Chapter 7

Analysis of Soil Borings for Liquefaction Resistance

7.1. Introduction.

This chapter addresses the analysis of subsurface soil data to determine the factor of safety against liquefaction and predict the liquefied soil thickness. The results of this analysis are required to compute the EPOLLS geotechnical parameters defined in Section 6.7. The EPOLIQAN code, documented in Appendix C, was written to calculate these parameters from the data in a single soil boring. Output results from the EPOLIQAN code, for each soil boring at a site, were used to compute the average and range of the geotechnical parameters for the EPOLLS case studies. The procedures employed in the EPOLIQAN code to assess the liquefaction resistance of a soil deposit and predict the liquefied thickness, based on Standard Penetration Test blowcounts in a single boring log, are discussed in this chapter.

It is widely accepted that only recent sediments or fills of saturated, cohesionless soils at shallow depths will liquefy in a large magnitude earthquake. As discussed in Chapter 2, the liquefaction susceptibility of a specific deposit is affected by geologic history, confining pressure, density, and characteristics of the soil grains. An approximate assessment of liquefaction potential can be made on the basis of ground water levels and depositional history (type, age, and depth of sediment). Based on observations from past earthquakes, Youd and Perkins (1978) have tabulated the liquefaction susceptibility of sedimentary deposits in qualitative terms. Youd and Perkins' criteria, reproduced here as Table 7.1, are useful for identifying sites prone to liquefaction in large earthquakes. When combined with topographic information, the criteria in Table 7.1 can be used to predict where lateral spreading may develop as a result of soil liquefaction.

More quantitative assessments of liquefaction susceptibility are possible with information from subsurface soil explorations. Two basic approaches have been used to predict the liquefaction potential of soil strata (De Alba et al. 1976; Seed 1979; Seed et al. 1983):

(1) Evaluations based on a comparison of the stresses induced by an earthquake and the stress conditions causing liquefaction in cyclic laboratory tests on soil samples.

(2) Empirical methods based on measurements of in situ soil strength and observations of field performance in previous earthquakes.

Unfortunately, liquefaction assessments based on laboratory tests are hindered by limitations in the ability of laboratory equipment to reproduce field stress conditions in small soil samples. Even more problematic, disturbance of field samples is nearly impossible to avoid and very difficult to quantify in laboratory tests (Seed 1979). As a result, early evaluations based on laboratory tests were often overly conservative in predicting liquefaction (Peck 1979). Consequently, empirical correlations based directly on observations of field behavior are usually preferred for assessments of liquefaction potential in soil deposits. These methods are commonly based on field penetration tests that can be correlated to the cyclic shear resistance of the in situ soil. In situ penetration tests are also preferred because field measurements provide an economical indication of deposit variability (Seed and De Alba 1986). That is, several penetration tests will yield a better understanding of highly variable natural sediments than careful laboratory testing of a few select soil samples. For this reason alone, empirical methods based on in situ penetration tests are almost always favored for engineering assessments of liquefaction potential.

Several empirical methods have been proposed for evaluating the likelihood that a soil deposit will liquefy in an earthquake. These models, based on in situ tests, are briefly reviewed in the next section. For EPOLLS, the liquefaction resistance was evaluated from Standard Penetration Test (SPT) data using a widely accepted empirical method. The procedure for estimating the cyclic resistance ratio (CRR), a measure of a soil's liquefaction resistance, is outlined in Section 7.3. Calculation of the cyclic stress ratio (CSR) generated by an earthquake is discussed in Section 7.4. As described in the last section of this chapter, the CRR and CSR are used in the EPOLIQAN code to predict the liquefied soil thickness in each soil boring.

7.2. Selection of Empirical Method for Liquefaction Assessment.

Methods for assessing liquefaction potential reviewed in this section are based on field performance of liquefiable soil deposits in past earthquakes. These empirical methods rely on some in situ measure of the liquefaction resistance of a soil. While any of a number of in situ soil tests could be used to evaluate liquefaction potential, most of the effort to develop suitable criteria has been based on one of four test measurements: Standard Penetration Test (SPT) blowcounts, Cone Penetration Test (CPT) tip resistance, Becker Penetration Test (BPT) blowcounts, and in situ shear wave velocity.

Approach used to develop empirical methods.

Regardless of the in situ test employed, the approach for developing an empirical criteria for liquefaction assessment is basically the same. Sites subject to possible soil liquefaction in past earthquakes are studied and a database (or "catalog") is compiled of soil deposits that did or did not liquefy. For each case study:

(1) Based on the field evidence, a judgment is made as to whether or not liquefaction occurred.

(2) A representative measure of the in situ soil strength is determined.
(3) The shear stresses induced in the soil by the earthquake are estimated.

A liquefaction assessment criteria is formulated by then attempting to separate conditions (represented by normalized shear stress and strength parameters) where a soil liquefied from those conditions where no liquefaction was observed.

In compiling data on sites subjected to possible soil liquefaction, a critical consideration is how to distinguish between soil layers that did and did not liquefy during an earthquake. For nearly all of the available assessment methods, this distinction is made on the basis of surface manifestations of soil liquefaction (Seed et al. 1985; Liao et al. 1988; Fear and McRoberts 1995; Stark and Olson 1995). Liquefaction is judged to have occurred if sand boils, ground cracking, lateral ground movements, settlement or translation of structures, bearing capacity failures, or uplifting of buried pipes and tanks is observed. If no such surface evidence is observed, the site is assumed to have not liquefied. Defining "liquefaction" in this manner is consistent with the definition adopted in this study (Section 2.1) and the term "liquifailure" (Section 2.4). However, at some sites with no apparent liquefaction, deeper soils could have liquefied without producing surface evidence. Ishihara (1985) investigated the conditions where evidence of liquefaction in deeper layers is suppressed by the intact overburden soil. However, Youd and Garris (1994; 1995) have shown that Ishihara's findings are not valid for sites subject to lateral spreading.

For each case study, a representative index of the liquefaction resistance of the soil deposit is needed. The in situ penetration resistance can be used because the same factors that contribute to cyclic shear strength will increase the resistance to penetration. In addition, when surface evidence indicates liquefaction, the specific subsurface soil layer that liquefied must be identified before a representative index value can be defined. Seed and his co-workers (1985) appear to have identified the critical, liquefied deposit at each site as the soil layer with the lowest penetration resistance, and then compiled the average SPT blowcount measured in this critical layer. However, re-examinations of their data set indicate that the minimum blowcount in a boring was frequently compiled (Liao and Whitman 1986; Fear and McRoberts 1995). While the lowest observed blowcount may be an erroneous or spurious data point, this approach has merit because the sublayer with minimum penetration resistance will also have the least resistance to liquefaction. In addition to using the minimum blowcount, both Liao and Whitman (1986) and Fear and McRoberts (1995) considered each boring at a site as one case study to decrease the correlation among observations in their data catalogs. Because judgment is involved, different blowcount values are inevitably compiled for the same sites in the various liquefaction catalogs.

To indicate the severity of the seismic loading imparted to a liquefiable soil, the cyclic shear stress generated by an earthquake is usually estimated from the peak horizontal accelerations at the ground surface. Most empirical liquefaction assessment methods are based on the cyclic stress ratio (CSR) defined in Section 7.4, which is calculated from the maximum horizontal surface acceleration (a_{max}) generated by an earthquake at a given site. Values of CSR in the available liquefaction catalogs are often based on fairly approximate values of a_{max} estimated from empirical attenuation equations (Liao and Whitman 1986; Ambraseys 1988).

Finally, the accumulation of excess pore pressures and shear strains is affected by static shear stresses in a slope that should be considered in a liquefaction assessment (Seed and Harder 1990). However, most liquefaction assessment models were developed from case studies of fairly level ground where the static shear stresses are very small. In using these methods to evaluate liquefaction potential at sites with gentle slopes, *"the effects of the initial sustained shear stress on the triggering of liquefaction are considered negligible and ignored for all practical purposes"* (Ishihara 1993). Hence, the static shear stress imparted by a mild surface slope in a lateral spread can be ignored in predicting soil liquefaction.

Methods based on the Standard Penetration Test.

The most comprehensive liquefaction data catalogs are based on Standard Penetration Test (SPT) blowcounts (N_{SPT}). Starting in the 1970's, H. B. Seed and his colleagues worked to develop a reliable method for assessing liquefaction potential based on SPT data. Their framework for SPT-based assessments of liquefaction potential was developed in a series of papers that includes Seed and Idriss (1971), Seed et al. (1977), Seed (1979), Seed and Idriss (1981; 1982), and Seed et al. (1983). Significant contributions were also suggested in the work of Tokimatsu and Yoshimi (1983). This research culminated in the liquefaction criteria published by Seed et al. (1985).

The empirical chart published by Seed et al. (1985) is based on a standardized SPT blowcount, $(N_1)_{60}$, and the cyclic stress ratio (CSR). To get $(N_1)_{60}$, the measured N_{SPT} is corrected for the energy delivered by different hammer systems and normalized with respect to overburden stress. Boundary curves separating liquefied from unliquefied soils, in terms of CSR and $(N_1)_{60}$, were conservatively drawn to encompass nearly all observed cases of liquefaction in the data catalog. Three separate boundary curves were presented for clean to silty sands. To consider the effects of earthquake magnitude on the duration of strong shaking, magnitude scaling factors were specified. Over the last decade, the empirical method given by Seed et al. (1985), sometimes referred to as the "simplified procedure", has been widely used for evaluating potential soil liquefaction in North America and around the world.

Recently, Fear and McRoberts (1995) carefully re-examined this liquefaction database and found that, while fines content affects the liquefaction resistance of a soil, this effect may be less pronounced than indicated by Seed et al. (1985). The analysis by Fear and McRoberts also suggests a transition zone, instead of a single boundary line, for separating conditions leading to severe liquefaction damage from those with no apparent liquefaction. Overall, Fear and McRoberts (1995) conclude that the liquefaction criteria established by Seed et al. (1985) will, as intended, conservatively predict liquefaction in some cases where no liquefaction damage would be observed.

In the empirical charts proposed by Seed and his colleagues, boundary lines between liquefied and unliquefied conditions were drawn subjectively. Several researchers have suggested using statistical analyses to construct these empirical lines more objectively. Liao et al. (1988) performed a statistical regression analysis to systematically develop a liquefaction criteria in terms of CSR and $(N_1)_{60}$. For a liquefaction probability of 0.18, the model proposed by Liao et al. (1988) agrees fairly well with the boundary line drawn by Seed et al. (1985). However, Liao and his colleagues found that their data does not support using fines content as a continuous variable; instead, they developed two models, one for clean sands and a second for silty sands.

Many of the empirical methods for liquefaction resistance rely on magnitude scaling factors published by Seed et al. (1983). These magnitude scaling factors were developed from cyclic laboratory test data and are based on a representative number of uniform load cycles in different magnitude earthquakes. Ambraseys (1988) points out that the idea of an equivalent, uniform stress cycle may oversimplify differences in ground motions from different magnitude earthquakes. Using essentially the same data as Seed et al. (1985), Ambraseys developed an empirical liquefaction criteria expressed directly in terms of CSR, $(N_1)_{60}$, and earthquake magnitude (M_w). Significantly, the analysis by Ambraseys indicates that Seed's magnitude scaling factors poorly represent the field data on liquefaction. Using a larger data catalog, Loertscher and Youd (1994) found that the magnitude scaling factors used by Seed et al. (1983) are significantly conservative for moderate-sized (M = 5 to 7) earthquakes.

Other researchers have developed SPT-based empirical methods that directly use the earthquake magnitude and source distance to represent the seismic energy imparted to the soil, instead of the cyclic stress ratio (based on a_{max}) and magnitude scaling factor. Two models of this type, which also use corrected $(N_1)_{60}$ values, are given by Liao et al. (1988) and Law et al. (1990). Because estimates of peak surface accelerations are not required, empirical correlations based on earthquake magnitude and distance are easier to use in a liquefaction assessment. However, because the attenuation of seismic energy varies in different geologic regimes, these methods are not necessarily valid for geographic regions other than those represented in the data used to develop the model. Liquefaction criteria based on site-specific estimates of surface accelerations are thus more easily applied in different geographic regions (Liao et al. 1988).

Finally, liquefaction assessments in Japan are often performed using the SPT-based, empirical method specified in the Japanese bridge design code (*Specifications...* 1990). This method was developed in Japan from a large number of cyclic triaxial tests on soil samples with a known SPT penetration resistance (Ishihara 1985; 1993). Hence, this empirical method is based largely on laboratory tests, where sample disturbance is a potential issue, in contrast to the direct correlation with field behavior used in the other methods described here.

Methods based on other in situ tests.

The Cone Penetration Test (CPT) yields a continuous profile of penetration resistance and is thus well-equipped for detecting thin, liquefiable layers within a larger, stable soil deposit. Early CPT-based empirical methods for liquefaction evaluations were developed by converting SPT blowcounts in the available liquefaction case studies to equivalent CPT tip resistances. Models of this type include those proposed by Seed et al. (1983), Robertson and Campanella (1985), and Seed and De Alba (1986). In a different approach, Mitchell and Tseng (1990) used a model of cone penetration together with laboratory test data to suggest a liquefaction criteria using the CPT. However, these methods suffer from a lack of direct correlation between the measured CPT tip resistance and observed field performance of liquefiable soils in earthquakes. Using data from sites mostly in China, Shibata and Teparaksa (1988) developed a CPT-based liquefaction criteria that was based directly on field performance data. Using a more extensive database, Stark and Olson (1995) also developed an empirical method based on measured CPT tip resistances. Stark and Olson used a normalized tip resistance and drew bounding curves between liquefied and unliquefied states for clean sand, silty sand, and sandy silt.

Because large gravel particles interfere with the penetration of both the SPT sampler and the cone penetrometer, the SPT and CPT are not reliable for evaluating the liquefaction potential of gravelly soil deposits. To overcome this problem, the Becker Penetration Test (BPT) has been used for investigating the liquefaction potential of gravelly soils in North America (Harder 1996). The BPT involves driving a large diameter (168 mm recommended), closed-end casing into the ground using a double-acting diesel hammer. The number of hammer blows is typically recorded for every 30 cm of penetration. Using empirical correlations, BPT blowcounts can be converted to equivalent SPT blowcounts as discussed by Harder (1996). The equivalent-N_{SPT} values can then be used to evaluate the potential for soil liquefaction using the SPT-based methods discussed in this chapter. Unfortunately, additional research and development is needed to further standardize the BPT and improve interpretations for liquefaction susceptibility.

Many of the same factors that contribute to the liquefaction resistance of a soil deposit (density, confinement, stress history, geologic age, etc.) also influence the velocity of traveling shear waves (Finn 1991; Robertson et al. 1992). Moreover, the shear wave velocity of a soil deposit can be measured economically with surface geophysics; this is particularly advantageous in evaluating gravelly soils that are difficult to penetrate or sample. Hence, several researchers have attempted to correlate liquefaction potential with in situ shear wave velocity. Both Robertson et al. (1992) and Tokimatsu et al. (1994) present correlations, in terms of a normalized shear wave velocity and cyclic stress ratio, that were developed directly from a limited number of field cases. Other methods for evaluating liquefaction potential based on shear wave velocities are reviewed by Andrus and Stokoe (1996). However, additional data is needed to validate and improve the proposed correlations between shear wave velocity and liquefaction resistance.

Selection of empirical method for EPOLLS.

For the EPOLLS database and model, the Standard Penetration Test was chosen for evaluating the liquefied soil thickness. For many of the EPOLLS case studies, SPT blowcounts are the only in situ data available on the subsurface soil conditions. Cone penetration logs are available for a few sites, but in each case SPT data are also available. At two sites with gravelly soils (EPOLLS Slide Nos. 50 and 51), equivalent-N_{SPT} values are available from the converted records of Becker Penetration Tests. In addition to the lack of other in situ data for the EPOLLS sites, SPT-based empirical methods for evaluating liquefaction potential are preferred because they have been well-developed using a large, world-wide database of field case studies.

In January of 1996, the National Center for Earthquake Engineering Research (NCEER) sponsored a workshop of experts to review recent developments in the empirical analysis of liquefaction resistance. Empirical methods based on the SPT, CPT, BPT, and shear wave velocity were reviewed. A prime objective of this meeting was to reach a consensus on the liquefaction evaluation procedures recommended for engineering practice. At the time of this writing, the final proceedings from the "NCEER Workshop on Evaluation of Liquefaction Resistance" have yet to be published, although copies were obtained of the draft papers prepared for this workshop. For evaluations based on SPT data, the NCEER group recommended the basic approach given by Seed et al. (1985) with new magnitude scaling factors, adjustment of $(N_1)_{60}$ to model silty sands, better definition of some correction factors, and minor modifications to the base curve. These recommendations are detailed in the draft papers prepared by Robertson and Fear (1996), Youd and Noble (1996), Youd (1996), and Harder and Boulanger (1996). Collectively referred to here as the *NCEER method*, the workshop recommendations were followed in evaluating the liquefied soil thickness in the EPOLLS case studies. An flowchart of the procedure for evaluating liquefaction potential, using the NCEER method, is shown in Figure 7.1. Details on these computations are given in Sections 7.3 through 7.5.

Like most empirical methods for liquefaction assessment, the NCEER method is derived from SPT data acquired at sites experiencing liquefaction in past earthquakes. Because of likely densification or possible loosening of the soil deposit, the measured N_{SPT} values may be significantly different after the earthquake (Liao et al. 1988). However, these changes are too poorly understood to allow correction of the measured SPT values to reflect pre-earthquake conditions. Possible changes in the penetration resistance caused by the earthquake were thus neglected in the analysis of the EPOLLS data.

Furthermore, analysis of the boring logs for the EPOLLS database is not entirely consistent with the manner in which the NCEER liquefaction model was developed. For EPOLLS, the model is used to predict the occurrence of liquefaction at each SPT location as well as the resulting liquefied thickness. On the other hand, the NCEER method is based on N_{SPT} values representative of a liquefied soil deposit and a base curve conservatively drawn to encompass all sites with surface evidence of liquefaction. These issues are probably significant only when conditions place a soil deposit very close to the boundary between liquefied and unliquefied states. Hence, it seems reasonable to assume that the NCEER method will yield a good, consistent estimate of the liquefied thickness in the EPOLLS case studies.

7.3. Cyclic Resistance Ratio of Soil from SPT Blowcounts.

As indicated in Figure 7.1, the first step in evaluating the potential for soil liquefaction is to compute corrected values of $(N_1)_{60}$ from the measured SPT blowcounts. Robertson and Fear (1996) list the drilling and SPT procedures recommended for use with the NCEER method:

- Boring diameter of 66 to 115 mm (2.5 to 4.5 inch).
- Borehole filled with drilling mud or cased to full depth.
- Drilling method: wash boring with side discharge bit or rotary boring with side or upward discharge bit. Clean the bottom of the borehole (maximum allowable heave of 70 mm) before perform SPT. Hollow stem auger techniques are not recommended unless extreme care is taken to avoid heave and disturbance.
- Standard sampler of 51 mm (2.00 inch) outside diameter, 35 mm (1.38 inch) inside diameter, and at least 457 mm (18 inch) long. If the sampler is made to hold a liner, a liner must be in place.
- Record number of blows for each 150 mm (6 inch) of penetration. N_{SPT} is the number of blows for penetration from 150 to 450 mm (6 to 18 inch) from bottom of the borehole.

Corrected SPT blowcounts.

The measured SPT blowcount (N_{SPT}) is first normalized for the overburden stress at the depth of the test and corrected to a standardized value of $(N_1)_{60}$. Using the recommended correction factors given by Robertson and Fear (1996), the corrected SPT blowcount is calculated with:

$$(N_1)_{60} = N_{SPT} \cdot C_N \cdot C_E \cdot C_B \cdot C_S \cdot C_R \qquad (7.1)$$

The first correction factor (C_N) normalizes the measured blowcount to an equivalent value under one atmosphere of effective overburden stress:

$$C_N = \sqrt{\frac{Pa}{\sigma'_{vo}}} \leq 2.0 \qquad (7.2)$$

where σ'_{vo} is the vertical effective stress at the depth of N_{SPT} and Pa is one atmosphere of pressure (101.325 kPa) in the same units as σ'_{vo}. The maximum value of 2.0 limits C_N at depths typically less than 1.5 m. The factor C_E is used to correct the measured SPT blowcount for the level of energy delivered by the SPT hammer. Using 60% of the theoretical maximum energy as a standard, this correction is given by:

$$C_E = \frac{\textit{actual energy delivered to top of drill rod}}{0.60 * \textit{theoretical maximum SPT hammer energy}} = \frac{ER}{60.} \qquad (7.3)$$

where ER is the energy ratio and the theoretical maximum SPT hammer energy is 4200 lb-in (from 140 weight dropping 30 inches in each blow). The energy ratio (ER) should be measured for the particular SPT equipment used. When such measurements are unavailable, the energy ratio and correction factor can be estimated from the average values given by Seed et al. (1985):

Country:	Hammer Type:	Hammer Release:	ER	C_E
United States	Safety	Rope and pulley	60	1.00
United States	Donut	Rope and pulley	45	0.75
Japan	Donut	Rope and pulley, special throw release	67	1.12
Japan	Donut	Free fall	78	1.30

The third correction factor in Equation 7.1, C_B, is for borehole diameters outside the recommended range. The following values are recommended (Robertson and Fear 1996):

Diameter of Borehole:	C_B
65 to 115 mm (2.5 to 4.5 inch)	1.00
150 mm (6 inch)	1.05
200 mm (8 inch)	1.15

The fourth correction factor in Equation 7.1, C_S, is for SPT samplers used without a sample liner. If the split spoon sampler is made to hold a liner but is used without one, the measured blowcount should be corrected with C_S=1.2. Otherwise, C_S=1.0 for a standard sampler.

The last correction factor in Equation 7.1 is C_R, which is used to correct for the loss of energy through reflection in short lengths of drill rod. In the NCEER recommendations, values of the correction factor C_R are given for ranges of rod length. For the analysis of the EPOLLS case studies, these recommended values of C_R were approximated with a linear equation:

- ***For z ≤ 3 m:*** $C_R = 0.75$
- ***For 3 < z < 9 m:*** $C_R = (15 + z) / 24$
- ***For z ≥ 9 m:*** $C_R = 1.0$ (7.4)

where z is the length of drill rod in meters (approximately equal to the depth of N_{SPT}). This approximation is shown in Figure 7.2 together with the values recommended in Robertson and Fear (1996).

Following the flowchart in Figure 7.1, the next step is to compute a clean-sand equivalent value of $(N_1)_{60}$. To do this, an adjustment is made based on the fines content of the soil sample:

$$\textit{clean-sand equivalent } (N_1)_{60} = (N_1)_{60} + \Delta(N_1)_{60} \tag{7.5}$$

The correction factor $\Delta(N_1)_{60}$ is shown in Figure 7.3 and computed with the linear function:

- ***For FC ≤ 5%:*** $\Delta(N_1)_{60} = 0.0$
- ***For 5 < FC < 35%:*** $\Delta(N_1)_{60} = 7*(FC - 5) / 30$
- ***For FC ≥ 35%:*** $\Delta(N_1)_{60} = 7.0$ *(7.6)*

where FC is the fines content (percent finer than 0.075 mm).

Cyclic resistance ratio.

The next step in the liquefaction analysis procedure is to find the cyclic resistance ratio (CRR) for the soil based on the computed clean-sand equivalent $(N_1)_{60}$. This is done using the empirical base curve drawn from the liquefaction catalog for a magnitude 7.5 earthquake. The base curve in the recommended NCEER method, as shown in Robertson and Fear (1996), was drawn manually and modified slightly from the curve given by Seed et al. (1985). To perform the liquefaction analysis with a computer code, a mathematical expression is needed for the NCEER base curve. By assuming a functional form and then successively adjusting the coefficients, a mathematical expression for the NCEER base curve was found and implemented in the EPOLIQAN code. Providing a good approximation for $(N_1)_{60} \leq 30$, this equation is:

$$100 \cdot CRR_{M=7.5} = \frac{95}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{1.3} - \frac{1}{2} \tag{7.7}$$

where $CRR_{M=7.5}$ is the cyclic resistance ratio for a M_w=7.5 earthquake and $(N_1)_{60}$ is the corrected, clean-sand equivalent SPT value. A value of $(N_1)_{60}>30$ indicates an unliquefiable soil with an infinite CRR. Equation 7.7 is plotted in Figure 7.4 together with points taken from the NCEER base curve shown in Robertson and Fear (1996).

Next, the value of $CRR_{M=7.5}$ must be adjusted for the magnitude of the earthquake under consideration. This is done with a magnitude scaling factor (MSF):

$$CRR = CRR_{M=7.5} * MSF \tag{7.8}$$

where CRR is the cyclic resistance ratio of the soil for an earthquake magnitude corresponding to MSF. Magnitude scaling factors were originally developed by assuming that the main effect of different magnitude earthquakes on liquefaction resistance is the number of significant stress cycles generated (Seed et al. 1983). As discussed by Youd and Noble (1996), more recent research has produced MSF values that more accurately represent the effect of magnitude on liquefaction resistance. The NCEER group recommends using the magnitude scaling factors plotted in Figure 7.5, although a fairly wide range of possible values is indicated for earthquakes of M_w<7.5. For analysis of the EPOLLS case studies, MSF values from the middle of this range were used as shown in Figure 7.5. In equation form, the magnitude scaling factors (MSF) used in the EPOLIQAN code are given by:

- ***For $M_w < 7.0$:*** $MSF = 10^{3.00} * M_w^{-3.46}$
- ***For $M_w \geq 7.0$:*** $MSF = 10^{2.24} * M_w^{-2.56}$ *(7.9)*

where M_w is the moment magnitude of the earthquake.

For liquefiable soil deposits subject to significant overburden or static shear stresses, the calculated CRR should be multiplied by two additional correction factors, $K\sigma$ and $K\alpha$. These correction factors account for the effect of stresses that are significantly higher than found in the cataloged field cases used to develop the liquefaction assessment model. Recommended values of $K\sigma$ and $K\alpha$, which are based on cyclic laboratory tests, are discussed by Harder and Boulanger (1996). The $K\sigma$ factor is correlated to the effective confining pressure while $K\alpha$ is based on the static horizontal shear stress, vertical effective stress, and relative density of the soil. However, additional research is needed to more reliably define these two correction factors. More significantly, the stress conditions in a typical lateral spread are consistent with the field data used in developing the liquefaction assessment criteria (sites with a fairly level ground surface and shallow liquefiable deposits). Therefore, the two correction factors $K\sigma$ and $K\alpha$ were not used in the analysis of the EPOLLS case studies.

Soils with plastic fines.

In general, soils with a significant plasticity are not susceptible to liquefaction, but some clayey soils in China have been observed to liquefy in past earthquakes. Based on the Chinese findings, Seed and Idriss (1981; 1982; Seed et al. 1983) recommended that soils with significant plastic fines should be evaluated for possible liquefaction based on the Atterberg limits. Additional work undertaken by the U.S. Army Corps of Engineers suggested modification of the Chinese criteria to account for uncertainty in the measured index properties. The resulting criteria, as reported by Finn (1991) and Finn et al. (1994), can be summarized as:

- percent finer than 0.005 mm ≤ 20%
- (Liquid Limit + 1%) ≤ 35%
- (water content + 2%) ≥ 0.9 * (Liquid Limit + 1%)
- Liquidity Index (based on Liquid Limit + 1% and water content + 2%) ≤ 0.75

A soil with plastic fines should be considered vulnerable to significant loss of strength or liquefaction in an earthquake if the measured index properties fall within these bounds.

7.4. Cyclic Stress Ratio Induced by Earthquake.

Once the liquefaction resistance is known at a certain depth, the average cyclic shear stress generated by an earthquake must be estimated as indicated on the right side of Figure 7.1. The representative horizontal shear stress is computed with a simplified equation suggested by Seed and Idriss (1971) and expressed in terms of the cyclic stress ratio (CSR) given by Seed et al. (1983; 1985):

$$CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma'_{vo}} r_d \qquad (7.10)$$

where g is the acceleration due to gravity (9.81 m/s^2), σ_{vo} is the total vertical overburden stress, and σ'_{vo} is the effective vertical overburden stress at the depth of interest. Because free water does not transmit shear stresses, standing water above the ground surface is not included in the calculation of σ_{vo} (Tokimatsu and Yoshimi 1983). However, hydrostatic pressures from water above the ground surface is included in calculating σ'_{vo}. The maximum acceleration (a_{max}) is the peak horizontal acceleration that would occur at the ground surface in the absence of excess pore pressures or liquefaction generated by the earthquake, consistent with the definition given earlier in Section 6.4. In the absence of ground motion records, the preferred method for estimating a_{max} is to use empirical attenuation equations developed for sites with similar soil stratigraphy (Youd 1996). For example, the empirical correlations given in Equations 6.3 and 6.4 are appropriate for making these estimates.

The last parameter in Equation 7.10 is the stress reduction factor (r_d), which accounts for soil flexibility as a function of depth. Using different soil profiles with sand in the upper 15 m and a variety of earthquake motions, Seed and Idriss (1971) calculated a range for the stress reduction factor at depths up to 30 m. Seed and Idriss concluded that using an average within the range of these values, as shown in Figure 7.6, would produce an error of less than 5% for depths less than 12 m. Several researchers have used simple linear equations to approximate these average r_d values, including Tokimatsu and Yoshimi (1983), Liao and Whitman (1986), and Kayen et al. (1992). These linear approximations, overlaid with the values of r_d given by Seed and Idriss, are shown in Figures 7.6a-c. While each of these linear equations probably do not introduce significant error, a higher order polynomial can give a better representation, over the full depth to 30 m, of the average r_d values given by Seed and Idriss (1971). For this study, a quartic polynomial was derived for depths up to 30 m:

$$r_d = 1.0 + 1.6*10^{-6}\left(z^4 - 42z^3 + 105z^2 - 4200z\right) \qquad (7.11)$$

where z is the depth below the ground surface in meters. Equation 7.11, used in the EPOLIQAN code and plotted in Figure 7.6, closely matches the average values suggested by Seed and Idriss to depths of 12 m.

7.5. Prediction of Liquefied Thickness.

As indicated in Figure 7.1, the last step in the liquefaction analysis for EPOLLS is to compute the factor of safety at each SPT location and the liquefied thickness. If the computed cyclic resistance ratio (CRR) of the soil is less than or equal to cyclic stress ratio (CSR) generated by the earthquake, liquefaction is assumed to occur at that location. The factor of safety against liquefaction (FS_{liq}) is defined with (Ishihara 1985; 1993; Seed and Harder 1990):

$$FS_{liq} = \frac{CRR}{CSR} \tag{7.12}$$

That is, the soil at the depth of the measured SPT blowcount is predicted to liquefy if $FS_{liq} \leq 1.0$, while $FS_{liq} > 1.0$ indicates no liquefaction. When the clean-sand equivalent $(N_1)_{60}$ is greater than 30, the soil is considered to be unliquefiable (corresponding to the vertical asymptote of the base curve in Figure 7.4). In this case, as well as for unsaturated soils and those with sufficient plasticity to be considered unliquefiable, the CRR is infinite and the FS_{liq} is assigned a value of -1.0 in the EPOLIQAN code.

As discussed in Section 6.7, a continuous soil profile is needed to estimate the liquefied thickness and other EPOLLS variables. That is, to find the boundaries between liquefied and unliquefied soil, the data from a soil boring must be interpolated between the depths of the Standard Penetration Tests to get a continuous profile. Because SPT blowcounts are known to vary with overburden stress, the normalized $(N_1)_{60}$ values should be used for this interpolation. As shown by Equation 7.7, the cyclic resistance ratio (CRR) is obtained directly from the clean-sand equivalent $(N_1)_{60}$. Hence, in a completely uniform deposit, measured N_{SPT} values would vary with depth but the corresponding $(N_1)_{60}$ and CRR values would be constant. Therefore, the continuous profile needed to predict the liquefied thickness can be based on an interpolation of CRR values at the SPT locations.

In the EPOLIQAN code, the CRR values are first determined at each SPT location. These values are then linearly interpolated to give a continuous profile of CRR values at 0.1 m depth increments. However, the CRR values are not interpolated across the interface between two soil deposits. Instead, the CRR determined nearest the top of a given soil unit is held constant to the upper boundary of the same deposit, and the deepest CRR is held constant to the bottom boundary. Interpolation of the CRR profile according to these rules is depicted in Figure 7.7. The cyclic stress ratio (CSR) is also computed at the same 0.1 m depth increments allowing the calculation of a continuous profile of FS_{liq}. The thickness of liquefied soil is then determined by simply identifying the thickness of the soil profile for which $FS_{liq} \leq 1.0$.

Table 7.1. Estimated susceptibility of sedimentary deposits to liquefaction during strong seismic shaking (from Youd and Perkins 1978).

Type of deposit	General distribution of cohesionless sediments in deposits	Likelihood that cohesionless sediments, *when saturated*, would be susceptible to liquefaction (by age of deposit)			
		< 500 year	Holocene	Pleistocene	Pre-pleistocene
Continental Deposits					
River channel	Locally variable	Very high	High	Low	Very low
Flood plain	Locally variable	High	Moderate	Low	Very low
Alluvial fan and plain	Widespread	Moderate	Low	Low	Very low
Marine terraces and plains	Widespread	-----	Low	Very low	Very low
Delta and fan-delta	Widespread	High	Moderate	Low	Very low
Lacustrine and playa	Variable	High	Moderate	Low	Very low
Colluvium	Variable	High	Moderate	Low	Very low
Talus	Widespread	Low	Low	Very low	Very low
Dunes	Widespread	High	Moderate	Low	Very low
Loess	Variable	High	High	High	Unknown
Glacial till	Variable	Low	Low	Very low	Very low
Tuff	Rare	Low	Low	Very low	Very low
Tephra	Widespread	High	High	?	?
Residual soils	Rare	Low	Low	Very low	Very low
Sebka	Locally variable	High	Moderate	Low	Very low
Coastal Zone					
Delta	Widespread	Very high	High	Low	Very low
Esturine	Locally variable	High	Moderate	Low	Very low
Beach: high wave energy	Widespread	Moderate	Low	Very low	Very low
Beach: low wave energy	Widespread	High	Moderate	Low	Very low
Lagoonal	Locally variable	High	Moderate	Low	Very low
Fore shore	Locally variable	High	Moderate	Low	Very low
Artificial					
Uncompacted fill	Variable	Very high	-----	-----	-----
Compacted fill	Variable	Low	-----	-----	-----

Note: When the water table is deeper than 10 m, the likelihood of liquefaction in most deposits is low.

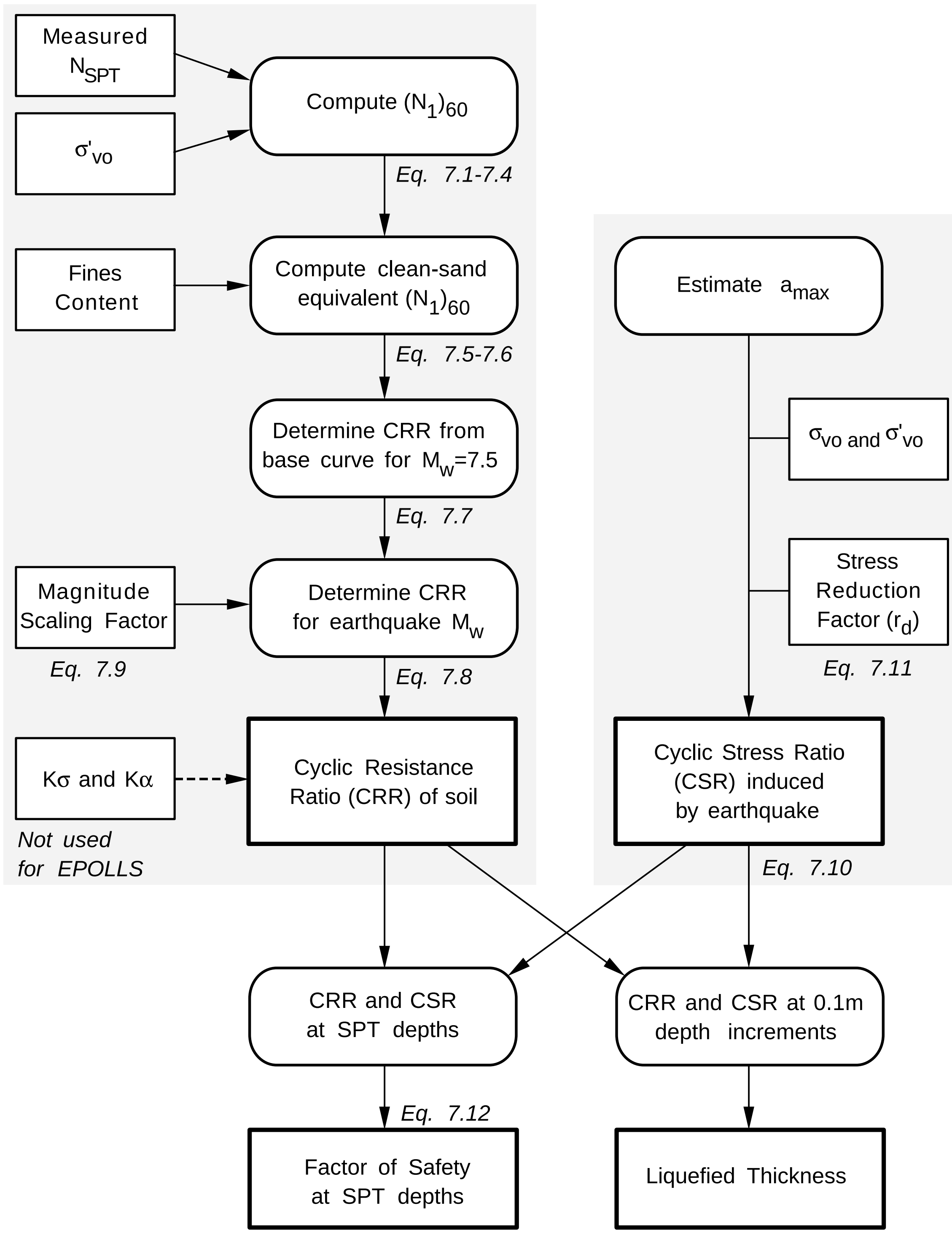

Figure 7.1. Flowchart for evaluating the liquefied thickness of soil based on SPT blowcounts.

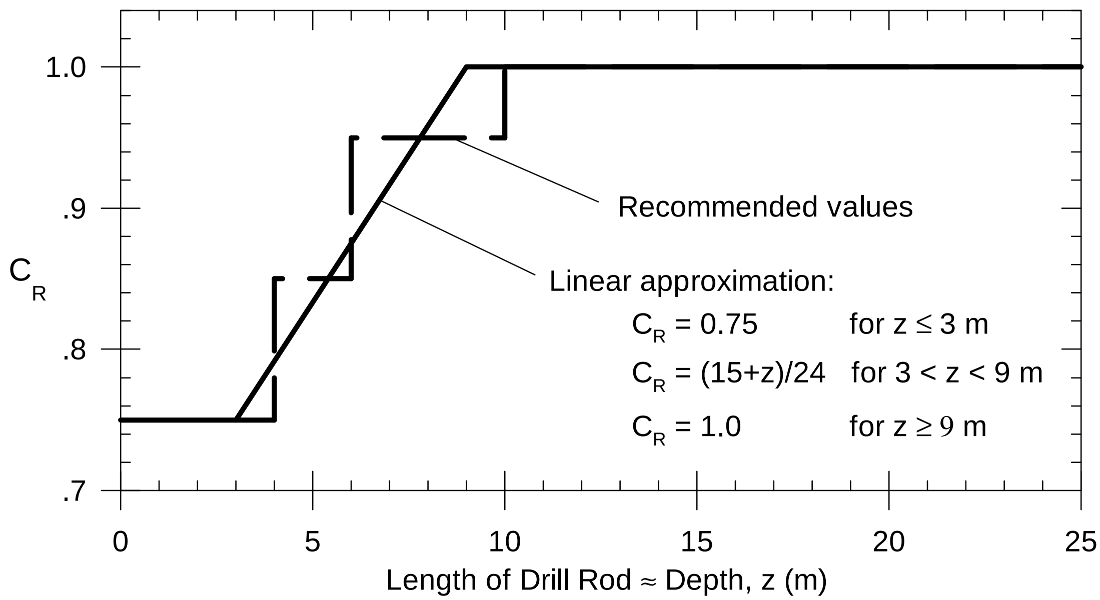

Figure 7.2. Correction factor (C_R) for length of drill rod.

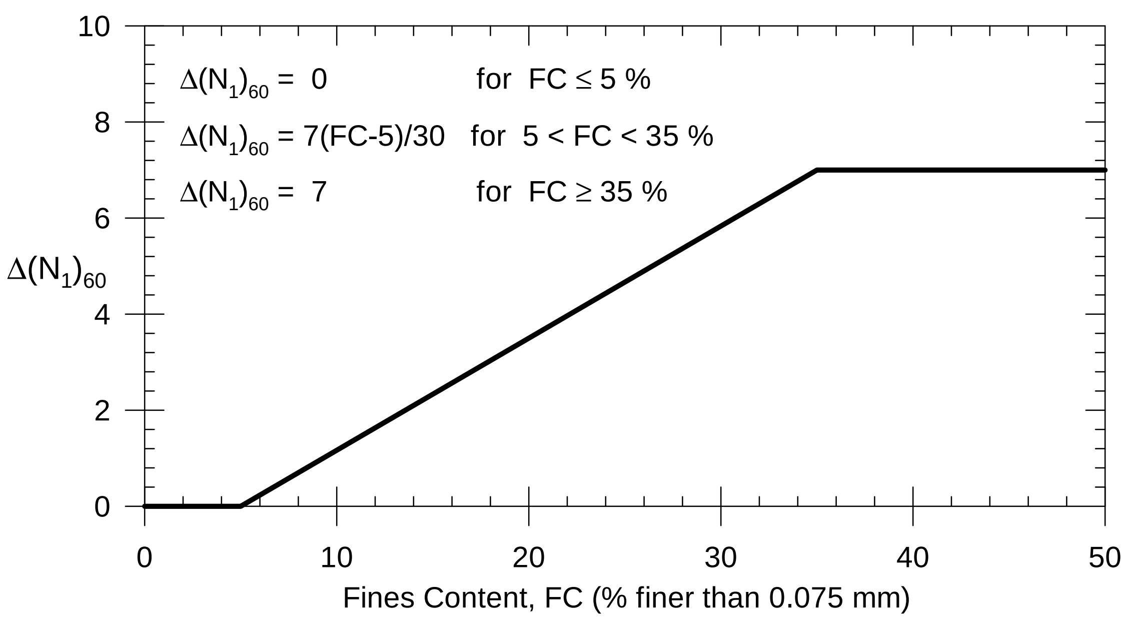

Figure 7.3. Correction factor $\Delta(N_1)_{60}$ for fines content.

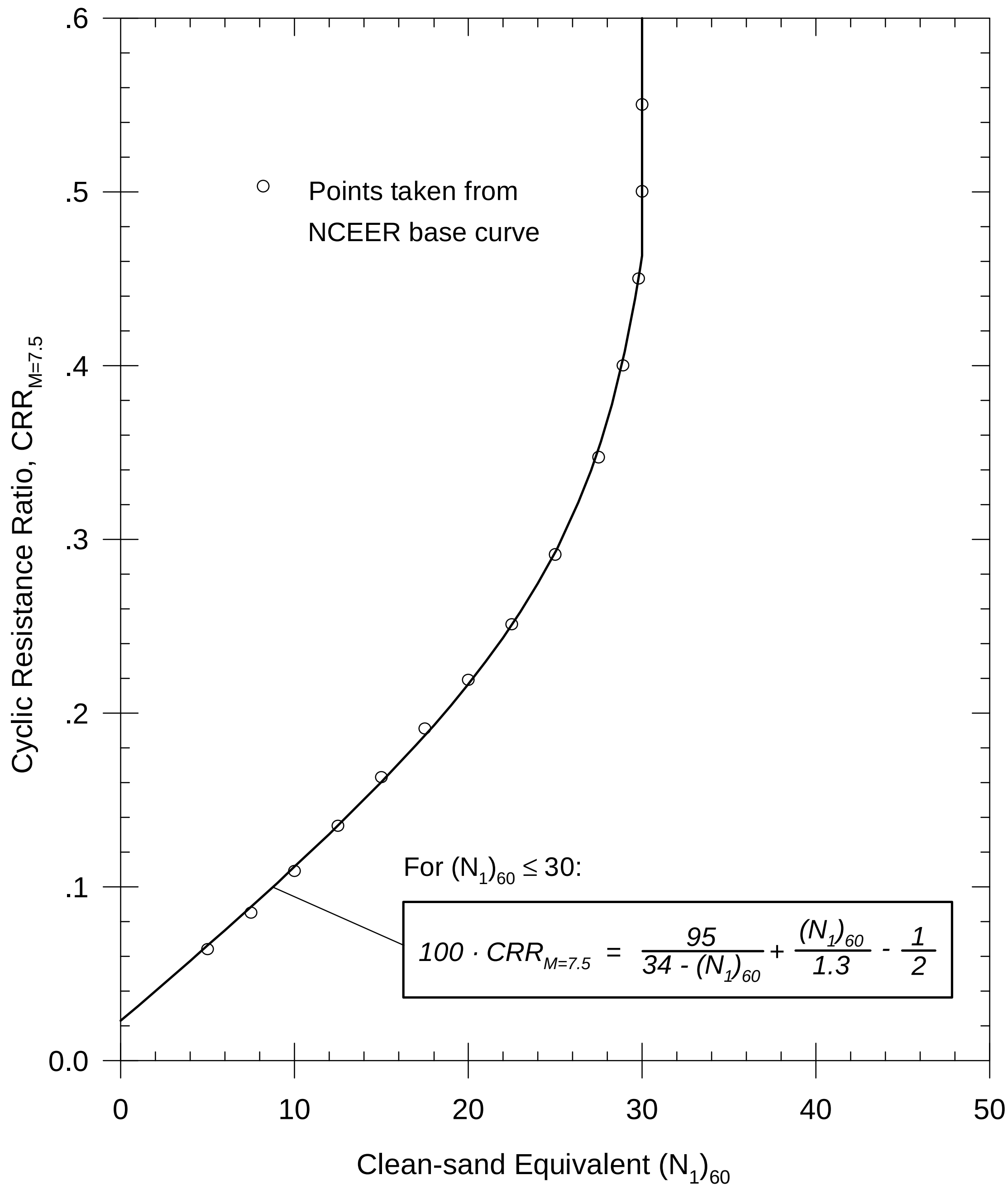

Figure 7.4. Base curve for getting $CRR_{M=7.5}$ from corrected SPT blowcount.

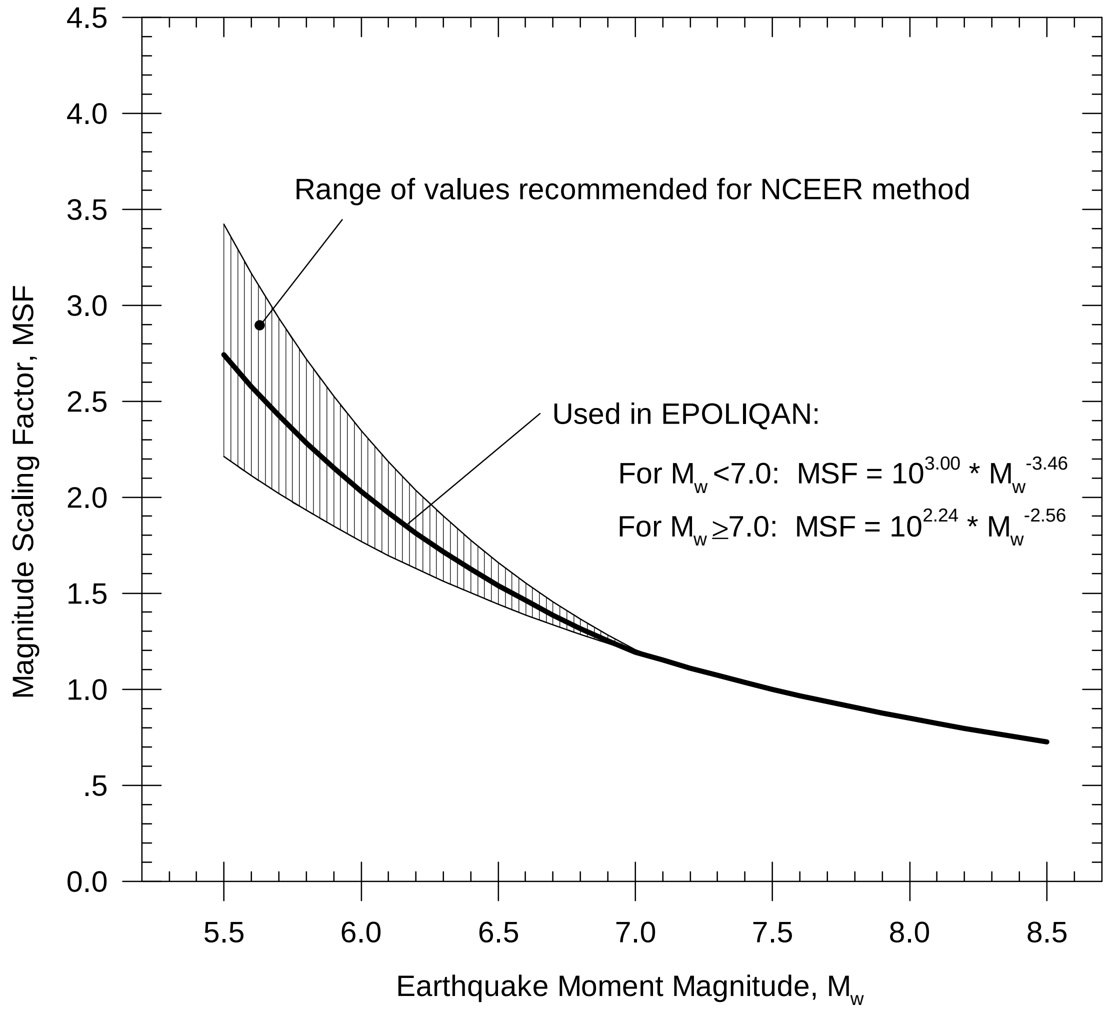

Figure 7.5. Magnitude scaling factors for computing CRR.

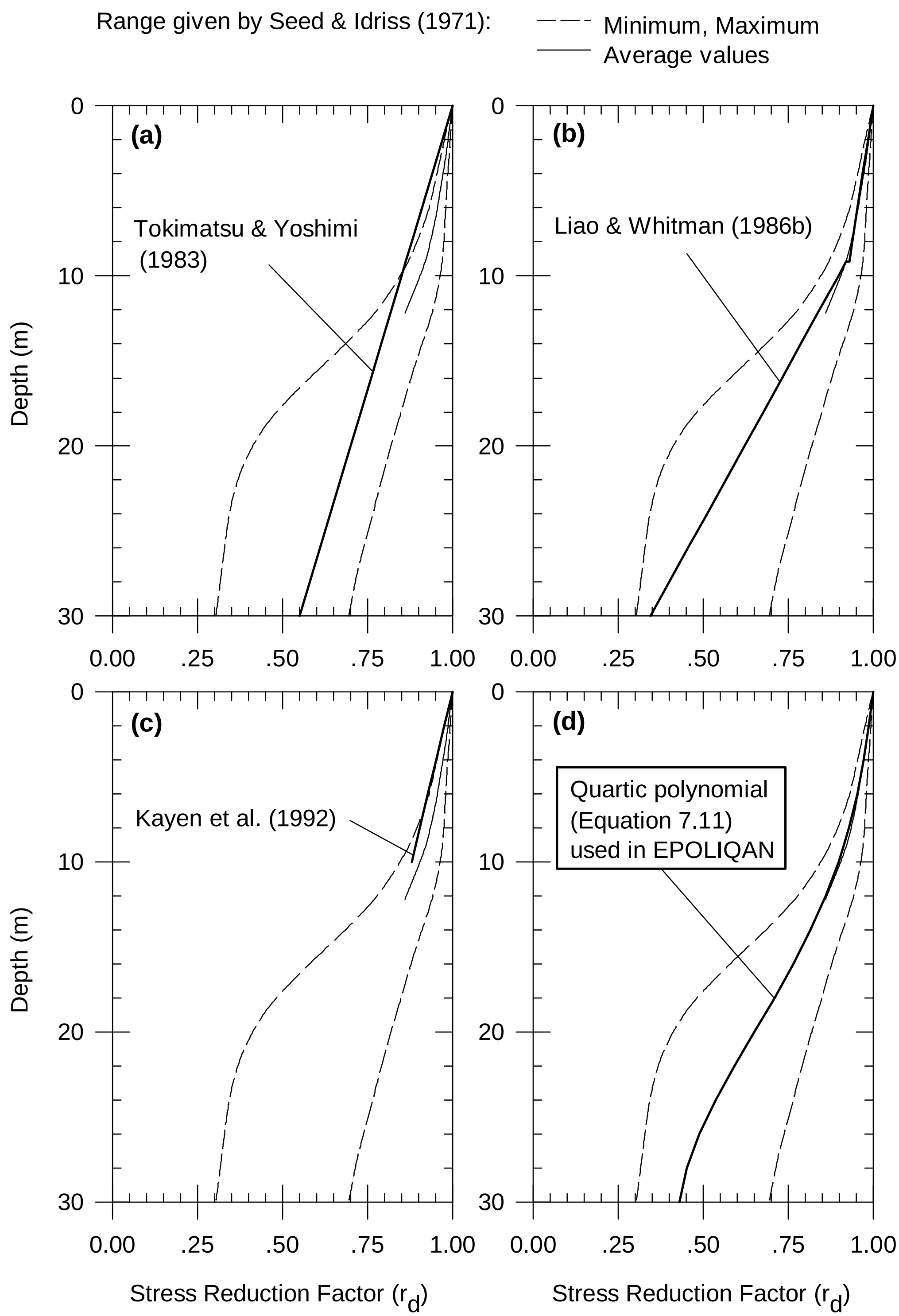

Figure 7.6. Approximations for the stress reduction factor (r_d) used in computing CSR.

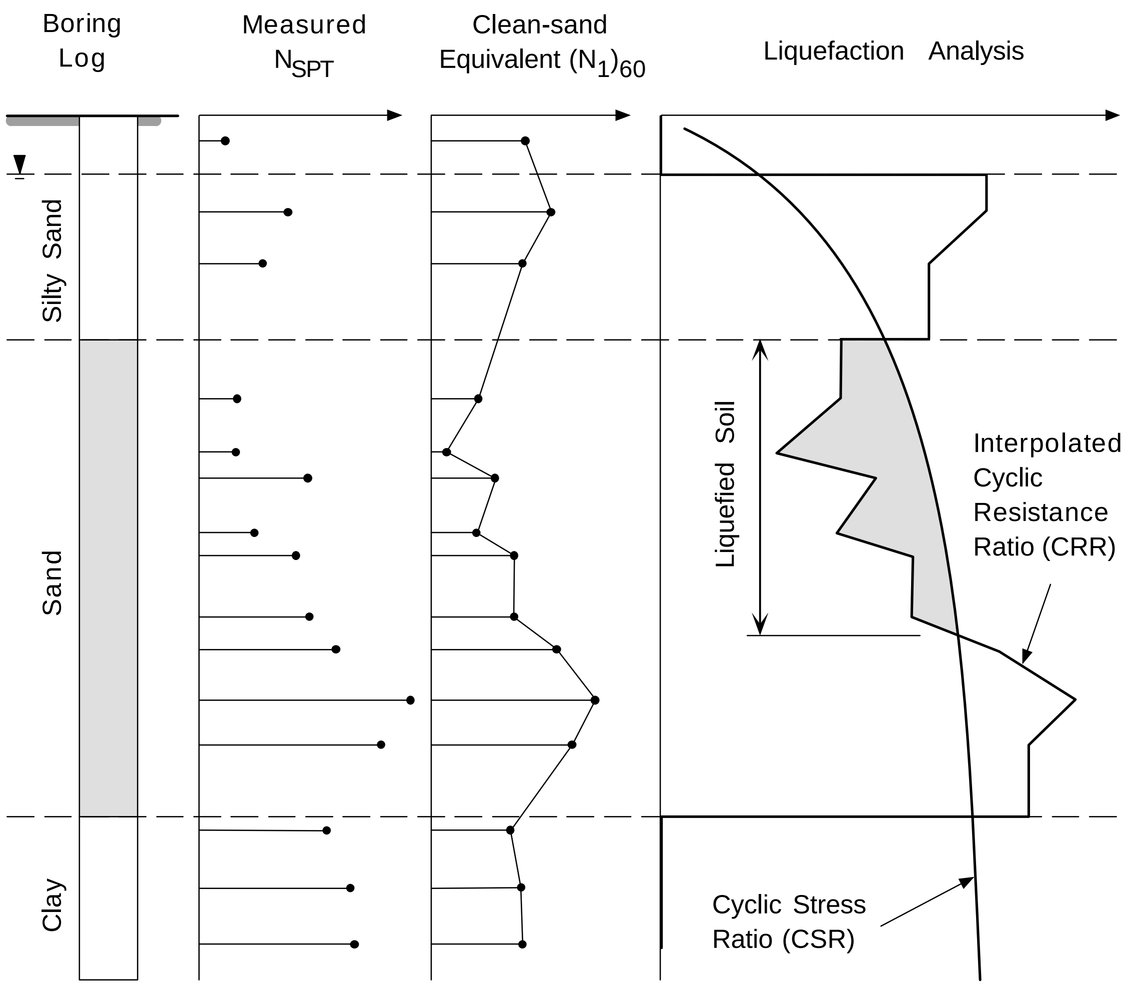

Figure 7.7. Interpolation of SPT data in EPOLIQAN to predict the liquefied thickness.