CHAPTER 3 LITERATURE REVIEW ON LIQUEFACTION ANALYSIS OF GROUND REINFORCEMENT SYSTEM

3.1 The Simplified Procedure for Liquefaction Evaluation

The Simplified Procedure was first proposed by Seed and Idriss (1971). The procedure has evolved over time and still been used worldwide to analyze liquefaction resistance of soil. The following section discusses the Simplified Procedure as proposed by the 1996 NCEER and 1998 NCEER/NSF Workshops (Youd, et al., 2001).

Seed and Idriss (1971) considered a soil column as a rigid body. As the seismic loading is excited at the base of the soil column and the shear wave propagates to the ground surface, shear stress is generated in the soil column and can be calculated by the following equation:

$$(\tau_{\max})_r = \sigma_0 * \frac{a_{\max}}{g} \qquad (3.1)$$

where:

$(\tau_{max})_r$ = maximum shear stress for rigid body

σ_0 = total overburden pressure

a_{max} = peak horizontal acceleration on the ground surface

g = acceleration of gravity

In reality, soil behaves as a deformable body instead of as a rigid body. Hence, the rigid body shear stress should be reduced with a correction factor to give the deformable body shear stress $(\tau_{max})_d$. This correction factor is called the stress reduction coefficient (r_d) and can be computed as follows:

$$(\tau_{\max})_d = r_d * (\tau_{\max})_r \qquad (3.2)$$

where:

$(\tau_{max})_d$ = maximum shear stress for deformable body

r_d = the stress reduction coefficient

The value of the stress reduction coefficient decreases with depth as shown on Figure 3.1 with a value of unity on the ground surface. The average value of r_d can be estimated with the following equations (Liao and Whitman 1986a):

$$r_d = 1.0 - 0.00765z \quad \text{for } z \le 9.15 \text{ meters} \tag{3.3a}$$

$$r_d = 1.174 - 0.0267z \quad \text{for } 9.15 \text{ meters} < z \le 23 \text{ meters} \tag{3.3b}$$

or as suggested by Blake (1996):

$$r_d = \frac{1.0 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.0 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^2} \tag{3.4}$$

where:

z = depth of interest in meter.

Both equations essentially result in similar values as shown on Figure 3.1. Some researchers suggested applying the stress reduction coefficient for depth greater than 15 meters but the Simplified Procedure has not been verified with case histories for these depths (Youd, et al, 2001).

If equation (3.1) is substituted into equation (3.2), the maximum shear stress for deformable body $(\tau_{max})_d$ can be calculated as:

$$(\tau_{max})_d = r_d * \sigma_0 * \frac{a_{max}}{g} \tag{3.5}$$

Figure 3.2 shows an example of shear stress time history during earthquake. It is apparent that the time history shows a jagged shape. For practical purpose, a value of an equivalent average of shear stress (τ_{ave}) should be used. Seed and Idriss (1971) suggested that a value of 65% of the maximum shear stress (τ_{max}) is reasonably accurate. They based their prediction by appropriate weighting of laboratory test data. Therefore, equation (3.5) can be written in term of equivalent average of shear stress (τ_{ave}):

$$\tau_{ave} = 0.65 * \sigma_0 * \frac{a_{max}}{g} * r_d \qquad (3.6)$$

If the equivalent average of shear stress (τ_{ave}) is normalized with the initial effective overburden pressure (σ_0'), the term is called the seismic demand of a soil layer or CSR (Cyclic Stress Ratio).

$$CSR = \frac{\tau_{ave}}{\sigma_0'} = 0.65 * \frac{\sigma_0}{\sigma_0'} * \frac{a_{max}}{g} * r_d \qquad (3.7)$$

The Cyclic Stress Ratio (CSR) is only one of the variables needed in the evaluation of liquefaction resistance of soil. Cyclic Resistance Ratio (CRR) is another one. CRR expresses the capacity of the soil to resist liquefaction. The following paragraphs explain the determination of CRR based on Standard Penetration Test (SPT). The discussion is limited on SPT because the values of SPT blow counts would be used in numerical analyses in this research. Procedures are also available based on Cone Penetration Tests (CPT), shear wave velocity measurement, and Becker Penetration Test (Youd, et al., 2001).

The values of CRR can be determined by using Figure 3.3. Figure 3.3 shows correlation between $(N_1)_{60}$ and CRR for earthquake magnitude of 7.5. $(N_1)_{60}$ is the SPT blow count corrected to an effective overburden pressure of 100 kPa (1 tsf) and to a hammer energy efficiency of 60%. Both correction factors will be discussed later in this section.

Curves for cohesionless soil with fines content of 5% or less, 15%, and 35% are shown on Figure 3.3. The curve for fines content of 5% or less is called “the SPT clean sand base curve”. Rauch (1998) suggested that this curve could be approximated by the following equation:

$$CRR_{M=7.5} = \frac{1}{34-(N_1)_{60cs}} + \frac{(N_1)_{60cs}}{135} + \frac{50}{[10*(N_1)_{60cs}+45]^2} - \frac{1}{200} \qquad (3.8)$$

where:

$(N_1)_{60cs}$ = equivalent clean sand value of $(N_1)_{60}$

All curves on Figure 3.3 were drawn using equation (3.8). The curves for fines content ≤ 5% and 35% are congruent with the original curve developed by Seed and Idriss (1971). The curve for fines content of 15% falls to the right of the original curve.

The value of SPT blow counts for soil with fines content can be adjusted to the equivalent clean sand value of $(N_1)_{60}$ so that equation (3.8) can be used. This can be done by applying constants, α and β, that are functions of fines content. Hence, the effect of fines content (FC) on the value of CRR is included as

$$(N_1)_{60cs} = \alpha + \beta (N_1)_{60} \qquad (3.9)$$

where α and β can be determined as follows:

$\alpha = 0$	for FC ≤ 5%	(3.10a)
$\alpha = \exp[1.76 - (190/FC^2)]$	for 5% < FC < 35%	(3.10b)
$\alpha = 5.0$	for FC ≥ 35%	(3.10c)
$\beta = 1.0$	for FC ≤ 5%	(3.11a)
$\beta = [0.99 + (FC^{1.5}/1000)]$	for 5% < FC < 35%	(3.11b)
$\beta = 1.2$	for FC ≥ 35%	(3.11c)

As noted previously, the values of SPT blow counts should be corrected as summarized on Table 3.1. The corrected blow count $(N_1)_{60}$ should be determined as follows:

$$(N_1)_{60} = N_m C_N C_E C_B C_R C_S \qquad (3.12)$$

where:

N_m = uncorrected SPT blow count

C_N = correction factor for effective overburden pressure

C_E = correction factor for hammer energy ratio

C_B = correction factor for borehole diameter

C_R = correction factor for rod length

C_S = correction factor for samplers with or without liners

The correction factor for effective overburden pressure (C_N) can be determined using either of the following equations (Liao and Whitman, 1986a):

$$C_N = \sqrt{\frac{P_a}{\sigma_0'}} \qquad (3.13a)$$

or as suggested by Kayen, et al. (1992):

$$C_N = \frac{2.2}{1.2 + \frac{\sigma_0'}{P_a}} \qquad (3.13b)$$

where:

σ_0' = effective overburden pressure

P_a = atmospheric pressure in the same unit as σ_0'

The value of C_N in equation (3.13a) should not exceed 1.7 while equation (3.13b) already limits the value of C_N to 1.7. The value of effective overburden pressure (σ_0') in both equations should be the pressure at the time SPT test is performed.

The SPT clean sand base curve can only be applied to earthquake with magnitude of 7.5. For earthquake magnitudes other than 7.5, a magnitude-scaling factor (MSF) should be applied. Youd, et al. (2001) suggested to use the following equations to determine the values of MSF. It is also shown on Table 3.2.

For earthquake magnitudes < 7.5:

$$\text{Lower bound: } MSF = \frac{10^{2.24}}{M_w^{\ 2.56}} \qquad (3.14a)$$

$$\text{Upper bound: } MSF = \left(\frac{M_w}{7.5}\right)^{-2.56} \qquad (3.14b)$$

where:

M_w = earthquake moment magnitude

The values of MSF for earthquake magnitudes larger than 7.5 can be seen on Table 3.2. These values should be used with judgment and consideration because there are only few well-documented data for liquefaction case histories for earthquake magnitude larger than 8.

As noted previously (refer to Figure 3.1), the Simplified Procedure applies for level to gently slope sites and for depths less than 15 meters. Therefore, the value of CRR should be corrected for greater depths that is for high overburden stresses. The correction factor (K_σ) is shown on Figure 3.4. It is apparent that K_σ is equal to unity for effective overburden pressure less than 1 tsf and then decreases with increasing effective overburden pressure. Figure 3.4 can also be estimated by using the following equation:

$$K_\sigma = \left(\frac{\sigma_0'}{P_a}\right)^{(f-1)} \qquad (3.15)$$

where:

f = exponent as a function of relative density, stress history, aging, and overconsolidation ratio.

f = 0.7 – 0.8 for relative densities between 40% and 60%

f = 0.6 – 0.7 for relative densities between 60% and 80%

There are correction factors for sloping ground (high static shear stress) and aging but since there is no enough data, these correction factors should be used with judgment. These aspects are still open for further research.

Hence, it can be concluded that the value of CRR for a particular earthquake magnitude can be determined as:

$$CRR = CRR_{M=7.5} * MSF * K_{\sigma} \qquad (3.16)$$

The factor of safety against liquefaction can be written as:

$$FS_L = \frac{CRR}{CSR} \qquad (3.17)$$

3.2 Baez and Martin Procedure

As noted in Chapter 2, efforts have been developed to mitigate the damage caused by liquefaction. The efforts include densification of the liquefiable soil and/or by providing drainage paths for pore water pressure dissipation. Baez and Martin (1993, 1994) proposed a design procedure that includes both criteria above. The procedure was developed for vibro-replacement stone column and vibro-concrete column and included densification, drainage, and stress concentration criteria. Of particular interest for this research is the stress concentration criterion. Explanation for both mitigation techniques can be seen in Chapter 2.

The basic assumption for their procedure is that the shear strains in the soil and in the stone column are compatible.

$$\gamma_s = \gamma_{sc} \qquad (3.18)$$

and since $\gamma = \tau/G$,

$$\frac{\tau_s}{G_s} = \frac{\tau_{sc}}{G_{sc}} \qquad (3.19)$$

where:

γ_s = shear strain in the soil matrix

γ_{sc} = shear strain in the stone column matrix

τ_s = shear stress in the soil matrix

τ_{sc} = shear stress in the stone column matrix

G_s = shear modulus of the soil matrix

G_{sc} = shear modulus of the stone column

Equilibrium also requires that the shear stress generated at a given depth be distributed to the shear stress in the soil matrix and in the stone column.

$$\tau_s A_s + \tau_{sc} A_{sc} = \tau A \qquad (3.20)$$

where:

τ = the input shear stress can be estimated using the Simplified Procedure (Seed and Idriss, 1971) using equation (3.6)

A = total plan area = $A_s + A_{sc}$

A_s = plan area of the soil matrix

A_{sc} = plan area of the stone column

The ratio between the area of stone column and the total plan area can be written as:

$$A_r = \frac{A_{sc}}{A} \qquad (3.21)$$

and the ratio between the shear modulus of the stone column and the shear modulus of the soil can be written as:

$$G_r = \frac{G_{sc}}{G_s} \qquad (3.22)$$

Therefore, using equation (3.22), equation (3.19) can be written as

$$\tau_s = \frac{\tau_{sc}}{G_r} \qquad (3.23)$$

By using equations (3.19), (3.20), (3.21), (3.22), and (3.23), the shear stress in the soil can be determined as follows:

$$\frac{\tau_s}{\tau} = \frac{\tau_s}{0.65\sigma_0 \frac{a_{\max}}{g} r_d} = \frac{1}{1 + A_r\left(G_r - 1\right)} \qquad (3.24)$$

Baez and Martin (1993, 1994) defined the ratio expressed in equation (3.24) as the shear stress reduction factor (K_G). This factor should be introduced because the Simplified Procedure does not take into account inclusion of any reinforcing elements. It is obvious that if there are no reinforcing elements installed in the ground ($G_r = 1$), the value of K_G will be equal to unity and the shear stress in the soil will be equal to that suggested by equation (3.6).

Equation (3.24) can be applied to develop a chart showing K_G as a function of area replacement ratio (A_r) and the shear modulus ratio (G_r) as shown on Figure 3.5.

3.3 Goughnour and Pestana Procedure

Goughnour and Pestana (1998) modified the procedure proposed by Baez and Martin (1993, 1994) by including the effect of slenderness ratio of the stone column and the vertical stress ratio. The slenderness ratio is defined as the ratio of the height to the width of the stone column and the vertical stress ratio is the ratio of overburden pressure within the stone column to the overburden pressure within the soil matrix.

They argued that the column might experience bending which is likely caused by the large slenderness ratio of the column. They derived equations by taking into account the effect of the slenderness ratio of the stone column and suggested that the equivalent shear modulus of the stone column with taking into account the effect of the slenderness ratio can be determined using:

$$G_{scm} = \left(\frac{\pi d_{sc}}{2\lambda}\right)^2 E_{sc} \qquad (3.25)$$

where:

d_{sc} = the diameter of the stone column

λ = the wave length

E = the elastic modulus of the stone column

The elastic modulus of stone column can be estimated using the value of the shear modulus of the stone column:

$$E_{sc} = 2(1+\nu)G_{sc} \qquad (3.26)$$

where:

ν = Poisson's ratio

By substituting equation (3.26) into equation (3.25), the following equation should be obtained:

$$G_{scm} = \frac{1}{2} G_{sc}(1+\nu)\left(\frac{\pi d_{sc}}{\lambda}\right)^2 \qquad (3.27)$$

where all terms have been explained previously.

It can be seen that the value of G_{scm} is equal to or larger than the value of G_{sc} for a value of d_{sc}/λ equals to or larger than 0.4. In practice, these values of d_{sc}/λ are unlikely to be found. Therefore, the value of G_{scm} might be smaller than the values of G_{sc} and G_s. It means the stone column is more flexible than the soil. Goughnour and Pestana (1998) argued that the shear stress in the soil would actually increase because the stone column will move together with the soil. This may sound contradictory but it can be explained since the shear stress on the interface between the soil matrix and the stone column is assumed to be negligible. It can be concluded that the presence of stone column fail to reduce the shear stress perceived by the soil matrix. This is the case for typical slenderness ratio used in practice.

Goughnour and Pestana (1998) also suggested the use of the vertical stress ratio (n) that is the ratio of the effective overburden pressure within the stone column to the effective overburden pressure within the soil matrix.

$$n = \frac{\sigma_0'_{sc}}{\sigma_0'_s} \qquad (3.28)$$

The value of $\sigma_0'_s$ can be estimated using equation proposed by Goughnour and Jones (1989):

$$\sigma_0'_s = \frac{(\sigma_0'_s)_{ave}}{1 + A_r(n-1)} \qquad (3.29)$$

where:

$(\sigma_0'_s)_{ave}$ = the average overburden pressure in the soil matrix.

The typical values of n vary between 4 and 10 based on model tests and 2 to >10 based on field measurement. Barksdale and Bachus (1989) suggested a different approach to calculate the vertical stress ratio (n).

$$n = \frac{\sigma_0'_{sc}}{\sigma_0'_s} = G_r \frac{\left[\frac{1-\nu}{1-2\nu}\right]_{sc}}{\left[\frac{1-\nu}{1-2\nu}\right]_s} \qquad (3.30)$$

where G_r is from equation (3.22).

As a conclusion of their study, Goughnour and Pestana (1998) suggested the use of the following value of shear stress reduction factor (K_G) instead of the one suggested by Baez and Martin (1993, 1994) in equation (3.24):

$$K_G = \frac{\tau_s}{\tau} = \frac{1 + A_r(n-1)}{1 + A_r(G_r - 1)} \qquad (3.31)$$

Note that the only new terms introduced here is the nominator, $1+A_r(n-1)$, instead of unity as suggested by Baez and Martin (1993, 1994).

3.4 Calculation Example

An example is given in this section to describe the use of the three procedures explained in the previous sections.

Suppose there is a soil profile with clean sand at a depth of 5 feet as shown on Figure 3.6. Later on, a stone column is installed for this site. The parameters for the soil, the earthquake, and the stone column are as follows:

Soil parameters:

Saturated unit weight (γ_{sat}) = 120 pcf

Corrected SPT blow counts [$(N_1)_{60}$] = 10 blows/feet

Shear modulus (G_s) = 100 MPa = 2,100,000 psf

Poisson's ratio = 0.3

Earthquake parameters:

Earthquake magnitude = 7.5

Peak horizontal acceleration on the ground surface (a_{max}) = 0.45g

Stone column parameters:

Diameter (d_{sc}) = 3 feet

Length = 14.5 feet (installed to depth of the rock surface)

Shear modulus (G_{sc}) = 220 MPa = 4,620,000 psf

Poisson's ratio = 0.2

3.4.1 Simplified Procedure (Seed and Idriss, 1971)

1. Determination of the stress reduction coefficient using equations (3.3) or (3.4) or Figure 3.1.

   By using equation (3.3) for depths less than 9.15 m, the value of r_d for depth of 5 feet (1.524 meters):

$$r_d = 1.0 - (0.00765 * 1.524) = 0.988$$

2. Calculation of the Cyclic Stress Ratio (CSR) using equation (3.7).

   First, the total and the effective overburden pressures have to be determined:

$$\sigma_0 = 120 * 5 = 600 \ psf = 0.3 \, tsf$$

$$\sigma_0' = (120 - 62.4) * 5 = 288 \ psf = 0.144 \, tsf$$

   Using equation (3.7), the value of CSR can be determined:

$$CSR = \frac{\tau_{ave}}{\sigma_0'} = 0.65 * \frac{0.3}{0.144} * \frac{0.45g}{g} * 0.988 = 0.6$$

3. Calculation of the Cyclic Resistance Ratio (CRR) using equation (3.8) or Figure 3.3.

   Since, the soil is clean sand, no correction for fines content is needed and since the SPT blow count is already a corrected one, there is no need for correcting the SPT blow count. Therefore, using equation (3.8) the CRR value becomes:

$$CRR_{M=7.5} = \frac{1}{(34-10)} + \frac{10}{135} + \frac{50}{[(10*10)+45]^2} - \frac{1}{200} = 0.113$$

4. Calculation of the Magnitude Scaling Factor (MSF) using equation (3.14).

   The earthquake magnitude is 7.5. Hence, the earthquake magnitude is not necessary to be corrected using MSF.

5. Calculation of the correction for high overburden pressure (K_σ) using equation (3.15) or Figure 3.4.

   From step 2, it was obtained that the effective overburden pressure (σ_0') is less than 1 tsf. Therefore, the correction for high overburden pressure (K_σ) is equal to unity (refer to Figure 3.4).

6. Calculation of the corrected CRR using equation (3.16).

   Since the values of MSF and K_σ are equal to unity,

$$CRR = CRR_{M=7.5} = 0.113$$

7. Calculation of factor of safety against liquefaction (FS_L) using equation (3.17).

   The factor of safety against liquefaction can be calculated as

$$FS_L = \frac{0.113}{0.6} = 0.19$$

   Since the factor of safety against liquefaction is less than unity, this site is susceptible to liquefaction.

3.4.2 Baez and Martin Procedure (1993, 1994)

1. Determination of the shear modulus ratio (G_r) using equation (3.22).

   The ratio between the shear modulus of the stone column and the shear modulus of the soil can be computed as:

$$G_r = \frac{4{,}620{,}000}{2{,}100{,}000} = 2.2$$

2. Calculation of the area replacement ratio (A_r) using equation (3.21).

   Supposedly, it is required to have an area replacement ratio (A_r) = 10%, that is the area of stone column covers 10% of the total plan area.

3. Calculation of the shear stress reduction factor (K_G) using equation (3.24) or Figure (3.5).

$$K_G = \frac{1}{1+[0.1*(2.2-1)]} = 0.89$$

4. Calculation of the shear stress induced by the earthquake (τ_{ave}) using equation (3.6).

   The shear stress induced by the earthquake (τ_{ave}) or the input shear stress can be estimated using the Simplified Procedure (Seed and Idriss, 1971).

$$\tau = 0.65*600*\frac{0.45g}{g}*0.988 = 173.394\ psf$$

5. Calculation of the shear stress in the soil (τ_s) using equation (3.24).

   Therefore, the shear stress in the soil matrix (τ_s) can be determined as

$$\tau_s = K_G * \tau = 0.89*173.394 = 154\ psf$$

3.4.3 Goughnour and Pestana Procedure (1998)

1. Determination of the vertical stress ratio (n) using equation (3.30).

   The only difference of this procedure to that of Baez and Martin (1993, 1994) is in the calculation of the shear stress reduction factor (K_G). For this procedure, the value of K_G can be estimated using equation (3.31). First, the vertical stress ratio (n) has to be determined:

$$n = 2.2 \frac{\left[\frac{(1-0.2)}{1-(2*0.2)}\right]}{\left[\frac{(1-0.3)}{1-(2*0.3)}\right]} = 1.68$$

2. Calculation of the shear stress reduction factor (K_G) using equation (3.31).

   After the value of n is obtained, the value of the shear stress reduction factor (K_G) can be computed as follows:

$$K_G = \frac{1+0.1(1.68-1)}{1+0.1(2.2-1)} = 0.95$$

3. Calculation of the shear stress in the soil (τ_s) using equation (3.31).

   The shear stress induced by the earthquake (τ_{ave}) has been estimated using the Simplified Procedure (refer to Section 3.4.2 step 4). Therefore, the shear stress in the soil matrix (τ_s) can be determined as

$$\tau_s = K_G * \tau = 0.95 * 173.394 = 165 \ psf$$

It can be concluded that for this example the Goughnour and Pestana procedure gives slightly higher shear stress in the soil matrix (τ_s = 165 psf) compared to that of the Baez and Martin procedure (τ_s = 154 psf).

Table 3.1 Corrections to SPT (after Youd, et al., 2001)

Factor	Equipment variable	Term	Correction
Overburden pressure	-	C_N	$(P_a/\sigma_0')^{0.5} \leq 1.7$
Energy ratio	Donut hammer	C_E	0.5 – 1.0
	Safety hammer	C_E	0.7 – 1.2
	Automatic-trip Donut-type hammer	C_E	0.8 – 1.3
Borehole diameter	65-115 mm	C_B	1.0
	150 mm	C_B	1.05
	200 mm	C_B	1.15
Rod length	< 3m	C_R	0.75
	3-4 m	C_R	0.8
	4 – 6 m	C_R	0.85
	6 – 10 m	C_R	0.95
	10 – 30 m	C_R	1.0
Sampling method	Standard sampler	C_S	1.0
	Sampler without liners	C_S	1.1 – 1.3

Table 3.2 Magnitude scaling factors (after Youd, et al., 2001)

Earthquake magnitude, M_w	**Lower bound [equation (3.14a)]**	**Upper bound [equation (3.14b)]**
5.5	2.20	2.80
6.0	1.76	2.10
6.5	1.44	1.60
7.0	1.19	1.25
7.5	1.00	1.00
8.0	0.84	
8.5	0.72	

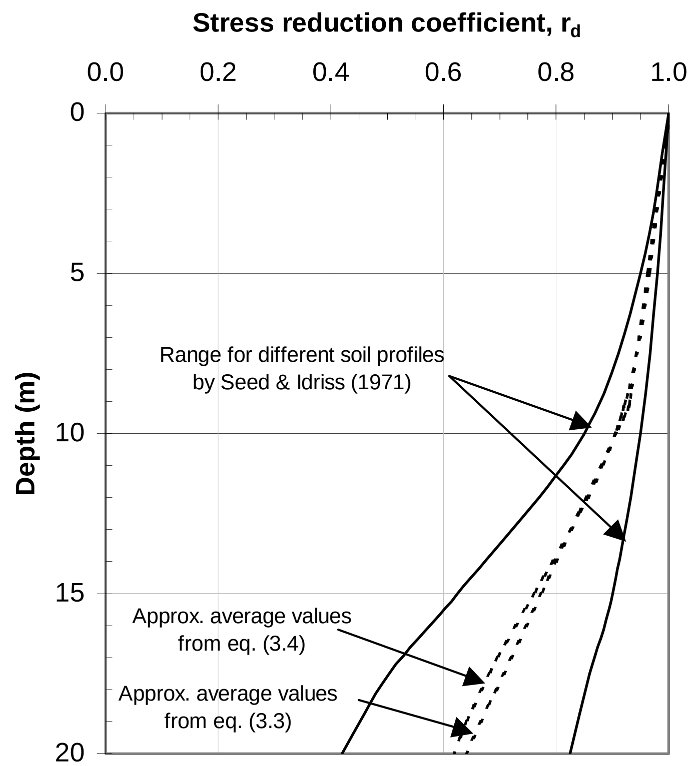

Figure 3.1 Values of stress reduction coefficient versus depth

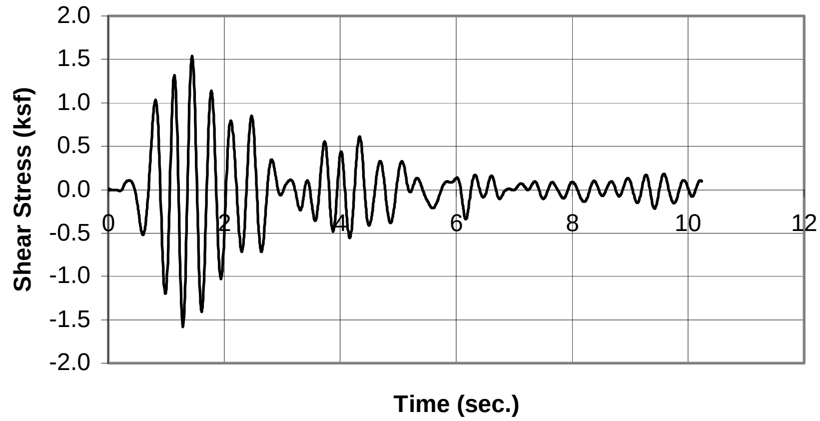

Figure 3.2 Example of shear stress time history during earthquake

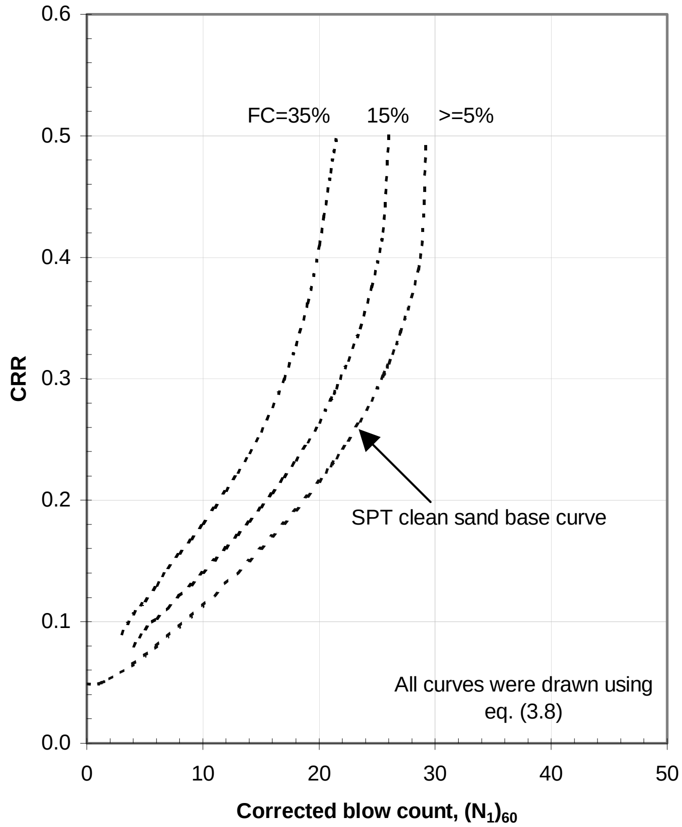

Figure 3.3 SPT clean-sand base curve for magnitude 7.5 earthquake

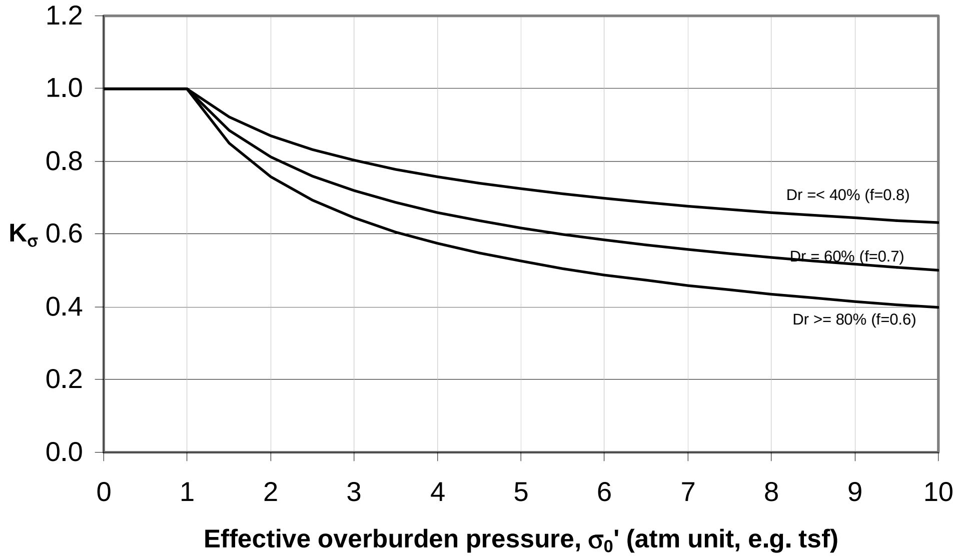

Figure 3.4 K_σ correction factor (after Youd, et al., 2001)

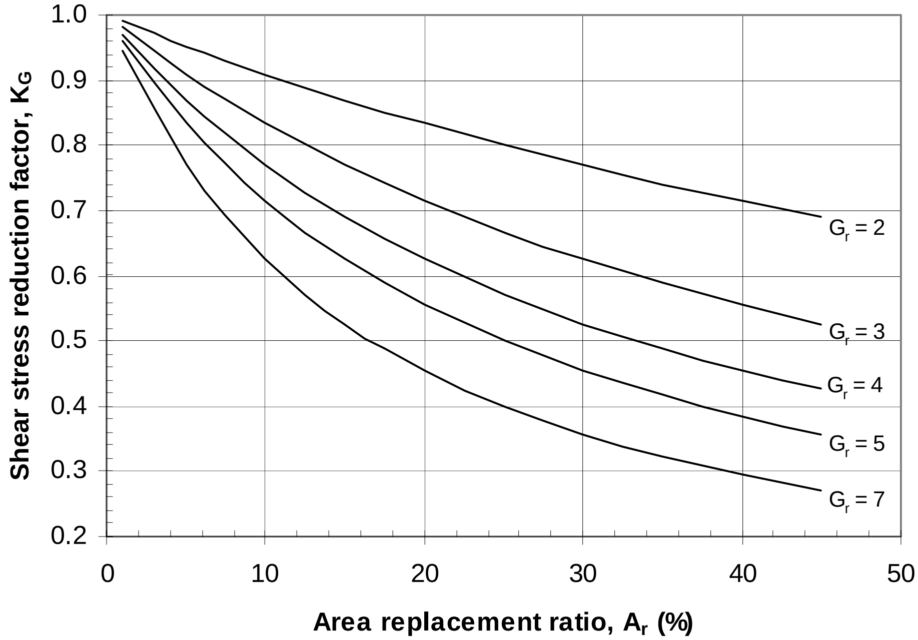

Figure 3.5 The shear stress reduction factor, K_G (after Baez and Martin, 1993, 1994)

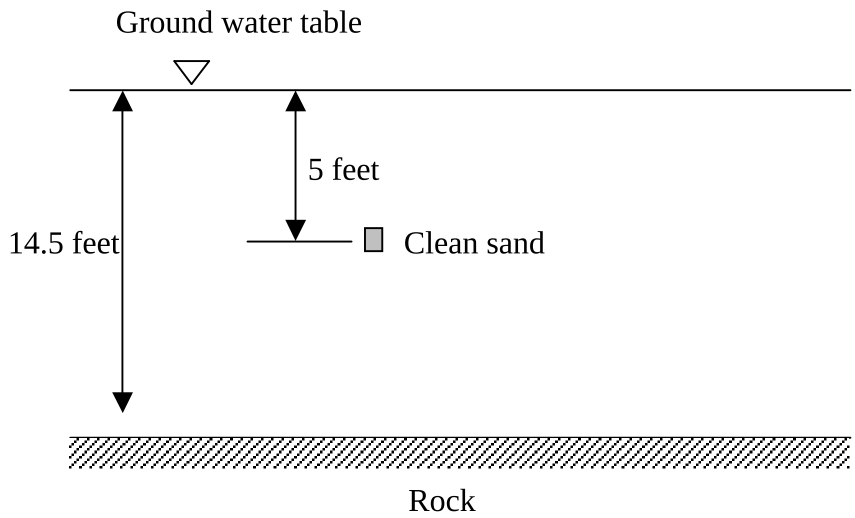

Figure 3.6 Soil profile for calculation example