurrence than the associated probabilistic curves. Thus the MSF listed in Columns 8, 9, and 10 (Table 3), are denoted by P_L<50%, P_L<32%, and P_L<20%, respectively. Because the derived MSF are less than 1.0, Youd and Noble do not recommend use of MSF for P_L<32% and P_L<20% for earthquakes with magnitudes greater than 7.0. Equations for defining the Youd and Noble MSF are listed below:

$$\text{Probability, } P_L < 20\%\ MSF = 10^{3.81}/M^{4.53} \quad \text{For } M < 7 \qquad (28)$$
$$\text{Probability, } P_L < 32\%\ MSF = 10^{3.74}/M^{4.33} \quad \text{For } M < 7 \qquad (29)$$
$$\text{Probability, } P_L < 50\%\ MSF = 10^{4.21}/M^{4.81} \quad \text{For } M < 7.75 \qquad (30)$$

Recommendations for Engineering Practice

The workshop participants reviewed the MSF listed in Table 3 and all but one (S.S.C. Liao) agree that the original factors were too conservative and that an increase is warranted for engineering practice for magnitudes less than 7.5. Rather than recommending a single set of factors, the workshop participants suggest a range of MSF with the engineer allowed to choose factors from within that range requisite with the conservatism required for the given application. For magnitudes less than 7.5, the lower bound for the recommended range is the revised set of magnitude scaling factors proposed by Idriss (Column 3, Table 3, or Equation 23). The upper bound for the suggested range is the MSF proposed by Andrus and Stokoe (Column 7, Table 3, or Equation 26). The upper bound values are consistent with MSF suggested by Ambraseys, Arango, and Youd and Noble for P_L<20% (generally within about 10 percent).

For magnitudes greater than 7.5, the factors recommended by Idriss (Column 3, Table 3; Equation 25) should be used for engineering practice. Above magnitude 7.5, these factors are smaller than the original Seed and Idriss (1982) factors, and hence application of the new factors leads to increased calculated liquefaction hazard compared to the original factors. The reasoning for this recommendation is that the original factors by Seed and Idriss (1982) may not have been sufficiently conservative for magnitudes greater than 7.5. There are insufficient case history data for earthquakes with magnitudes greater than 8 to support use of the lower MSF values listed in Table 3. These lower values were generally extrapolated from smaller magnitude earthquakes. Thus, these more conservative MSF are not recommended for engineering practice.

Corrections for High Overburden Pressures, Static Shear Stresses, and Age of Deposit

The correction factors K_σ and K_α were developed by Seed (1983) to adjust cyclic resistance ratios (CRR) to static overburden and shear stresses larger than those embodied in the development of the simplified procedure. As noted, the simplified procedure is only valid for level to gently sloping sites (low static shear stress) and depths less than about 15 m (low overburden pressures). The K_σ correction factor extends cyclic ratios to high overburden pressures, while the K_α correction factor allows extension of the simplified procedure to more steeply sloping ground conditions. Because there are virtually no case histories available to help define these correction factors, the results from laboratory test programs have been used to develop corrections for engineering practice.

K_σ Correction Factor

Cyclically loaded, isotropically consolidated triaxial compression tests show that while liquefaction resistance of a soil increases with increasing confining pressure, the resistance, as measured by the cyclic stress ratio, is a nonlinear function that decreases with increased normal stress. To incorporate the nonlinear effect of decreasing cyclic stress ratio with increasing confining pressure, Seed (1983) recommended incorporation of a correction factor, K_σ, for overburden pressures greater than 100 kPa. This factor allows correction of results obtained from the simplified procedure to overburden pressures that are greater than those generally extant in the observational data base from which the procedure was derived. Because of the lack of case history data, extrapolation of the simplified

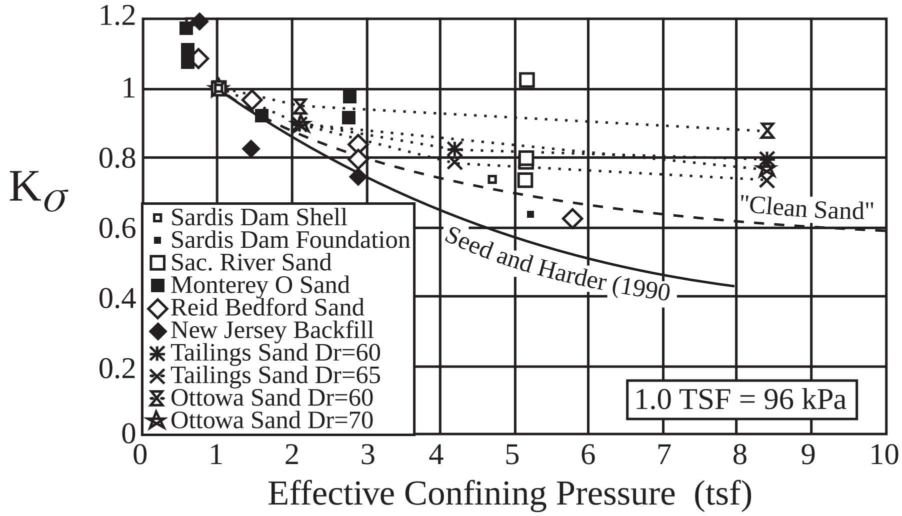


Figure 13 K_σ values determined by various investigators (After Seed and Harder, 1990)

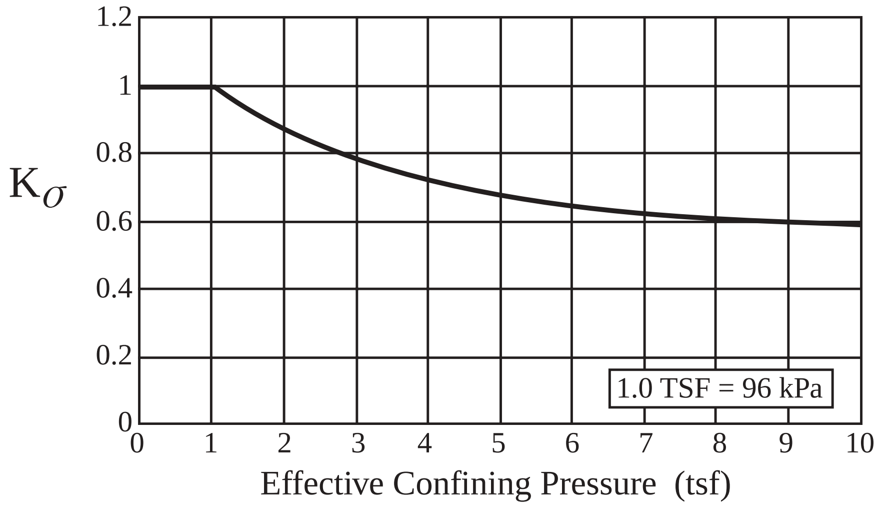


Figure 14. Minimum Values for K_σ Recommended for Clean and Silty Sands and Gravels (After Harder and Boulanger, this report)

procedure to depths greater than 15 m using K_σ factors yields results, such as factors of safety, that are less certain than at shallower depths.

The K_σ values developed by Seed (1983) were obtained by normalizing cyclic resistance ratios of isotropically consolidated cyclic triaxial compression tests to CRR values associated with an effective confining pressure of 100 kPa. For confining pressures greater than 100 kPa, the K_σ correction factor is less than one and decreases with increasing pressure. The original analyses by Seed (1983) yielded a band of suggested K_σ factors that decreased approximately linearly with effective overburden pressure from a value of 1.0 at 100 kPa to values ranging from about 0.40 to 0.65 at 800 kPa (Figure 13). Seed and Harder (1990) analyzed additional data and suggested generally lower values that are defined by a single concave curve with a K_σ value of 0.44 at an effective confining pressure of 800 kPa. Vaid et al. (1985) and Vaid and Thomas (1994) performed constant-volume cyclic simple shear tests on clean sands and derived smaller decreases in K_σ. From tests on mine-tailing, Ottawa, and Frazer Delta sands (Figure 13), several investigators (Byrne and Harder, 1991; Pillai and Byrne, 1994; Arango, 1996) calculated values for K_σ ranging from about 0.75 to 0.90 for effective overburden pressures of 1,000 kPa to minimal values of 0.67 for effective overburden pressures as great as 2,600 kPa. These analyses indicate that lower relative densities generally produced higher K_σ values. The various analyses confirm the considerable variability in derived K_σ values, and that the factors developed by Seed and Harder were overly conservative.

Based on the above discussion and a review of test results presented by Harder and Boulanger (this report), the workshop participants gained consensus that the Seed and Harder K_σ values were too conservative and that an increase is recommended for general engineering practice. Based on this review, the workshop recommended K_σ values represented by the curve in Figure 14 as minimal values for engineering practice for both clean and silty sands and for gravels.

K_α Correction Factor for Sloping Ground

Sloping ground induces static shear within the body of a soil mass before the onset of earthquake shaking. The relative magnitude of the static shear, τ_{st}, on the horizontal plane can be assessed by normalizing it with respect to the effective vertical stress, σ'_{vo}. The resulting parameter is called the alpha ratio, where $\alpha = \tau_{st}/\sigma'_{vo}$. For level ground conditions, the alpha ratio is zero. Early researchers suggested that the presence of a static shear stress always improved the cyclic resistance of a soil because higher cyclic shear stresses were required to cause stress reversal. This conclusion is true for dense soils under relatively low confining pressures. However, loose soils and some soils under high confining pressures have lower liquefaction resistance under the influence of initial static shear stresses than in the absence of these stresses. This behavior is due to the potential strain softening nature of very loose soils.

To incorporate the effect of static shear stresses on liquefaction resistance of soils, Seed (1983) recommended use of a correction factor, K_α. This factor is used to correct results obtained from the simplified procedure for level ground to sloping ground sites with constant static shear stress.

For the workshop, Harder and Boulanger (this report) reviewed past publications, tests, and analyses relative to K_α. They concluded that the wide ranges in potential K_α values developed by past investigators indicate a lack of consensus and a need for continued research and field verification of the effects of static shear stress on liquefaction resistance. Different rates of pore pressure generation and different limiting values for excess pore pressure at different locations within a slope make liquefaction analyses for sloping ground conditions an extremely complicated endeavor.

The workshop participants agreed that the evaluation of liquefaction resistance beneath sloping ground or embankments (slopes greater than about six percent) is not well understood and that such evaluations are beyond routine application of the simplified procedure. Although curves relating K_α to α have been published (Harder and Boulanger, this report), the participants concluded that general recommendations for use of K_α by the engineering profession is not advisable at this time.

Influence of Age of Deposit

Several investigators have shown that liquefaction resistance of soils increases with age. For example, Seed (1979) observed significant increases in liquefaction resistance with age of reconstituted sand specimens tested in the laboratory. Cyclically loaded tests were conducted on freshly reconstituted sand specimens and on similar sand specimens at periods ranging up to one hundred days. Increases of as much as 25 percent in cyclic resistance ratio were noted between the freshly constituted and the 100-day-old specimens. Youd and Hoose (1977) and Youd and Perkins (1978) note that liquefaction resistance increases markedly with geologic age. Sediments deposited within the past few hundred years are generally much more susceptible to liquefaction than older Holocene sediments; Pleistocene sediments are even more resistant; and Pre-Pleistocene sediments are essentially insusceptible to liquefaction. Although qualitative increases in liquefaction resistance have been well documented, insufficient quantitative data have been assembled from which correction factors for age can be defined.

The age, and concomitantly the liquefaction resistance, of naturally sedimented deposits generally increases with depth. In natural soils, this increase may partially or wholly counteract the influence of the K_σ factor which generates an apparent decrease in liquefaction resistance with depth. In the absence of quantitative correction factors for age, engineering judgement is required in assessing liquefaction resistance of sediments older than a few hundred years. In some instances where deeper sediments have been dated as more than a few thousand years old, knowledgeable engineers have ignored the K_σ factor as partial compensation for unquantifiable, but known increases in liquefaction resistance with age. For man-made structures, such as thick fills and embankment dams, ageing effects are generally minimal and should be ignored in calculating liquefaction resistance.

Seismic Factors

Application of the simplified procedure for evaluating liquefaction resistance requires estimates of earthquake magnitude and peak horizontal ground acceleration. In the procedure, these factors characterize duration and intensity of ground shaking, respectively. The workshop addressed the following questions with respect to selection of magnitude and peak acceleration.

Earthquake Magnitude

Records from past earthquakes indicate that the relationship between duration and magnitude is rather uncertain and that factors other than magnitude influence duration. For example, unilateral faulting, in which rupture begins at one end of the fault and propagates to the other, usually produces longer shaking duration for a given magnitude than bilateral faulting, in which slip begins near the midpoint on the fault and propagates in both directions. Duration also generally increases with distance from the seismic energy source and may vary with site conditions and with bedrock topography (basin effects). The workshop addressed the following questions with respect to the use of magnitude as an index for shaking duration, and developed the following consensus answers.

Question: Should correlations or correction factors be developed to adjust duration of shaking to account for the influence of earthquake source mechanism and other factors?

Answer: Faulting characteristics and variations in shaking duration are difficult to predict in advance of an earthquake event. The influence of distance is generally of secondary importance within the range of distances to which potentially damaging effects of liquefaction commonly develop. Basin effects are not yet sufficiently predictable to be adequately accounted for in engineering practice. Thus workshop participants recommend continued use of conservative relationship between magnitude and duration embodied in the simplified procedure for routine evaluation of liquefaction resistance.

Question: An important difference between eastern US earthquakes and western US earthquakes is that eastern ground motions are generally richer in high frequency energy and thus could generate more significant stress cycles and equivalently longer durations than western earthquakes of the same magnitude. Should a correction be made to account for higher frequencies of ground motions generated by eastern US earthquakes?

Answer: The high-frequency motions of eastern earthquakes are generally limited to rock sites. High-frequency motions attenuate or are damped out rather quickly as they propagate through soil layers. This filtering action reduces the high-frequency energy at soil sites and should reduce differences in numbers of significant loading cycles between eastern and western earthquakes. Because liquefaction occurs only within soil strata, duration differences on soil sites between eastern and western earthquakes are not likely to be great. Without more instrumentally recorded data from which differences in ground motion characteristics can be quantified, there is little basis for the development of additional correction factors for eastern localities.

Another difference between eastern and western US earthquakes is that strong ground motions generally propagate to greater distances in the east than in the west. By applying present state-of-the-art procedures for estimating peak ground acceleration at eastern sites, differences in ground motion propagation between western and eastern earthquakes are properly accounted for.

Question: Which magnitude scale should be used by engineers in selecting a magnitude for use in liquefaction resistance analyses?

Answer: Seismologists commonly calculate earthquake magnitudes using five different scales: (1) local or Richter magnitude, M_L; (2) surface-wave magnitude, M_s; (3) short-period body-wave magnitude, m_b; (4) long-period body-wave magnitude, m_B; and (5) moment magnitude, M_w. Moment magnitude is the scale most commonly used for engineering applications and is the scale preferred for calculation of liquefaction resistance. As shown on Figure 15, magnitudes from other scales may be substituted directly for M_w within the following limits: $M_L < 6$, $m_B < 7.5$, and $6 < M_s < 8$. m_b, a scale commonly applied in the eastern US, may be used for magnitudes between 5 and 6, provided such magnitudes are corrected to M_w using the curves plotted in Figure 15 (Idriss, 1985).

Peak Acceleration

In the simplified procedure, peak horizontal acceleration (a_{max}) is used to characterize the intensity of ground shaking. To provide guidance for estimation of a_{max}, the workshop addressed the following questions.

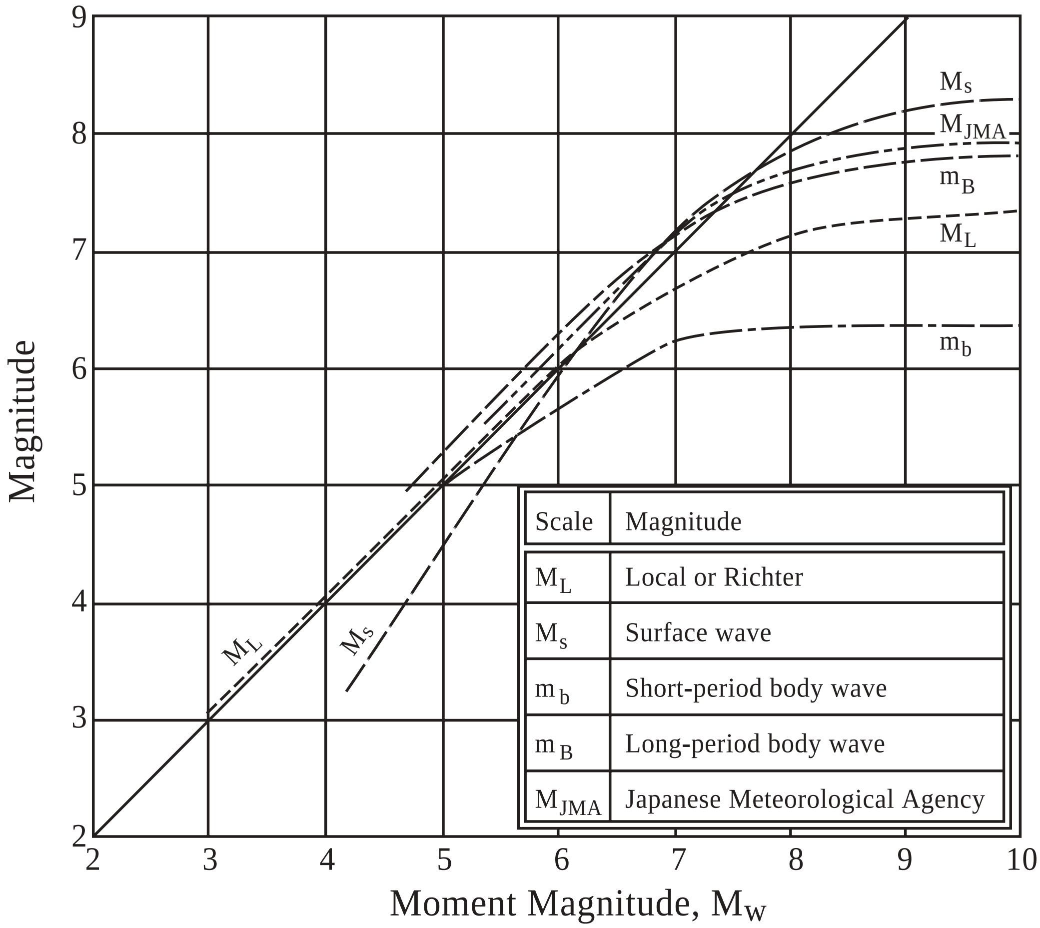


Figure 15 Relationship between Moment, M_w, and Other Magnitude Scales (After Heaton et al., 1982)

<u>Question</u>: What procedures are preferred for estimating a_{max} at potentially liquefiable sites?

<u>Answer</u>: The following three methods, in order of preference, may be used for estimating a_{max}:

(1) The preferred method for estimating a_{max} at a site is through application of empirical correlations for attenuation of a_{max} as a function of earthquake magnitude, distance from the seismic energy source, and local site conditions. Several correlations have been developed for estimating a_{max} for sites on bedrock or stiff to moderately stiff soils. Preliminary attenuation relationships have also been developed for soft soil sites (Idriss, 1991). Selection of an attenuation relationship should be based on factors such as region of the country, type of faulting, site condition, etc.

(2) For soft sites and other soil profiles that are not compatible with available attenuation relationships, a_{max} may be estimated from local site response analyses. Computer programs such as SHAKE, DESRA, etc., may be used for these calculations. Input ground motions in the form of recorded accelerograms are preferable to synthetic records. Accelerograms

derived from white noise should be avoided. A suite of plausible earthquake records should be used in the analysis, including as many records as feasible from earthquakes with similar magnitudes, source distances, etc.

(3) The third and least desirable method for estimating peak ground acceleration is through amplification ratios, such as those developed by Idriss (1990; 1991), Seed et al.(1994), and BSSC (1994). These factors use a multiplier or ratio by which bedrock outcrop motions are amplified to estimate motions at ground surface. Because amplification ratios are influenced by strain level, earthquake magnitude, and perhaps frequency content, caution and considerable engineering judgment are required in the application of these relationships.

Question: Which peak acceleration should be used? (a) the largest horizontal acceleration recorded on a three-component accelerogram; (b) the geometric mean (square root of the product) of the two maximum horizontal components; or (c) a vectorial combination of horizontal accelerations.

Answer: According to I.M. Idriss (oral communication at workshop), where recorded motions were available, the larger of the two horizontal peak components of acceleration were used in the original development of the simplified procedure. Where recorded values were not available, which was the circumstance for most sites in the data base, peak acceleration values were estimated from attenuation relationships based on the geometric mean of the two orthogonal peak horizontal accelerations. In nearly all instances where recorded motions were used, the peaks from the two horizontal records were approximately equal. Thus where a single peak was used that peak and the geometric mean of the two peaks were about the same value. Based on this information, the workshop participants concurred that use of the geometric mean is more consistent with the derivation of the procedure and is preferred for use in engineering practice. However, use of the larger of the two orthogonal peak accelerations would be conservative and is allowable. Vectorial accelerations are seldom calculated and should not be used. Peak vertical accelerations are ignored for calculation of liquefaction resistance.

Question: Liquefaction usually develops at soil sites where ground motion amplification may occur and where sediments may soften as excess pore pressures develop. How should investigators account for these factors in estimating peak acceleration?

Answer: The procedure recommended by the workshop is to calculate or estimate a peak acceleration that incorporates the influence of site amplification, but neglects the influence of excess pore-water pressure. Simply stated, the peak acceleration to be used in liquefaction resistance evaluations is the peak horizontal acceleration that would have occurred at ground surface at the site in the absence of increased pore-water pressure or the occurrence of liquefaction.

Question: Should high-frequency spikes (periods less than 0.1 sec) in acceleration records be considered or ignored?

Answer: In general, short-duration, high-frequency acceleration spikes should be ignored for liquefaction resistance evaluations. By using attenuation relationships for estimation of peak acceleration, as noted above, high frequency spikes are essentially ignored because few high-frequency peaks are incorporated in data bases from which attenuation relationships have been derived. Similarly, ground response analyses programs such as SHAKE and DESRA generally attenuate or filter out high-frequency spikes, reducing their influence. Where amplification ratios are used engineering judgment should be used to determine which bedrock accelerations should be amplified.

Energy-Based Criteria and Probabilistic Analyses

The workshop considered two additional topics: liquefaction resistance criteria based on seismic energy passing through a liquefiable layer (Youd et al., this report) and probabilistic analyses of case history data (Youd and Noble, Statistical and Probabilistic Analyses, this report). Although risk analyses for several localities and facilities have been made using probabilistic criteria, the workshop attendees agreed that probabilistic procedures are still outside the mainstream of standard practice. Similarly, energy-based criteria need further development before recommendations can be made for general practice. The workshop participants did agree that research and development should continue on both of these potentially useful procedures.

Conclusions

The participants in the NCEER workshop reviewed the state-of-the-art for evaluation of liquefaction resistance and proposed several augmentations to that procedure that have been developed over the past ten years. Specific conclusions, including recommended procedures and equations, are listed within each section of this summary paper. General consensus recommendations from the workshop are as follows:

1. Four field tests are recommended for general use in evaluating liquefaction resistance--the cone penetration test (CPT), the standard penetration test (SPT), measurement of shear-wave velocity (V_s), and for gravelly sites, the Becker penetration test (BPT). The workshop reviewed and revised criteria for each test to incorporate recent developments and to maximize compatibilities between liquefaction resistances determined via the various tests. Each field test has its advantages and limitations. The CPT provides the most detailed soil stratigraphy and provides a preliminary estimate of liquefaction resistance. The SPT has been used more widely and provides disturbed soil samples from which fines content and other grain characteristics can be determined. V_s measurements provide fundamental information for evaluation of small-strain constitutive relations and can be applied at gravelly sites where CPT and SPT may not be reliable. The BPT test has been used primarily at gravelly sites and requires use of rough correlations between BPT and SPT. In many instances, two or more test procedures should be applied to assure that both adequate definition of soil stratigraphy and a consistent evaluation of liquefaction resistance is attained.

2. The magnitude scaling factors originally derived by Seed and Idriss (1982) have proven to be very conservative for earthquake magnitudes less than 7.5. The consensus of the workshop was that a range of scaling factors should be recommended for engineering practice, with the lower end of the range being the revised MSF recommended by Idriss (Column 3, Table 3),

and the upper end of the range being the MSF suggested by Andrus and Stokoe (Column 7, Table 3). These MSF are defined by Equations 25 and 26, respectively. For magnitudes greater than 7.5, the revised factors by Idriss (Column 3, Table 3) should be used. The latter factors are significantly more conservative than the original Seed and Idriss (1982) factors, but the consensus was that these more conservative factors should be applied.

3. The K_σ factors suggested by Seed and Harder (1990) are too conservative for recommended use in general engineering practice. The workshop participants recommend the K_σ values represented by the curve in Figure 14 as minimal values for engineering practice for clean and silty sands and for gravels.

4. The workshop participants agreed that the evaluation of liquefaction resistance beneath sloping ground or embankments (slopes greater than about six percent) is not well understood at this time and that such evaluations are beyond the applicability of the simplified procedure. Special expertise is required for evaluation of liquefaction resistance beneath sloping ground.

5. Moment magnitude, M_w, should be used as an estimate of earthquake size for liquefaction resistance calculations. No general corrections are recommended to adjust earthquake magnitude to account for differences in duration due to source mechanism or geographic region (eastern versus western US earthquakes).

6. The peak acceleration, a_{max}, recommended for calculation of cyclic stress ratio, CSR, is the a_{max} that would have occurred at the site in the absence of pore pressure increases or liquefaction generated by the earthquake. Application of attenuation relationships compatible with conditions at a given site is the preferred procedure for estimating a_{max}. Where site conditions are incompatible with existing attenuation relationships, site-specific response calculations, using programs such as SHAKE or DESRA, should be used. The least preferable technique is application of amplification factors.

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