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A NOVEL METHOD TO ASSESS PAST EARTHQUAKE CHARACTERISTICS

by

Yick Bun Lock

B.Eng., M.Phil.

A thesis submitted to
the Faculty of Graduate Studies and Research
in partial fulfillment of
the requirements for the degree of

Doctor of Philosophy

Department of Civil and Environmental Engineering

The Doctor of Philosophy program
in Civil and Environmental Engineering
is a joint program with the University of Ottawa,
administrated by The Ottawa-Carleton Institute for Civil Engineering

Carleton University

Ottawa, Ontario

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“A NOVEL METHOD TO ASSESS PAST EARTHQUAKE CHARACTERISTICS”

submitted by

Yick Bun Lock, B.Eng., M.Phil.

in partial fulfillment of the requirements for
the degree of Doctor of Philosophy

Chair, Department of Civil and Environmental Engineering

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October 2001

ABSTRACT

The energy method is one of the possible approaches in assessing seismic liquefaction potential. With the recent advance in the development of earthquake strong motion modeling, a better estimation of the attenuation parameter for the seismic energy arriving at a specific site can be obtained. Together with the idea of normalized seismic energy and equivalent clean sand SPT (Standard Penetration Test) value, a new energy method is developed based on a large database of 363 case records.

In order to facilitate interchangeable use of the SPT and CPT (Cone Penetration Test) values, a reliable correlation between the two values is necessary. This is particular important when the correlation is used for the purpose of assessing past earthquake characteristics. Besides, there are equally large number of methods based on either one of these test values and in many situations, the use of both values are required. A new correlation between SPT and CPT values is introduced with several advantages over those existing correlations. The new correlation is based on case records of liquefiable soils only and is a linear function between the equivalent clean sand SPT value and the equivalent clean sand CPT value. Empirical validations have been performed via other soil parameters and good agreements have been shown.

In the past, earthquakes can only be recorded qualitatively or in many cases they were not even recorded. A novel method is proposed in this thesis for the quantification of past earthquake characteristics based on a combination of geotechnical and seismological principles. A total of four independent equations are developed which can be solved with information commonly obtained in a geotechnical ground investigation program. The earthquake parameters obtained are the moment magnitude, the maximum peak horizontal accelerations at bedrock and ground surface and the hypocentral distance of an earthquake that had caused soil liquefaction. Four case studies have been conducted in order to examine the applicability of the proposed method. Comparison of the estimated seismic parameters with actual measurements from four cases shows that this method has a good potential for assessing the parameters of past earthquakes from all over the world.

TO MY PARENTS

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LIST OF SYMBOLS

A	: Maximum peak horizontal acceleration amplification factor
a	: Material attenuation factor
a_i	: Constants for different geological conditions ($i = 1$ to 10)
$a_{\max}$	: Maximum peak horizontal acceleration of the bedrock
$a_{s-\max}$	: Maximum peak horizontal acceleration at the ground surface
B	: Energy attenuation coefficient
b_i	: Constants ($i = 1$ to 4)
C	: SPT and CPT correlation factor
C_1	: CPT correction factor for effective stress
C_2	: CPT correction factor for mean grain size
C_E	: SPT correction factor for different sampling equipment
C_{JAP}	: SPT correction factor for Japan data
C_{LIN}	: SPT correction factor for sampler without liners
C_N	: SPT correction factor for effective stress
C_Q	: CPT correction factor for overburden stress
C_{ROD}	: SPT correction factor for depth of SPT less than 3.0 m
CC	: Calibration chamber
CPT	: Cone penetration test

CRR	: Cyclic resistance ratio
CRR_{7.5}	: Cyclic resistance ratio equivalent to an earthquake magnitude of 7.5
CSR	: Cyclic stress ratio
c_i	: Constants or coefficients (i = 1 to 4)
D_r	: Relative density
D₅₀	: Mean grain size
d	: Closest distance to the fault plane
E	: Total seismic energy
E(R)	: Seismic energy arriving at a site with distance R
E_n(R)	: Normalized seismic energy arriving at a site with distance R
E(r)	: Seismic energy arriving at a site with distance r
ENA	: Eastern North America
ER	: Energy ratio
e_{max}	: Maximum void ratio
e_{min}	: Minimum void ratio
F	: Normalized friction ratio, foundation factor
F_i()	: Function of the factors within the brackets (i = 1 to 4)
FC	: Fines content
f_s	: CPT sleeve friction stress
G	: Shear modulus
g	: Gravitational acceleration
h	: Equivalent point-source depth

I_c	: Soil behaviour type index
K_c	: CPT grain characteristics correction factor
K_0	: Coefficient of earth pressure at rest
L	: Linear function between SPT and CPT
Lg -wave	: Higher mode surface wave
M	: Earthquake magnitude
M_0	: Seismic moment
M_s	: Surface wave magnitude of an earthquake
M_w	: Moment magnitude of an earthquake
MSF	: Magnitude scaling factor
m_N	: Nuttli magnitude
N	: SPT resistance
N_1	: Normalized SPT N-value
$N_{1,60}$	: Corrected normalized SPT N-value
$(N_{1,60})_{cs}$	: Equivalent clean sand SPT resistance
N_{60}	: SPT resistance standardized to 60% energy efficiency
NC	: Normally consolidated
n	: Exponent or total number of data points
OC	: Overconsolidated
OCR	: Overconsolidation ratio
P_a	: Reference pressure in the same units as σ'_v
P_{a2}	: Reference pressure in the same units as q_c

PGA	: Peak ground acceleration
PGV	: Peak ground velocity
PSA	: Pseudo-acceleration
p	: Attenuation parameter
Q	: Normalized CPT penetration resistance
q_c	: CPT tip resistance
q_{cl}	: Corrected CPT q_c value
(q_{cl})_{cr}	: Critical normalized CPT tip resistance
q_{clN}	: Truly normalized cone penetration resistance
(q_{clN})_{cs}	: Equivalent clean sand normalized cone penetration resistance
R	: Hypocentral or closest distance to the fault
r	: Distance between the seismic source and the site
r_d	: Stress reduction factor
S	: Seismic energy function
SF	: Scaling factor for application to earthquake magnitude other than 7-1/2
SPT	: Standard penetration test
T	: Seismic energy intensity function
z	: Depth below the ground surface
α, λ	: SPT fines content correction factors
β	: Shear-wave velocity
φ	: Friction angle of sand
η_I	: Dynamic strength function

- θ : In-situ properties of the soil that liquefied under the given earthquake
- σ_v : Total vertical in-situ stress
- σ'_v : Effective vertical in-situ stress
- τ_{ave} : Average cyclic shear stress
- ψ : SPT energy correction factor

Chapter 1

INTRODUCTION

One of the most significant factors leading to ground failure during earthquakes is the liquefaction of loose to medium-dense sands below the watertable. Attention was focussed on this problem for the first time as a result of the widespread ground failures during the 1964 earthquake in Niigata, Japan which caused building settlement, tilting and flotation, sand boils, ground cracks, landslides, dam instability, highway embankment failures and many other hazards. Most of the damage in Niigata attributable directly to the earthquake was associated with liquefaction and the hazards associated with soil liquefaction during earthquakes have then become increasingly apparent (Wong et al., 1975). Well-known examples in North America are the 1964 Alaska earthquake (the second largest earthquake ever recorded) and the Loma Prieta earthquake of 1989.

Terzaghi and Peck (1948) referred to “spontaneous liquefaction” to describe the sudden loss of strength of loose sand fills and natural sand deposits that flow like a viscous fluid due to a slight disturbance. During an earthquake, the shaking of a granular soil tends to compact it. The water in the pores cannot escape quickly enough, at least in the finer sands, to accommodate instantaneously the compaction. Therefore stresses are thrown on

the water to generate an excess porewater pressure resulting in a reduction in the effective or intergranular stresses between the sand particles. This in turn leads to strength loss in the granular material as it is a frictional material whose strength depends on the effective stress.

The shear modulus, G , is also a function of the mean normal effective stress, and therefore as the excess porewater pressure increases, the shear modulus decreases. In the extreme case, such as in Niigata, where nearly all shear strength and shear stiffness was lost at some sites, the sand behaved like a liquid, with disastrous consequences for structures. This phenomenon is known as *liquefaction*.

The liquefaction potential of a level site, however, is affected by a number of factors, including soil properties, groundwater table, overburden pressure and the intensity and duration of earthquake shaking. In the last thirty years, the factors that control soil liquefaction have been investigated by means of laboratory tests, in-situ tests, and case studies. Some approaches to the assessment of liquefaction have been developed based on the investigations (e.g. Seed and DeAlba, 1986, Law et al., 1990 and Robertson and Wride, 1998) and the potential for liquefaction occurrence can now be estimated with a reasonable degree of confidence.

Nevertheless, how good is the prediction of liquefaction occurrence depends on how well the earthquake magnitude is estimated. It is difficult to estimate the maximum earthquake magnitude in any region, because our history of past earthquakes is not long

enough for us to have a reliable estimate of maximum earthquake size, especially in regions with relatively low recurrence rates for large events. Earthquakes have occurred for millions of years and will continue in the future as they have in the past. Thus, a more accurate quantification of past earthquakes can improve the accuracy of the seismic zoning maps and enhance our ability to assess potential seismic hazards.

The aim of this research is to investigate the possibility that liquefaction failures, incorporating both geotechnical and seismological principles, may be used to assess the magnitudes, hypocentral distances and/or peak ground motions of past earthquakes. The idea of this novel method is to combine four independent equations governing the liquefaction relationships between ground acceleration, earthquake magnitude, hypocentral distance and attenuation characteristics of seismic energy together and then solve simultaneously in order to obtain the required unknowns. The method will lead to a more accurate quantification of past earthquakes in terms of earthquake magnitudes, hypocentral distances and ground motion accelerations.

To achieve the goal of the research, the development of a new energy approach for assessing seismic liquefaction potential is needed. This is in view of the fact that all existing energy methods in the literature are based on simple assumptions of the attenuation properties of the energy arriving at a specific site and thus are not sufficiently accurate to adopt in the novel method discussed above. The energy method proposed here incorporates recent advance in the modeling of earthquake strong motion in the past 10 years. The incorporation leads to the development of a new equation for estimating

the energy arriving at a site. The new energy approach, in order to have a better representation for SPT data at different depths and with different fines contents, also introduced the idea of normalized seismic energy and equivalent clean sand SPT N-value. The new method is then developed based on a large database of 160 liquefaction cases and 203 non-liquefaction cases from 30 earthquakes of more than 10 countries over the world.

Lastly, a new correlation between Standard Penetration Test (SPT) and Cone Penetration Test (CPT) values is also presented. The conventional correlations between SPT resistance, N , and CPT tip resistance, q_c , rely on relationships between the q_c/N ratio and the mean grain size, D_{50} , for application to different granular soils. The new correlation, however, allows a more direct application by using the concept of “equivalent clean sand” for both the N -value and the q_c -value. Another improvement is that the correlation is developed specifically on the basis of liquefiable materials only. As a result, the correlation produces a more reliable assessment of the characteristics of past earthquakes that have caused soil liquefaction. Validations of the new correlation via other soil parameters are also given.

The next chapter gives a literature review of relevant equations on both pertinent geotechnical and seismological principles that are related to earthquake liquefaction failures. After that, Chapters 3 through 5 describe the above mentioned three main themes of this research. Chapter 6 then presents case studies on four earthquakes followed by the last chapter on conclusions and suggestions for future research.

Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

In order to develop a novel method to assess past earthquakes that had caused soil liquefaction failures, both pertinent geotechnical and seismological principles related to earthquake liquefaction failures are needed. The purpose of this chapter is to outline the relevant equations that are generally accepted and used by engineers or seismologists nowadays.

Geotechnical methods in evaluating the liquefaction potential of a deposit of saturated sand subjected to earthquake shaking can be divided into two classes of analysis. The first class is directed largely towards the evaluation of the loading conditions required to trigger liquefaction. This loading is described in terms of cyclic shear stresses, and liquefaction potential is evaluated on the basis of the amplitude and number of cycles of earthquake-induced shear stress. This general approach has come to be known as the *stress method*. The second class is based on the dissipated seismic energy and is called the *energy method*. The use of dissipated energy as a measure of liquefaction resistance

offers a number of advantages: (1) dissipated energy is related to both cyclic stresses and cyclic strain; (2) it can be related to inherently stochastic earthquake ground motions in a way that methods based on peak ground motion parameters alone cannot; (3) it is a scalar quantity; and (4) it can be related to fundamental earthquake parameters. Details will be given in Chapter 3 in developing the new energy approach.

2.2 THE STRESS METHODS

The essential principle of this method is to compare the dynamic strength of the soil with the seismic stress induced during an earthquake. The dynamic strength can be obtained either by means of laboratory cyclic triaxial tests on representative samples of the soil deposit involved or from the tip resistance of the Cone Penetration Test (CPT) or the resistance of the Standard Penetration Test (SPT). The seismic stress is obtained from the maximum peak horizontal acceleration at the ground surface induced by the earthquake. When the strength is less than the stress, liquefaction failure will take place. Well-known examples of this approach are given by Seed and DeAlba (1986), Robertson and Campanella (1985) and Shibata and Teparaksa (1988).

After decades of development and gathering of more liquefaction field data, the most up-to-date method that can account for the effects of fines content (FC) for different types of sandy materials is the one by Robertson and Wride (1998). The following sections give a brief review on all the four above-mentioned methods, with the last method described in a

more detail as it is the one that is going to be used in the past earthquake characteristics assessment method developed in Chapter 5.

Method by Seed and DeAlba (1986):

The method by Seed and DeAlba (1986) is an updated version of the method proposed earlier by Seed et al. (1983). The original method is based on the observations of granular soil performance during earthquakes with magnitude, M , of about 7-1/2. From these observations, the cyclic stress ratio (CSR) was calculated, the SPT resistance, N , recorded and the existence or non-existence of liquefaction failure noted. The cyclic stress ratio corresponds to the average cyclic shear stress, τ_{ave} , during the earthquake, normalized by the effective vertical in-situ stress, σ'_v . It is given by

$$CSR = \frac{\tau_{ave}}{\sigma'_v} = 0.65 \frac{a_{s-max}}{g} \cdot \frac{\sigma_v}{\sigma'_v} r_d \quad (2.1)$$

where a_{s-max} = maximum peak horizontal acceleration at the ground surface

g = gravitational acceleration

σ_v, σ'_v = total and effective vertical in-situ stress at the depth where the CSR is estimated

r_d = a stress reduction factor that decreases from a value of 1.0 at the ground surface to a value about 0.9 at a depth of 10.7 m

The values of τ_{ave}/σ'_v are then plotted vs the normalized SPT resistance, N_1 , obtained by multiplying N with C_N . The value of C_N is given by a curve (Figure 2.1) which is based on data developed by Bieganousky and Marcuson (1976) and Marcuson and Bieganousky (1977). This curve branches into two parts for σ'_v exceeding about 96 kPa. One is for soil with a relative density, D_r , of 60% to 80% and the other for $D_r = 40\%$ to 60%. The curve terminates at a limit of $\sigma'_v = 36$ kPa. Extrapolation of C_N at pressures below this value is unreliable because the slope is quite flat there.

From the plot of τ_{ave}/σ'_v vs N_1 , a curve is drawn to separate the liquefied and the non-liquefied cases. Thus, this curve represents the maximum cyclic stress ratio the soil can sustain for a given N_1 value before liquefaction failure. This maximum cyclic stress ratio can be a function of N_1 or q_{cl} by means of the simple conversion

$$q_{cl} = CN_1 \quad (2.2)$$

where C is a factor depending on the mean grain size and q_{cl} is the corrected CPT resistance. The relationship between $(\tau_{ave}/\sigma'_v)_{max}$ and q_{cl} can be written as:

$$(\tau_{ave}/\sigma'_v)_{max} = SF \cdot F(q_{cl}) \quad (2.3)$$

where SF is a scaling factor for application to earthquake magnitude other than 7-1/2 and the value is shown in Table 2.1 and F is a function of q_{cl} .

Figure 2.1: Chart for values of C_N (after Seed and DeAlba, 1986)

M	8-1/2	7-1/2	6-3/4	6	5-1/4
SF	0.89	1.0	1.13	1.32	1.5

Table 2.1: Scaling factor SF for various earthquake magnitude M (after Seed and DeAlba, 1986)

In the upgraded version of Seed and DeAlba (1986), improvements were made on the C factor and different curves were developed to account for soil of different grain sizes.

Method by Robertson and Campanella (1985):

This method is in principle similar to that of Seed et al. (1983). The same equation for seismic stress ratio and the scaling factor, SF, for application to earthquake magnitudes other than 7-1/2 are employed as in the method of Seed et al. The curve for dividing the liquefied and non-liquefied cases, however, is different because additional data of liquefied cases have been considered. Hence, for the same soil deposit subject to the same seismic event this method predicts a higher tip resistance required to prevent liquefaction failure. This is particularly true in the range of seismic stress ratio ranging from 0.1 to 0.3, a range within which many soil liquefaction cases occur.

This method also attempts to partially account for the effects of grain size. Two curves are proposed: one for mean grain size, $D_{50} \geq 0.25$ mm and the other for $D_{50} \leq 0.15$ mm.

Method by Shibata and Teparaksa (1988):

This method is based on direct correlation between CPT tip resistance, q_c , and the seismic stress ratio measured at sites with and without liquefaction failures. As it has been recognized that the continuous nature of the CPT profile is more suitable for the stress method. The value of q_c is first corrected to a reference effective vertical in-situ stress, $\sigma'_v = 0.1$ MPa by

$$q_{cl} = C_1 \cdot q_c \quad (2.4)$$

where q_{cl} = corrected tip resistance

q_c = measured tip resistance

C_1 = correction factor, a function of σ'_v , at the depth at which the CPT was conducted

The correction factor, C_1 is based on the comprehensive data produced by Baldi et al. (1982), and can be explicitly represented by:

$$C_1 = \frac{0.17}{\sigma'_v + 0.07} \quad (2.5)$$

where σ'_v is in MPa.

A critical normalized tip resistance, $(q_{cl})_{cr}$, is then obtained directly from the data and is expressed as

$$(q_{cl})_{cr} = C_2 \left[5 + 20 \left\{ \frac{(\tau_h/\sigma'_v - 0.1)}{(\tau_h/\sigma'_v + 0.1)} \right\} \right] \quad (2.6)$$

where $(q_{cl})_{cr}$ is in MPa, C_2 is a factor depending on the mean grain size, D_{50} , and is given by

$$C_2 = 1.0 \text{ for } D_{50} \geq 0.25 \text{ mm} \quad (2.7a)$$

$$C_2 = \frac{D_{50}}{0.25} \text{ for } D_{50} < 0.25 \text{ mm} \quad (2.7b)$$

The cyclic stress ratio (τ_h/σ'_v) in this case is obtained by incorporation of the earthquake magnitude

$$\frac{\tau_h}{\sigma'_v} = 0.1(M - 1) \frac{a_{s-\max}}{g} \frac{\sigma_v}{\sigma'_v} (1 - 0.015z) \quad (2.8)$$

where M = earthquake magnitude, and z = depth below the ground surface in metres (for $z < 25$ m). Equation 2.8 was originally proposed by Tokimatsu and Yoshimi (1983) adopting the reduction factor for dynamic shear stress accounting for the elastic performance of the ground by Iwasaki et al. (1978).

Method by Robertson and Wride (1998):

The state-of-the-art liquefaction potential assessment method by Robertson and Wride (1998) is developed based on discussions at the 1996 NCEER workshop on evaluation of liquefaction resistance of soils (NCEER, 1997) and is a modification of the original method by Robertson and Campanella (1985) in order to be more consistent with the recent increases in the availability of CPT field performance data. The resulting recommended CPT correlation for clean sand is

$$\text{if } 50 \leq (q_{c1N})_{cs} < 160, \quad CRR = 93 \left[\frac{(q_{c1N})_{cs}}{1000} \right]^3 + 0.08 \quad (2.9a)$$

$$\text{if } (q_{c1N})_{cs} < 50, \quad CRR = 0.833 \left[\frac{(q_{c1N})_{cs}}{1000} \right] + 0.05$$

(2.9b)

where CRR is the cyclic resistance ratio and q_{c1N} is the truly normalized (i.e. dimensionless) cone penetration resistance corrected for overburden stress and is given by

$$q_{c1N} = \left(\frac{q_c}{P_{a2}} \right) C_Q \quad (2.10)$$

where q_c is the measured cone tip penetration resistance; $C_Q = (P_a/\sigma'_v)^n$ is a correction factor for overburden stress; the exponent n is typically equal to 0.5; P_a is a reference

pressure in the same units as σ'_v (i.e., $P_a = 100$ kPa if σ'_v is in kPa); and P_{a2} is a reference pressure in the same units as q_c (i.e., $P_{a2} = 0.1$ MPa if q_c is in MPa). A maximum value of $C_Q = 2$ is generally applied to CPT data at shallow depths. The normalized cone penetration resistance, q_{c1N} , is dimensionless.

The equivalent clean sand normalized CPT penetration resistance, $(q_{c1N})_{cs}$, is a function of both the measured penetration resistance, q_{c1N} , and the grain characteristics of the soil, as follows

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad (2.11)$$

where K_c is the grain characteristics correction factor that depends on the soil behaviour type index. I_c , as proposed by Robertson (1990):

$$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5} \quad (2.12)$$

where $Q = \left(\frac{q_c - \sigma_v}{P_{a2}} \right) \left(\frac{P_a}{\sigma'_v} \right)^n$ is the normalized CPT penetration resistance (dimensionless);

the exponent n is typically equal to 1.0; $F = [f_s / (q_c - \sigma_v)] 100$ is the normalized friction ratio, in percent; f_s is the CPT sleeve friction; σ_v and σ'_v are the total and effective vertical in-situ stresses, respectively; P_a is a reference pressure in the same units as σ'_v

(i.e. $P_a = 100 \text{ kPa}$ if σ'_v is in kPa); and P_{a2} is a reference pressure in the same units as q_c and σ_v (i.e., $P_{a2} = 0.1 \text{ MPa}$ if q_c and σ_v are in MPa).

The exponent $n = 1.0$ is recommended for soils when $I_c > 2.6$. However, when $I_c \leq 2.6$, it is recommended to use $n = 0.5$. Hence, the recommended procedure is to first use $n = 1.0$ to calculate Q and, therefore, an initial value of I_c for CPT data. If $I_c > 2.6$, assume $q_{c1N} = Q$. However, if $I_c \leq 2.6$, the exponent to calculate Q should be changed to $n = 0.5$ (i.e., essentially calculate q_{c1N} using Equation 2.10, since $\sigma_v \ll q_c$) and I_c should be recalculated based on q_{c1N} and F . If the recalculated I_c remains less than 2.6, use q_{c1N} based on $n = 0.5$. If, however, I_c iterates above and below a value of 2.6, depending which value of n is used, a value of $n = 0.75$ should be selected to calculate q_{c1N} (using Equation 2.10). Note that if the effective vertical in-situ stresses are in the order of 50 – 150 kPa , the choice of normalization has little effect on the calculated normalized penetration resistance.

Then the recommended relationship between I_c and the correction factor K_c is given by the following equations

$$\text{if } I_c \leq 1.64, \quad K_c = 1.0 \quad (2.13a)$$

$$\text{if } I_c > 1.64, \quad K_c = -0.403 I_c^4 + 5.581 I_c^3 - 21.63 I_c^2 + 33.75 I_c - 17.88 \quad (2.13b)$$

Experience has shown that the CPT friction ratio increases with increasing fines content and the soil behaviour type index can be used to estimate the apparent fines content by the following simplified relationship

$$\text{if } I_c < 1.26 \quad \text{apparent fines content FC (\%)} = 0 \quad (2.14a)$$

$$\text{if } 1.26 \leq I_c \leq 3.5 \quad \text{apparent fines content FC (\%)} = 1.75 I_c^{3.25} - 3.7 \quad (2.14b)$$

$$\text{if } I_c > 3.5 \quad \text{apparent fines content FC (\%)} = 100 \quad (2.14c)$$

In summary, Equation 2.9 to Equation 2.14 can be combined to provide an integrated method for evaluating the cyclic resistance ($M = 7.5$) of saturated sandy soils based on the CPT. This integrated CPT method is summarized in Figure 2.2 in the form of a flow chart. The flow chart clearly shows the step-by-step process involved in using the integrated method based on the CPT for evaluating $CRR_{7.5}$ and indicates the recommended equations for each step of the process.

$CRR_{7.5}$ is the cyclic resistance ratio (CRR) value taken at about 15 cycles of uniform loading to represent an equivalent earthquake loading of magnitude 7.5. The CRR for any other size earthquake can be estimated using the following equation

$$CRR = (CRR_{7.5})(MSF) \quad (2.15)$$

where MSF is the magnitude scaling factor

Figure 2.2: Flow chat illustrating the application of the integrated CPT method (after Robertson and Wride, 1998)

$$MSF = \left(\frac{7.5}{M_w} \right)^{2.56} \quad (2.16)$$

as recommended by the 1996 NCEER workshop on evaluation of liquefaction resistance of soils (NCEER, 1997), where M_w is the moment magnitude of an earthquake.

At any depth, if CSR is greater than CRR, liquefaction is possible. The equation of CSR is the same as given by Equation 2.1 but with the value of r_d being more well defined.

The following equations are used to estimate average values of r_d

$$r_d = 1.0 - 0.00765 z \quad \text{for } z \leq 9.15 \text{ m} \quad (2.17a)$$

$$r_d = 1.174 - 0.0267 z \quad \text{for } 9.15 \text{ m} < z \leq 23 \text{ m} \quad (2.17b)$$

$$r_d = 0.744 - 0.008 z \quad \text{for } 23 < z \leq 30 \text{ m} \quad (2.17c)$$

$$r_d = 0.50 \quad \text{for } z > 30 \text{ m} \quad (2.17d)$$

where z is the depth below ground surface in metres.

The CPT provides continuous profiles of penetration resistance, which are useful for identifying soil stratigraphy and for providing continuous profiles of estimated cyclic resistance ratio (CRR). Therefore, CPT is preferred in the stress method. However, corrections are required for both the SPT and CPT for grain characteristics, such as the fines content. The advantage of Robertson and Wride (1998) method is that it allows the estimation of the apparent fines content directly from CPT results. As a result, following

the traditional SPT-based approach, the estimated apparent fines content can be used to estimate the correction necessary to obtain the clean sand equivalent penetration resistance and thus the liquefaction potential can be assessed immediately without the necessity of other index testing to get the soil characteristics.

The key advantage of this integrated CPT method is that the algorithms can easily be incorporated into a spreadsheet. The result obtained is a straightforward method for analyzing entire CPT profiles in a continuous manner. This provides a useful tool for the engineer to review the potential for cyclic liquefaction across a site using engineering judgement. Although, the procedures involved in the integrated method maybe somewhat too complicated to calculate by hand.

2.3 THE ENERGY METHODS

The densification of dry soil involves rearrangement of grains and hence the expenditure of energy. As a cyclically loaded dry soil densifies and approaches its minimum void ratio, the amount of energy required to rearrange individual soil grains increases. For a saturated soil, however, the tendency for densification causes the pore pressure to increase and the interparticle contact forces to decrease. As these contact forces decrease, the amount of energy needed to rearrange soil grains decreases (Kramer, 1996). By combining these observations, Nemat-Nasser and Shokooh (1979) developed a simple, unified theory that related densification under drained conditions and pore pressure generation under undrained conditions to dissipated energy. Others have since attempted

to characterize the relationship between excess pore pressure and dissipated energy experimentally (e.g. Simcock et al., 1983; Cao and Law, 1990; Figueroa, 1990; Cao and Law, 1991; Figueroa and Dahisaria, 1991). Davis and Berrill (1982), Berrill and Davis (1985) and Law et al. (1990) have successfully demonstrated the potential for using energy to relate liquefaction behavior to fundamental earthquake parameters such as magnitude and distance.

The above methods, however, are based on simple assumptions of the attenuation properties of the energy arriving at a specific site. This is due to the lack of the more recent sophisticated developments in the seismological principles at the time when the methods were developed. Therefore, these methods are not used to develop a new approach to assess past earthquake characteristics as described in Chapter 5. Instead, a new method to incorporate updated seismological findings is established in Chapter 3.

Method by Davis and Berrill (1982):

A statistical representation of seismic liquefaction was advanced based on the postulate that pore water pressure increases are proportional to the dissipated seismic energy density. The energy content of the seismic waves arriving at the site is estimated from the total radiated energy of the earthquake and the distance to the centre of energy release. The total radiated energy is related to the earthquake magnitude via the Gutenberg and Richter relationship. The energy arriving at a site is assumed using the simple energy attenuation model for spherically expanding waves, of which the energy is inversely

proportional to the square of distance. The line of demarcation separating the liquefied and nonliquefied sites, based on approximately fifty case histories, relates the standard penetration value to earthquake magnitude, distance to centre of energy release and initial effective overburden stress is then obtained as below

$$\frac{r^2 \sigma_v'^{3/2}}{10^{1.5M}} = \frac{450}{N_1^2} \quad (2.18)$$

where r is the distance to the centre of energy release and is in metres, N_1 is the normalized SPT N-value as has been described previously in the stress method and σ_v' is in kN/m^2 .

Method by Berrill and Davis (1985):

The Berrill and Davis (1985) method is a revision of the earlier method by Davis and Berrill (1982). It is also based on liquefaction case history data and the hypothesis that increase in pore water pressure is proportional to the density of seismic energy dissipation, distance, initial effective overburden stress and standard penetration value of the site soil. The revised method, however, includes an allowance for material attenuation between the earthquake source and the site and a non linear relation, based on laboratory testing, between pore pressure increase and density of dissipated seismic energy in the site soil.

The equation then becomes

$$\left[\frac{r^2 \sigma_v'^{3/2}}{a \cdot 10^{1.5M}} \right]^{0.5} = \frac{120}{N_1^{1.5}} \quad (2.19)$$

where r is the distance of the site from the centre of energy release, N_1 is the normalized SPT N-value, σ_v' is in kN/m^2 and a is the material attenuation factor that decreases from a value of 1.0 at close distance to a value of 0 at very far away distance.

Method by Law et al. (1990):

A method based on an energy concept for assessing seismic liquefaction potential has been proposed by Law et al. (1990). This concept is derived from the experimental observation that liquefaction potential of granular soil can be related to the energy dissipated through hysteretic damping and plastic deformation. During an earthquake, energy is released and travels to various parts of the earth surface where soil deposits lie. By comparing the earthquake energy arriving at a site with the energy required to cause liquefaction there, an assessment of liquefaction potential can be made. Mathematically, this condition is expressed as

$$T/\eta_1 \geq 1.0 \quad (2.20)$$

where T = seismic energy intensity function

η_1 = dynamic strength function

A number of coefficients are involved in the two functions. They are evaluated on the basis of total energy from a seismic event, the attenuation of seismic energy with hypocentral distance, R , and case records of North and South America, China and Japan. The seismic energy intensity function, T , is defined as

$$T = 10^{1.5M}/R^B \quad (2.21)$$

where B is the energy attenuation coefficient and depends on the rock properties.

The value of B is determined indirectly by assuming that energy attenuation is proportional to earthquake intensity attenuation. The proportionality constant is then lumped into one of the coefficients associated with η_1 . Based on the work of Hasegawa et al. (1981), B is taken as 4.3 for the American west coast where rock is highly fractured due to interplate tectonic activities and 3.2 for the Eastern Canada where bedrock is much sounder.

The dynamic strength function was determined by plotting T vs N_1 , the normalized SPT resistance, for all the cases studied. In normalizing, an arbitrary reference $\sigma'_v = 100$ kPa was used as proposed by Seed et al. (1983) and the following correction applies:

$$N_1 = C_N N_{60} \quad (2.22)$$

where C_N = a function of σ'_v at the depth where the penetration test was conducted.

Before correction for the effective overburden pressure, the SPT resistance has to be corrected for energy and operational variations by using

$$N_{60} = \psi N \quad (2.23)$$

as suggested by Seed et al. (1985) where N_{60} = SPT resistance standardized to 60% energy efficiency, $\psi = ER/60$ and ER = energy efficiency of SPT system from which the uncorrected resistance N was obtained. The fully corrected N_f value was then used and lines are drawn to divide liquefied and non-liquefied cases for sand and silty sand. From these lines the following are obtained

$$\eta_1(N_f) = 2.28 \times N_f^{11.5} \times 10^{-10} \quad \text{for sand} \quad (2.24a)$$

$$\eta_1(N_f) = 1.14 \times N_f^{11.5} \times 10^{-9} \quad \text{for silty sand} \quad (2.24b)$$

2.4 THE SEISMOLOGICAL PRINCIPLES

The most commonly used measure of the amplitude of a particular ground motion is the maximum peak horizontal acceleration, $a_{\max}$. The $a_{\max}$ for a given component of motion is simply the largest (absolute) value of horizontal acceleration obtained from the

accelerogram of that component. By taking the vector sum of two orthogonal horizontal components, the maximum resultant $a_{\max}$ can be obtained.

Horizontal accelerations have commonly been used to describe ground motions because of their natural relationship to inertial forces; indeed, the largest dynamic forces induced in certain types of structures (i.e. very stiff structures) are closely related to the $a_{\max}$. The $a_{\max}$, on the other hand, can also be correlated to earthquake intensity. Although this correlation is far from precise, it can be very useful for estimating $a_{\max}$ when only intensity information is available, as in the cases of earthquakes that occurred before strong motion instruments were available (preinstrumental earthquakes). The use of intensity-attenuation relationships also allows estimation of the spatial variability of peak acceleration from the isoseismal maps of historical earthquakes (Kramer, 1996).

First correlations of peak accelerations with earthquake magnitude and distance were reported by Gutenberg and Richter in 1942 and 1956 and by Neumann in 1954. The study by Gutenberg and Richter in 1956 represents the first systematic analysis of peak accelerations. Numerous other correlations have been presented since then (e.g. Trifunac and Brady, 1976; Hasegawa et al., 1981; Atkinson, 1984). For the development of the novel method to assess past earthquakes proposed in this study, the most up-to-date correlations of peak acceleration with earthquake magnitude, distance and site conditions are used. These include the Atkinson and Boore (1995) ground-motion relations for Eastern North America and the Atkinson and Silva (2000) ground-motion relations for California.

Correlation by Atkinson and Boore (1995):

Empirical studies (Atkinson, 1993a; Atkinson and Mereu, 1992 and Atkinson, 1993b) of over 1500 seismograms from Eastern North America (ENA) earthquakes in the magnitude ranging from 3.5 to 6.8 have provided significant new information on ENA ground-motion processes. These studies show that (1) the source spectra for ENA earthquakes of $M_w > 4$ deviate significantly from the Brune 100-bar model (Atkinson, 1993a); (2) the attenuation of spectral amplitudes is slightly disrupted by the transition from direct-wave to *Lg*-wave spreading, suggesting a hinged trilinear form for the attenuation curve (Atkinson and Mereu, 1992); (3) the duration of motion increases with distance in a complex manner (Atkinson, 1993b); and (4) the ratio of horizontal-to-vertical component ground motions is frequency dependent, but independent of distance (Atkinson, 1993b). All the studies described were based on data derived mostly from earthquakes in Southeastern Canada and the Northeastern United States. Wave-propagation studies suggest that ground-motion relations show little regional variability over most of ENA, with the exception of the Gulf Coast region (EPRI, 1993). Therefore ground-motion relations derived from data in Southeastern Canada and the Northeastern United States should be applicable over most of ENA.

Based on the above new information on ENA source and attenuation processes, Atkinson and Boore (1995) developed predictive relations for ground motions from Eastern North American earthquakes of moment magnitude, M_w , ≥ 4.0 and ≤ 7.25 at hypocentral

distances, R , between 10 to 500 km. The prediction parameters are response spectra at frequencies of 0.5 to 20 Hz, and peak ground acceleration and velocity. The predictions are derived from an empirically based stochastic ground-motion model. The relations differ from previous work in the improved empirical definition of input parameters and empirical validation of results. The relations are in demonstrable agreement with ground motions from earthquakes of $M_w = 4$ to 5. There are insufficient data to adequately judge the relations at larger magnitudes, although they are consistent with data from the Saguenay ($M_w = 5.8$) and Nahanni ($M_w = 6.8$) earthquakes. The underlying model parameters are constrained by empirical data for events as large as $M_w = 6.8$ (Atkinson and Boore, 1995).

Moment magnitude is preferred in the ground-motion relations, rather than the more widely catalogued (but more ambiguously defined) Nuttli magnitude (m_N), because it has a simple physical interpretation (Hanks and Kanamori, 1979), and because there is some hope of being able to specify M_w for a future expected earthquake based on geological evidence. The M_w value has been estimated for most of the large historical ENA earthquakes from special studies.

The estimated median ground motions for the random horizontal component at rock sites are summarized in an appendix table in the original paper for different magnitudes and frequencies. Simple quadratic equations to approximate the estimates for the purposes of seismic hazard calculations are also provided as:

$$\log \text{PSA} = c_1 + c_2(M_w - 6) + c_3(M_w - 6)^2 - \log R - c_4R \quad (2.25)$$

where PSA is the pseudo-acceleration (5% damped) for the random horizontal component on rock, PGA is the peak ground acceleration in cm/s^2 and PGV is the peak ground velocity in cm/s according to the coefficients listed in Table 2.2, M_w is the moment magnitude and R is the hypocentral distance.

Frequency (Hz)	c_1	c_2	c_3	c_4
0.5	2.27	0.634	-0.0170	0.0000
0.8	2.60	0.635	-0.0308	0.0000
1.0	2.77	0.620	-0.0409	0.0000
1.3	2.95	0.604	-0.0511	0.0000
2.0	3.26	0.550	-0.0640	0.0000
3.2	3.54	0.475	-0.0717	0.000106
5.0	3.75	0.418	-0.0644	0.000457
7.9	3.92	0.375	-0.0562	0.000898
10.0	3.99	0.360	-0.0527	0.00121
13.0	4.06	0.346	-0.0492	0.00153
20.0	4.19	0.328	-0.0477	0.00226
PGA	3.79	0.298	-0.0536	0.00135
PGV	2.04	0.422	-0.0373	0.0000

Table 2.2: Coefficients for the quadratic equation (ENA)

Correlation by Atkinson and Silva (2000):

The conventional method in developing ground-motion relations for California is by empirical analysis due to the fact that there are sufficient strong ground motion data to provide a solid foundation for empirical ground-motion relations, at least for earthquakes of moment magnitude, M_w , 6 to 7 at distance from 10 to 30 km. Atkinson and Silva

(2000), on the other hand, developed stochastic-based ground-motion relations for California with the following reasons:

- *the stochastic model provides a conceptual framework for understanding some of the basic underlying physical parameters that control observed ground-motion amplitudes and their variability.*
- *ground-motion relations that are underpinned by a physical model form a sound basis for ground-motion predictions in magnitude-distance ranges that are not well represented by the empirical database. This is a particularly important issue for seismic hazard analysis because the common practice, of necessity, has been to apply selected empirical ground-motion relations over a range of magnitudes and distances that is much wider than that covered by the underlying empirical database.*
- *the development of a stochastic ground-motion model for California will facilitate comparisons of ground-motion processes between regions. Insights gained from modeling in the data-rich California region can be used to improve ground motion relations for ENA and other regions with few strong-motion data.*

Ground-motion relations are thus developed for California using a stochastic simulation method that exploits the equivalence between finite-fault models and a two-corner point-source model of the earthquake spectrum. First, stochastic simulations are generated for

finite-fault ruptures, in order to define the average shape and amplitude level of the radiated spectrum at near-source distances as a function of earthquake size. The length and width of the fault plane are defined based on the moment magnitude of the earthquake and modeled with an array of subfaults. The radiation from each subfault is modeled as a Brune point source using the stochastic model approach; the subfault spectrum has a single-corner frequency. An earthquake rupture initiates at a randomly chosen subfault (hypocenter), and propagates in all directions along the fault plane. A subfault is triggered when rupture propagation reaches its center. Simulations are generated for an observation point by summing the subfault time series, appropriately lagged in time. Fourier spectra are computed for records simulated at many azimuths, placed at equidistant observation points around the fault. The mean Fourier spectrum for each magnitude, at a reference near-source distance, is used to define the shape and amplitude levels of an equivalent point-source spectrum that mimics the salient finite-fault effects. The functional form for the equivalent point-source spectrum contains two corner frequencies.

Stochastic point-source simulations, using the derived two-corner source spectrum, are then performed to predict peak-ground-motion parameters and response spectra for a wide range of magnitudes and distances, for generic California rock sites (Atkinson and Silva, 2000).

The median response spectra and peak ground motions for rock site conditions ($\beta = 620$ m/sec), from the stochastic equivalent-point-source simulations, are listed in an appendix

table in the original paper for different combination of magnitude, fault distance and frequency. For soil sites, the predicted ground-motion amplitudes are modified according to the empirical soil factors of Abrahamson and Silva (1997), which express the nonlinear soil amplification as a function of frequency and input rock peak-ground acceleration.

Simple functional forms for convenient use in seismic hazard analyses are also given in the form

$$\log \text{PSA} = c_1 + c_2(M_w - 6) + c_3(M_w - 6)^2 - \log R - c_4 R \quad (2.26)$$

where PSA is the pseudo-acceleration (5% damped) for the random horizontal component on rock, PGA is the peak ground acceleration in cm/s^2 and PGV is the peak ground velocity in cm/s according to the coefficients listed in Table 2.3, $R = \sqrt{(d^2 + h^2)}$, d is the closest distance to the fault plane in km, h is the equivalent point-source depth ($\log h = -0.05 + 0.15M_w$), and M_w is the moment magnitude.

The stochastic ground-motion relations are in good agreement with the empirical strong-motion database for California and agree well with other empirical regression equations in the magnitude-distance ranges well represented by the data, but are better constrained at large distance, due to the use of attenuation parameters based on regional seismographic data. The stochastic ground-motion relations provide a sound basis for estimating ground motions for earthquakes of magnitudes 4 through 8, at distances from 1 to 200 km.

Frequency (Hz)	c_1	c_2	c_3	c_4
0.1	1.416	0.943	-0.0461	0.00000
0.2	2.031	0.888	-0.0736	0.00000
0.32	2.228	0.858	-0.0809	0.00000
0.5	2.628	0.795	-0.1070	0.00000
1.0	3.150	0.660	-0.1060	0.00620
2.0	3.573	0.508	-0.0760	0.00155
3.2	3.776	0.431	-0.0541	0.00247
5.0	3.911	0.379	-0.0391	0.00349
10.0	3.944	0.328	-0.0232	0.00464
20.0	3.713	0.318	-0.0228	0.00394
PGA	3.556	0.333	-0.0270	0.00303
PGV	2.203	0.502	-0.0207	0.00021

Table 2.3: Coefficients for the quadratic equation (California)

Chapter 3

A NEW ENERGY APPROACH FOR ASSESSING SEISMIC LIQUEFACTION POTENTIAL

3.1 INTRODUCTION

The liquefaction of saturated sandy soil induced by earthquake strong ground motions is a major cause of damage to civil engineering structures. During seismic loading, initial liquefaction occurs when the excess pore water pressure has increased to a level equal to the original effective confining stress. As described in Chapter 2, since the mid-1960's, researchers have attempted to determine the parameters that can define the liquefaction potential of a soil deposit. A procedure based on the standard penetration resistance and cyclic stress was developed by Seed and his colleagues (1967, 1971, 1983 and 1984) using the maximum peak horizontal acceleration at the ground surface to assess the initial liquefaction of soil. This method, known as the "*Stress Method*", is now widely used around the world. On the other hand, since Nemat-Nasser and Shokooh (1979) introduced the energy concept in the analysis of the densification and liquefaction of cohesionless soils, experimental studies have been conducted by Simcock et al. (1983), Cao and Law (1990), Figueroa (1990), Cao and Law (1991) and Figueroa and Dahisaria

(1991) to establish a relationship between pore-water pressure increments and dissipated energy. From the relation, the applicability of the energy concept in the evaluation of the liquefaction potential of a soil deposit has been demonstrated. Since energy is a more fundamental property or measure of earthquake excitation than peak acceleration, it is suggested that the amount of seismic energy arriving at specific points in a soil layer may correlate better than the cyclic stress ratio used in the stress method with the development of liquefaction. The "*Energy Method*", however, has a much shorter history than the stress method and is therefore less commonly used in engineering practice.

This study incorporates recent advance in the modeling of earthquake strong motion in the past 10 years. A new method for estimating the energy arriving at a site is proposed. By using a new energy attenuation parameter and a larger earthquake case history database of liquefied soils, a new energy approach for assessing seismic liquefaction potential is proposed. The idea of normalized seismic energy and the equivalent clean sand SPT N-value are also introduced in developing the new method.

3.2 ENERGY ARRIVING AT A SITE

The total seismic energy, E , released during an earthquake is often estimated from the relationship (Gutenberg and Richter, 1956)

$$\log E = 11.8 + 1.5 M_s \quad (3.1)$$

where E is expressed in ergs and M_s is the magnitude from the amplitude of surface waves for shallow teleseisms (Gutenberg, 1945). However, same as the body wave and Richter local magnitudes, the surface wave magnitude scale is an empirical quantity based on instrumental measurements of ground-shaking characteristics. As the total amount of energy released during an earthquake increases, the ground-shaking characteristics do not necessarily increase at the same rate. For strong earthquakes the measured ground-shaking characteristics become less sensitive to the size of the earthquake than for smaller earthquakes. This phenomenon is referred to as *saturation*; the surface wave magnitude saturates at about $M_s = 8$ while the body wave and Richter local magnitudes saturate at magnitudes of 6 to 7. Therefore, in order to describe the size of very large earthquakes, a magnitude scale that does not depend on ground-shaking levels, and consequently does not saturate, would be desirable. The only magnitude scale that is not subject to saturation is the moment magnitude (Kanamori, 1977; Hanks and Kanamori, 1979) since it is based on the seismic moment, which is a direct measure of the factors that produce rupture along the fault.

Besides, it has also been stated at the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils (NCEER, 1997) that the Moment magnitude, M_w , is the scale most commonly used for engineering applications and is the scale preferred for calculation of liquefaction resistance. For example, engineering seismologists have found that M_w generally correlates better than other scales with attenuation of peak ground motions and spectral ordinates, and, consequently, with earthquake damage. Moment magnitude is calculated from the amount of seismic energy released in earthquakes rather than peak

displacement of an accelerogram trace. Thus M_w is a more theoretically correct measure of earthquake strength and duration than other magnitude estimates for engineering purposes.

As a result, a direct relationship between E and M_w is desirable for energy related liquefaction potential assessment. Kanamori (1983) showed that E can be estimated from the seismic moment, M_o , using the equation: $E = \frac{M_o}{2 \cdot 10^4}$. As $M_w = \frac{\log M_o}{1.5} - 10.7$ (Kanamori, 1977; Hanks and Kanamori, 1979), the relationship between E and M_w becomes:

$$\log E = 1.5 M_w + 11.7 \quad (3.2)$$

where E is expressed in ergs, or

$$\log E = 1.5 M_w + 4.7 \quad (3.3)$$

where E is expressed in joules.

Of this amount of total energy, only a fraction will arrive at a site away from the earthquake source. Some energy will be dissipated by anelastic material attenuation along ray paths, and further attenuation occurs due to geometric spreading. This phenomenon can be represented by a simplified mathematical form that implies the

energy arriving at a site, $E(r)$, is inversely proportional to the power of the distance, r , between the source and the site:

$$E(r) = b_1 \frac{E}{r^p} \quad (3.4)$$

where p is the attenuation parameter and b_1 is a constant used to represent the correlation.

Different investigators use different values of p in their energy equation. Davis and Berrill (1982) and Berrill and Davis (1985), by assuming the simple spherically expanding wave model, put $p = 2$ for the energy content of the seismic waves arriving at the site. Law et al. (1990), determined the value of p indirectly by assuming that energy attenuation is proportional to earthquake intensity attenuation and is equal to 4.3 for the American west coast (where rock is highly fractured due to interplate tectonic activities) and 3.2 for the eastern Canada (where bedrock is much sounder). The intensity attenuation values used are after the work of Hasegawa et al. (1981). All these values of p can now be re-examined. As with the advance in the knowledge of earthquake characteristics in the past decade, it is now possible to have a better estimation of p by using ground motion models that have been developed recently.

The estimation of the energy arriving at a site can actually start with the very basic theory that energy is directly proportional to the square of the velocity. As a result, by plotting the logarithmic decay of the square of the peak ground velocity for a given region against

the logarithmic distance, for various magnitudes of earthquake, the average slope of the curves will then be equal to the value of p for that region. Figure 3.1 and Figure 3.2 show such plots based on the simple functional form of the ground motion models developed by Atkinson and Boore (1995) (i.e. Equation 2.25) for Eastern North American (ENA) and Atkinson and Silva (2000) (i.e. Equation 2.26) for California respectively. It can be seen from both figures that the values of p are around 2 in both cases, despite regional differences in ground motion attenuation. This can be explained by the fact that the larger attenuation of more highly fractured rock in Western North American is offset by the higher frequency caused by the content of peak ground velocity for ENA. As a result the values of p are about the same for both areas. The fact that higher frequencies decay more rapidly is due to greater energy loss that occurs when high frequency wave is passing through the earth's crust. This is illustrated in the next paragraph.

In reality, as a wave motion passes through a medium, the elastic energy associated with the wave motion is gradually absorbed by the medium, reappearing ultimately in the form of heat. This process is called *absorption* and is responsible for the eventual complete disappearance of the wave motion (Sheriff and Geldart, 1995). The decrease of amplitude due to absorption appears to be exponential with distance for elastic waves in rocks (Figure 3.3). Thus, we can write the equation for the decrease in amplitude because of absorption as $A = A_0 e^{-\mu x}$, where A and A_0 are values of the amplitudes of a plane wavefront at two points a distance x apart, and μ is the absorption coefficient. Experimental evidence (Attewell and Ramana, 1966; Bradley and Fort, 1966; Toksoz and Johnston, 1981) suggests that the absorption coefficient, μ , is approximately proportional

Figure 3.1: Plot $\log v^2$ vs $\log R$ - ENA (data based on Atkinson and Boore, 1995)

Figure 3.2: Plot $\log v^2$ vs $\log R$ - California (data based on Atkinson and Silva, 2000)

to frequency. Higher frequencies are attenuated more than lower frequencies so that the spectrum of a seismic wavelet changes with time (Figure 3.3). Such an increase of absorption with frequency provides one mechanism for the observed loss of high frequencies and the change of waveshape with distance. Toksoz and Johnston (1981) summarized the state of our knowledge about attenuation and absorption. Internal friction in the form of sliding friction (or sticking and sliding) and viscous losses in the interstitial fluids are probably the most important mechanisms, the latter being more important in high-permeability rocks. Other effects, probably of minor significance in general, are the loss when part of the heat generated during the compressive part of the wave is conducted away, piezoelectric and thermoelectric effects, and the energy used to create new surfaces (of importance only near the source). Often, no distinction is made between “attenuation” and “absorption”. Because of difficulties in measuring absorption and also because the quantity of interest is usually the net decrease in wave amplitude, measurements are often made of total attenuation without regard to its cause and the results used to determine a value of μ .

The value of p , thus, can be chosen as equals to 2 for the general use in developing the energy method. The $E(r)$ general equation (i.e. Equation 3.4) can now be expressed specifically as:

$$E(R) = b_2 \frac{E}{R^2} \quad (3.5)$$

where b_2 is a constant and R is the hypocentral or closest distance to the fault in km as

Figure 3.3: Loss of amplitude as a function of one-way traveltime, based on measurements with a geophone clamped in borehole with the source at the surface. The curves labeled “divergence” and “divergence plus transmission loss” are calculated from sonic-log data allowing for loss of energy in transmission through reflecting interfaces. The 20-, 40-, 60-Hz curves show attenuation at those frequencies (after Sheriff and Geldart, 1995)

used in Atkinson's equations.

Substituting Equation 3.3 into the above equation yields

$$E(R) = b_2 \frac{(10^{1.5M_s + 4.7})}{R^2} \quad (3.6a)$$

$$\text{or} \quad E(R) = b_3 \frac{10^{1.5M_s}}{R^2} \quad (3.6b)$$

where b_3 is once again, a constant used to represent the correlation.

In order to turn the energy term arriving at a site into a dimensionless quantity, the idea of “*normalized seismic energy*” is applied by dividing the energy with the vertical effective in-situ stress at the point where liquefaction potential is assessed. This results in a dimensionless $E(R)$,

$$E_n(R) = b_4 \frac{10^{1.5M_s}}{\sigma'_v R^2} \quad (3.7)$$

provided that R is measured in km and b_4 has a dimension of m^{-1} .

The liquefaction potential can now be determined by a graph showing the N-value (SPT blow counts) vs the seismic energy function, S , together with information on whether the

soil liquefied or not at each site and at each depth for different earthquakes, where

$$S = \frac{10^{1.5M_w}}{\sigma'_v R^2} \quad (3.8)$$

The next section describes this procedure by the analysis of recorded observations made during past earthquakes all around the world.

3.3 CASE HISTORIES ANALYSIS

Part of the objective of this research is to compile a large case history database of earthquake induced liquefaction. This is for the purpose of building up a more reliable liquefaction potential assessment method.

A total of 363 independent cases of liquefaction and non-liquefaction have been compiled. Among them, there are 160 liquefaction cases and 203 non-liquefaction cases. These cases are from 30 earthquakes of more than 10 countries over the world. The period of time covering these liquefaction or non-liquefaction events is from 1891 to 1999. For each site the following information was gathered: earthquake magnitude, hypocentral or closest distance to the fault of the site, depth of soil studied, depth of groundwater table, standard penetration resistance, fines content and identification of liquefaction occurrence. Table 3.1 shows all the above information for each liquefaction case.

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
1	Mino-Owari	10/27/91	7.8	30.0	0.8	13.7	17	5	19.40	4.3E+06	1	57
2	Mino-Owari	10/27/91	7.8	30.0	2.0	9.1	10	5	11.80	5.5E+06	1	57
3	Mino-Owari	10/27/91	7.8	30.0	2.4	5.0	13	4	18.00	7.7E+06	1	57
4	Mino-Owari	10/27/91	7.8	51.7	0.9	13.7	17	0	17.83	1.5E+06	1	57
5	Mino-Owari	10/27/91	7.8	51.4	1.8	9.1	10	0	11.96	2.0E+06	1	57
6	Mino-Owari	10/27/91	7.8	51.4	1.8	7.6	19	0	24.64	2.3E+06	0	57
7	Mino-Owari	10/27/91	7.8	51.4	2.4	6.1	16	0	22.09	2.6E+06	1	57
8	San Francisco	8/20/06	7.9	54.0	2.4	7.6	7	15	8.47	2.9E+06	1	115
9	San Francisco	8/20/06	7.9	54.0	5.0	7.0	26	0	18.80	2.3E+06	0	115
10	San Francisco	8/20/06	7.9	54.0	2.0	4.5	5	0	5.10	3.7E+06	1	115
11	San Francisco	8/20/06	7.9	54.0	1.8	2.3	43	0	29.00	5.9E+06	0	115
12	San Francisco	8/20/06	7.9	54.0	3.7	4.6	4	0	3.30	2.9E+06	1	115
13	San Francisco	8/20/06	7.9	54.0	4.1	6.5	14	0	10.20	2.3E+06	0	115
14	San Francisco	8/20/06	7.9	185.7	2.0	7.6	3	0	2.30	2.1E+05	1	115
15	San Francisco	8/20/06	7.9	54.0	1.7	4.6	5	0	4.70	3.9E+06	1	22
16	San Francisco	8/20/06	7.9	54.0	1.7	4.6	14	0	13.20	3.9E+06	0	22
17	Tokyo	6/20/18	7.6	65.0	0.0	4.5	8	15	17.59	1.6E+06	0	48
18	Tokyo	6/20/18	7.6	58.0	0.5	3.5	8	15	18.43	2.3E+06	0	48
19	Kwanto	9/1/23	7.9	72.0	4.0	6.0	1	14	3.35	1.3E+06	1	109
20	Kwanto	9/1/23	7.9	72.0	4.0	4.3	2	22	6.77	1.9E+06	1	109
21	Kwanto	9/1/23	7.9	72.0	1.0	5.0	12	5	17.30	2.4E+06	1	109
22	Kwanto	9/1/23	7.9	102.0	0.0	4.5	8	15	17.59	1.8E+06	0	48
23	Kwanto	9/1/23	7.9	105.0	0.5	3.5	8	15	18.43	1.9E+06	0	48
24	Kwanto	9/1/23	7.9	96.0	1.0	8.0	3	15	6.48	1.0E+06	1	48
25	Kwanto	9/1/23	7.9	77.0	1.0	7.5	4	15	9.00	1.7E+06	1	48
26	Nishi-Saitama	9/21/31	6.6	20.0	2.5	3.5	4	15	8.79	3.7E+05	1	48
27	Nishi-Saitama	9/21/31	6.6	19.0	0.0	4.5	8	5	14.40	5.9E+05	1	48
28	Nishi-Saitama	9/21/31	6.6	34.0	0.5	3.5	8	15	18.43	2.1E+05	0	48
29	Nishi-Saitama	9/21/31	6.6	51.0	1.0	8.0	3	15	6.48	4.1E+04	1	48
30	Nishi-Saitama	9/21/31	6.6	65.0	1.0	7.6	4	15	7.95	2.7E+04	0	48
31	El Centro	5/19/40	7.0	12.8	4.6	4.6	9	0	9.92	2.3E+06	1	85
32	El Centro	5/19/40	7.0	12.8	6.1	7.6	4	0	3.59	1.6E+06	1	85
33	El Centro	5/19/40	7.0	12.8	1.5	6.1	1	0	1.18	2.7E+06	1	85
34	Tonankai	12/7/44	8.0	165.0	2.0	2.4	1	26	5.96	9.7E+05	1	57
35	Tonankai	12/7/44	8.0	165.0	0.5	3.7	1	27	6.74	9.2E+05	1	57
36	Tonankai	12/7/44	8.0	165.0	2.5	3.0	2	30	8.63	4.3E+05	1	57
37	Tonankai	12/7/44	8.0	200.0	2.0	7.0	10	5	12.60	2.9E+05	1	57
38	Tonankai	12/7/44	8.0	163.8	1.5	4.0	4	0	6.76	7.8E+05	1	57
39	Tonankai	12/7/44	8.0	163.8	0.6	2.4	1	15	5.00	1.6E+06	1	57
40	Fukui	6/28/48	7.3	21.0	1.2	3.0	3	0	4.30	5.3E+06	1	57

Table 3.1: Site conditions and earthquake data for liquefaction study

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
41	Fukui	6/28/48	7.3	20.6	0.8	8.0	29	2	40.20	2.9E+06	0	57
42	Fukui	6/28/48	7.3	20.6	0.9	6.0	8	21	17.36	4.0E+06	0	57
43	Fukui	6/28/48	7.3	20.6	1.8	3.0	2	1	3.50	4.4E+06	1	57
44	Fukui	6/28/48	7.3	16.3	3.4	7.0	18	0	22.08	3.7E+06	1	57
45	Fukui	6/28/48	7.3	16.3	0.9	7.0	28	0	40.01	5.0E+06	0	57
46	Fukui	6/28/48	7.3	16.3	1.3	3.1	3	0	5.86	9.3E+06	1	57
47	Fukui	6/28/48	7.3	16.3	0.9	6.1	5	0	7.72	5.8E+06	1	57
48	San Francisco DC	3/22/57	5.3	18.0	2.4	7.6	7	0	5.70	3.2E+03	0	115
49	San Francisco DC	3/22/57	5.3	18.0	5.0	7.0	26	0	18.80	2.6E+03	0	115
50	San Francisco DC	3/22/57	5.3	16.0	2.0	4.5	5	0	5.10	5.4E+03	0	115
51	San Francisco DC	3/22/57	5.3	16.0	1.8	2.3	43	0	29.00	8.5E+03	0	115
52	San Francisco DC	3/22/57	5.3	14.0	3.7	4.6	4	0	3.30	5.5E+03	0	115
53	San Francisco DC	3/22/57	5.3	14.0	4.1	6.5	14	0	10.20	4.3E+03	0	115
54	San Francisco DC	3/22/57	5.3	18.0	1.7	4.6	5	0	4.70	4.4E+03	0	22
55	San Francisco DC	3/22/57	5.3	18.0	1.7	4.6	14	0	13.20	4.4E+03	0	22
56	Chile	5/21/60	9.5	127.9	3.7	4.6	6	0	7.08	1.5E+08	1	93
57	Chile	5/21/60	9.5	127.9	3.7	4.6	8	0	9.44	1.5E+08	1	93
58	Chile	5/21/60	9.5	127.9	3.7	6.1	18	0	19.39	1.3E+08	0	93
59	Alaska	3/27/64	9.2	157.0	0.0	9.9	28	18	25.63	2.6E+07	0	86
60	Alaska	3/27/64	9.2	143.0	0.0	6.1	5	18	8.35	5.1E+07	1	86
61	Alaska	3/27/64	9.2	147.0	0.0	7.6	42	7	36.53	3.8E+07	0	86
62	Alaska	3/27/64	9.2	93.0	0.0	7.0	15	15	16.54	1.0E+08	1	86
63	Alaska	3/27/64	9.2	133.0	0.0	6.0	13	10	18.03	5.9E+07	1	86
64	Alaska	3/27/64	9.2	142.0	2.3	5.6	6	10	7.82	4.0E+07	1	86
65	Alaska	3/27/64	9.2	72.0	2.0	10.0	8	6	7.36	1.0E+08	1	86
66	Alaska	3/27/64	9.2	74.0	4.5	6.0	23	15	25.98	1.1E+08	0	86
67	Alaska	3/27/64	9.2	102.0	0.0	6.1	5	0	6.89	1.2E+08	1	86
68	Alaska	3/27/64	9.2	102.0	2.4	6.1	5	0	5.90	8.4E+07	1	86
69	Alaska	3/27/64	9.2	117.7	0.0	7.6	40	0	47.20	6.3E+07	0	86
70	Alaska	3/27/64	9.2	94.5	6.1	6.1	10	0	13.78	1.3E+08	1	86
71	Alaska	3/27/64	9.2	65.3	1.5	6.1	13	0	16.48	2.4E+08	1	23
72	Niigata	6/16/64	7.5	62.0	0.0	4.3	6	10	10.68	1.1E+06	1	34
73	Niigata	6/16/64	7.5	62.0	9.0	12.5	11	15	10.36	2.2E+05	0	34
74	Niigata	6/16/64	7.5	62.0	5.0	7.0	2	15	4.38	3.9E+05	1	34
75	Niigata	6/16/64	7.5	62.0	1.4	14.0	11	2	9.00	3.0E+05	1	34
76	Niigata	6/16/64	7.5	62.0	1.0	10.0	15	2	14.30	4.2E+05	1	34
77	Niigata	6/16/64	7.5	62.0	1.0	9.0	8	2	8.00	4.6E+05	1	34
78	Niigata	6/16/64	7.5	62.0	1.4	5.0	10	2	12.50	7.2E+05	1	34
79	Niigata	6/16/64	7.5	62.0	2.5	7.0	4	15	6.80	4.9E+05	1	34
80	Niigata	6/16/64	7.5	62.0	2.5	5.0	12	5	13.90	6.2E+05	1	34

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
81	Niigata	6/16/64	7.5	62.0	2.5	5.0	35	15	44.42	6.2E+05	0	34
82	Niigata	6/16/64	7.5	62.0	2.3	8.5	18	15	20.63	4.3E+05	0	34
83	Niigata	6/16/64	7.5	62.0	0.8	10.0	9	15	11.62	4.3E+05	1	34
84	Niigata	6/16/64	7.5	62.0	1.2	6.4	8	15	12.14	6.1E+05	1	34
85	Niigata	6/16/64	7.5	62.0	5.2	10.0	15	15	15.08	3.0E+05	0	34
86	Niigata	6/16/64	7.5	62.0	1.2	9.5	18	15	20.74	4.3E+05	0	34
87	Niigata	6/16/64	7.5	62.0	0.0	2.5	10	15	18.22	1.9E+06	0	34
88	Niigata	6/16/64	7.5	62.0	3.5	9.5	8	15	9.83	3.6E+05	0	34
89	Niigata	6/16/64	7.5	62.0	0.1	9.3	11	15	14.34	4.9E+05	0	34
90	Niigata	6/16/64	7.5	94.0	0.0	9.0	2	15	4.70	2.2E+05	1	34
91	Niigata	6/16/64	7.5	62.0	1.0	15.0	1	15	3.34	2.9E+05	1	34
92	Niigata	6/16/64	7.5	65.2	0.9	6.1	6	0	9.26	7.3E+05	1	54
93	Niigata	6/16/64	7.5	65.2	0.9	7.6	15	0	20.71	5.8E+05	1	90
94	Niigata	6/16/64	7.5	65.2	0.9	6.1	12	0	18.52	7.3E+05	0	90
95	Niigata	6/16/64	7.5	65.2	3.7	7.6	6	0	7.17	4.4E+05	0	90
96	Caracas	7/29/67	6.5	58.3	0.9	0.9	3	0	7.55	1.0E+05	1	93
97	Tokachioki	5/16/68	8.2	291.0	1.0	6.0	10	5	12.00	3.4E+05	1	58
98	Tokachioki	5/16/68	8.2	181.0	1.0	2.0	12	5	16.40	2.0E+06	0	79
99	Tokachioki	5/16/68	8.2	181.0	1.5	2.0	15	5	19.00	1.7E+06	0	79
100	Tokachioki	5/16/68	8.2	181.0	1.0	2.0	15	5	20.50	2.0E+06	0	79
101	Tokachioki	5/16/68	8.2	181.0	1.5	2.5	11	5	14.00	1.7E+06	0	79
102	Tokachioki	5/16/68	8.2	181.0	1.0	2.0	1	5	1.40	2.0E+06	1	79
103	Tokachioki	5/16/68	8.2	178.1	0.9	3.7	14	0	26.47	1.6E+06	0	79
104	Tokachioki	5/16/68	8.2	74.7	0.9	3.7	6	0	11.34	9.3E+06	1	79
105	Tokachioki	5/16/68	8.2	178.1	1.5	3.1	15	0	28.36	1.6E+06	0	79
106	Tokachioki	5/16/68	8.2	162.2	0.9	4.6	6	0	10.15	1.6E+06	1	58
107	Santa Rosa	10/1/69	4.8	80.6	1.7	4.6	5	0	4.70	3.9E+01	0	22
108	Santa Rosa	10/1/69	4.8	80.6	1.7	4.6	14	0	13.20	3.9E+01	0	22
109	San Fernando	2/9/71	6.6	18.8	0.0	13.7	9	20	9.88	1.6E+05	1	92
110	San Fernando	2/9/71	6.6	11.6	4.6	6.1	2	35	7.45	6.2E+05	1	92
111	Haicheng	2/4/75	7.0	56.6	1.5	8.2	11	9	11.74	1.1E+05	1	98
112	Haicheng	2/4/75	7.0	69.0	1.5	9.1	8	15	9.42	6.4E+04	1	98
113	Haicheng	2/4/75	7.0	92.0	1.5	8.2	9	74	15.80	4.0E+04	0	98
114	Guatemala	2/4/76	7.4	170.0	1.5	8.8	3	5	2.30	4.2E+04	1	94
115	Guatemala	2/4/76	7.4	170.0	2.6	5.8	7	5	5.70	5.2E+04	1	94
116	Tangshan	7/27/76	7.6	118.0	3.5	7.6	16	27	21.32	1.6E+05	0	112
117	Tangshan	7/27/76	7.6	121.0	3.3	6.5	12	12	14.04	1.8E+05	0	112
118	Tangshan	7/27/76	7.6	121.0	2.0	5.0	10	9	12.76	2.5E+05	0	112
119	Tangshan	7/27/76	7.6	119.0	3.3	6.0	5	9	5.85	1.9E+05	1	112
120	Tangshan	7/27/76	7.6	121.0	3.0	5.0	9	5	10.10	2.1E+05	1	112

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
121	Tangshan	7/27/76	7.6	119.0	3.2	6.0	8	5	8.30	1.9E+05	1	112
122	Tangshan	7/27/76	7.6	121.0	1.3	9.0	23	43	32.24	1.7E+05	0	112
123	Tangshan	7/27/76	7.6	30.0	3.1	7.5	49	15	52.39	2.6E+06	0	112
124	Tangshan	7/27/76	7.6	26.5	1.8	13.5	20	17	20.19	2.3E+06	0	112
125	Tangshan	7/27/76	7.6	23.6	5.2	6.2	56	15	57.42	4.0E+06	0	112
126	Tangshan	7/27/76	7.6	15.0	3.7	4.5	9	59	16.88	1.4E+07	1	112
127	Tangshan	7/27/76	7.6	16.0	7.3	8.0	40	15	36.35	6.4E+06	0	112
128	Tangshan	7/27/76	7.6	16.1	4.3	6.0	25	5	24.60	9.4E+06	0	112
129	Tangshan	7/27/76	7.6	15.4	4.5	6.0	15	5	14.60	1.0E+07	0	112
130	Tangshan	7/27/76	7.6	15.0	3.0	4.5	12	42	21.68	1.5E+07	0	112
131	Tangshan	7/27/76	7.6	12.1	3.8	6.5	14	15	16.96	1.7E+07	0	112
132	Tangshan	7/27/76	7.6	11.2	2.1	4.8	11	37	20.84	2.9E+07	0	112
133	Tangshan	7/27/76	7.6	16.1	1.5	4.5	22	16	32.70	1.6E+07	0	112
134	Tangshan	7/27/76	7.6	18.8	1.9	3.5	12	12	18.37	1.3E+07	1	112
135	Tangshan	7/27/76	7.6	18.6	1.5	6.5	17	7	19.30	9.1E+06	1	112
136	Tangshan	7/27/76	7.6	14.6	3.0	6.5	12	1	12.30	1.2E+07	1	112
137	Tangshan	7/27/76	7.6	15.2	2.2	5.8	3	4	3.40	1.4E+07	1	112
138	Tangshan	7/27/76	7.6	15.9	1.5	9.0	9	3	8.80	9.5E+06	1	112
139	Tangshan	7/27/76	7.6	17.2	0.9	3.5	14	9	22.02	1.9E+07	1	112
140	Tangshan	7/27/76	7.6	18.8	1.6	6.5	8	7	9.10	8.8E+06	1	112
141	Tangshan	7/27/76	7.6	23.6	1.3	3.5	10	10	15.39	9.4E+06	1	112
142	Tangshan	7/27/76	7.6	26.1	1.2	11.0	16	4	14.50	3.0E+06	1	112
143	Tangshan	7/27/76	7.6	20.7	0.6	4.0	4	2	5.90	1.3E+07	1	112
144	Tangshan	7/27/76	7.6	17.0	1.1	6.0	18	5	21.40	1.2E+07	1	112
145	Tangshan	7/27/76	7.6	45.2	0.7	8.5	15	5	15.60	1.3E+06	1	112
146	Tangshan	7/27/76	7.6	41.4	0.7	8.5	12	5	12.50	1.6E+06	1	112
147	Tangshan	7/27/76	7.6	45.2	0.8	9.5	11	5	10.80	1.2E+06	1	112
148	Tangshan	7/27/76	7.6	49.0	0.7	11.0	15	67	21.68	8.9E+05	0	112
149	Tangshan	7/27/76	7.6	52.3	1.0	5.0	14	23	23.97	1.5E+06	0	112
150	Tangshan	7/27/76	7.6	49.0	1.8	3.0	7	6	10.14	2.2E+06	1	112
151	Tangshan	7/27/76	7.6	43.3	2.1	3.0	6	7	8.59	2.6E+06	1	112
152	Tangshan	7/27/76	7.6	34.9	3.7	6.0	14	5	14.20	2.1E+06	1	112
153	Tangshan	7/27/76	7.6	47.1	4.5	10.0	18	5	14.90	7.8E+05	1	112
154	Tangshan	7/27/76	7.6	47.1	3.5	4.0	31	22	43.10	1.5E+06	0	112
155	Tangshan	7/27/76	7.6	49.0	4.5	7.2	18	19	21.36	8.9E+05	0	112
156	Tangshan	7/27/76	7.6	39.6	2.8	3.5	16	17	24.53	2.5E+06	0	112
157	Tangshan	7/27/76	7.6	47.1	1.1	5.0	6	20	11.99	1.9E+06	1	112
158	Tangshan	7/27/76	7.6	42.4	3.6	9.5	3	15	5.22	1.1E+06	1	112
159	Tangshan	7/27/76	7.6	41.2	1.1	5.5	21	8	26.33	2.2E+06	0	112
160	Tangshan	7/27/76	7.6	47.1	3.1	6.5	14	53	22.16	1.2E+06	0	112

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
161	Tangshan	7/27/76	7.6	42.4	1.4	4.0	1	16	4.19	2.6E+06	1	112
162	Tangshan	7/27/76	7.6	72.4	0.9	2.0	2	19	7.40	1.7E+06	1	112
163	Tangshan	7/27/76	7.6	66.5	2.0	8.7	10	9	10.42	5.3E+05	0	112
164	Tangshan	7/27/76	7.6	63.6	1.6	3.5	14	2	19.60	1.2E+06	0	112
165	Tangshan	7/27/76	7.6	71.4	1.1	4.2	5	2	6.90	9.3E+05	1	112
166	Tangshan	7/27/76	7.6	94.6	0.6	5.0	7	16	12.67	5.0E+05	1	112
167	Tangshan	7/27/76	7.6	93.1	0.7	3.5	1	15	4.07	6.9E+05	1	112
168	Tangshan	7/27/76	7.6	92.1	0.7	5.0	10	2	13.20	5.2E+05	0	112
169	Tangshan	7/27/76	7.6	66.3	2.3	3.0	4	11	6.78	1.1E+06	1	112
170	Tangshan	7/27/76	7.6	71.4	2.3	9.5	17	7	15.78	4.2E+05	0	112
171	Tangshan	7/27/76	7.6	59.7	2.5	4.3	9	3	10.90	1.0E+06	0	112
172	Tangshan	7/27/76	7.6	68.0	2.3	3.5	9	18	15.93	9.4E+05	1	112
173	Tangshan	7/27/76	7.6	66.3	2.3	7.0	15	2	15.60	6.1E+05	1	112
174	Tangshan	7/27/76	7.6	67.5	2.9	6.5	11	3	11.30	5.9E+05	1	112
175	Tangshan	7/27/76	7.6	81.2	1.4	4.0	19	18	30.85	7.1E+05	0	112
176	Tangshan	7/27/76	7.6	84.5	1.2	5.0	14	13	20.34	5.7E+05	0	112
177	Tangshan	7/27/76	7.6	80.0	0.8	6.0	5	48	12.32	5.8E+05	1	112
178	Tangshan	7/27/76	7.6	83.2	1.4	6.5	8	17	12.55	4.6E+05	1	112
179	Tangshan	7/27/76	7.6	86.1	1.2	5.0	8	15	13.19	5.5E+05	1	112
180	Tangshan	7/27/76	7.6	78.7	0.8	4.5	16	12	24.25	7.7E+05	0	112
181	Tangshan	7/27/76	7.6	83.6	1.0	4.5	24	15	36.46	6.5E+05	0	112
182	Tangshan	7/27/76	7.6	78.3	0.5	2.0	3	4	6.00	1.6E+06	1	112
183	Tangshan	7/27/76	7.6	87.1	1.6	2.3	9	15	17.59	8.5E+05	0	112
184	Tangshan	7/27/76	7.6	22.5	2.1	3.6	19	15	28.91	8.7E+06	0	112
185	IzuOhshima kinki	1/14/78	6.7	40.3	2.0	7.0	1	94	6.32	7.7E+04	1	45
186	IzuOhshima kinki	1/14/78	6.7	40.3	2.0	6.0	1	94	6.20	8.6E+04	1	45
187	Miyagiken-Oki I	2/20/78	6.4	131.0	0.0	3.0	13	0	20.00	7.7E+03	0	48
188	Miyagiken-Oki I	2/20/78	6.4	130.0	0.0	4.0	8	0	14.20	5.9E+03	0	48
189	Miyagiken-Oki I	2/20/78	6.4	130.0	1.8	5.0	2	60	8.24	3.5E+03	0	48
190	Miyagiken-Oki I	2/20/78	6.4	130.0	0.9	3.3	4	0	5.20	5.6E+03	0	48
191	Miyagiken-Oki I	2/20/78	6.4	130.0	1.7	4.3	4	10	6.90	3.9E+03	0	48
192	Miyagiken-Oki I	2/20/78	6.4	130.0	0.3	4.0	8	7	13.94	5.5E+03	0	48
193	Miyagiken-Oki I	2/20/78	6.4	130.0	0.9	5.0	3	15	7.11	4.0E+03	0	48
194	Miyagiken-Oki I	2/20/78	6.4	98.0	4.3	6.3	9	5	9.80	3.9E+03	0	48
195	Miyagiken-Oki I	2/20/78	6.4	98.0	2.4	6.3	8	4	9.60	4.8E+03	0	48
196	Miyagiken-Oki I	2/20/78	6.4	96.0	0.0	5.0	4	15	9.10	8.6E+03	0	48
197	Miyagiken-Oki I	2/20/78	6.4	96.0	0.0	6.0	3	15	7.00	7.2E+03	0	48
198	Miyagiken-Oki I	2/20/78	6.4	130.0	2.7	10.0	1	15	3.02	1.9E+03	0	48
199	Miyagiken-Oki I	2/20/78	6.4	96.0	0.0	9.3	14	15	19.58	4.6E+03	0	48
200	Miyagiken-Oki I	2/20/78	6.4	130.0	0.5	3.3	5	5	9.20	6.4E+03	0	48

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
201	Miyagiken-Okii 1	2/20/78	6.4	130.0	1.3	3.3	7	4	12.00	5.5E+03	0	48
202	Miyagiken-Okii 1	2/20/78	6.4	98.0	2.5	8.0	3	15	5.96	3.9E+03	0	48
203	Miyagiken-Okii 1	2/20/78	6.4	130.0	0.9	3.3	18	4	32.00	5.9E+03	0	48
204	Miyagiken-Okii 1	2/20/78	6.4	130.0	0.9	3.3	8	15	13.61	5.9E+03	0	48
205	Miyagiken-Okii 1	2/20/78	6.4	130.0	2.2	5.3	19	0	27.00	3.3E+03	0	48
206	Miyagiken-Okii 1	2/20/78	6.4	96.0	1.5	8.0	8	15	12.04	4.5E+03	0	48
207	Miyagiken-Okii 1	2/20/78	6.4	130.0	2.5	7.2	15	15	20.32	2.4E+03	0	48
208	Miyagiken-Okii 1	2/20/78	6.4	131.0	4.0	7.3	11	15	14.55	2.1E+03	0	48
209	Miyagiken-Okii 1	2/20/78	6.4	131.0	4.3	7.3	15	15	18.95	2.0E+03	0	48
210	Miyagiken-Okii 1	2/20/78	6.4	131.0	3.4	6.3	21	15	27.55	2.4E+03	0	48
211	Miyagiken-Okii 1	2/20/78	6.4	130.0	8.0	10.0	5	15	6.80	1.3E+03	0	48
212	Miyagiken-Okii 1	2/20/78	6.4	129.0	0.8	6.3	25	15	40.02	3.9E+03	0	48
213	Miyagiken-Okii 1	2/20/78	6.4	98.0	0.0	5.0	15	15	26.71	8.3E+03	0	48
214	Miyagiken-Okii 1	2/20/78	6.4	98.0	0.0	4.0	15	15	30.27	1.0E+04	0	48
215	Miyagiken-Okii 1	2/20/78	6.4	98.0	3.4	5.3	13	15	19.90	5.4E+03	0	48
216	Miyagiken-Okii 2	6/12/78	7.5	114.0	0.0	4.0	8	0	14.20	3.4E+05	1	48
217	Miyagiken-Okii 2	6/12/78	7.5	113.0	1.8	5.0	2	60	8.24	2.0E+05	1	48
218	Miyagiken-Okii 2	6/12/78	7.5	113.0	0.9	3.3	4	0	5.20	3.3E+05	1	48
219	Miyagiken-Okii 2	6/12/78	7.5	112.0	1.7	4.3	4	10	6.90	2.4E+05	1	48
220	Miyagiken-Okii 2	6/12/78	7.5	112.0	0.3	4.0	8	7	13.94	3.3E+05	1	48
221	Miyagiken-Okii 2	6/12/78	7.5	110.0	0.9	5.0	3	15	7.11	2.5E+05	1	48
222	Miyagiken-Okii 2	6/12/78	7.5	96.0	4.3	6.3	9	5	9.80	1.8E+05	1	48
223	Miyagiken-Okii 2	6/12/78	7.5	95.0	2.4	6.3	8	4	9.60	2.3E+05	1	48
224	Miyagiken-Okii 2	6/12/78	7.5	85.0	0.0	5.0	4	15	9.10	4.9E+05	1	48
225	Miyagiken-Okii 2	6/12/78	7.5	85.0	0.0	6.0	3	15	7.00	4.1E+05	1	48
226	Miyagiken-Okii 2	6/12/78	7.5	111.0	2.7	10.0	1	15	3.02	1.1E+05	1	48
227	Miyagiken-Okii 2	6/12/78	7.5	114.0	0.5	3.3	5	5	9.20	3.7E+05	1	48
228	Miyagiken-Okii 2	6/12/78	7.5	114.0	1.3	3.3	7	4	12.00	3.2E+05	1	48
229	Miyagiken-Okii 2	6/12/78	7.5	100.0	2.5	8.0	3	15	5.96	1.7E+05	1	48
230	Miyagiken-Okii 2	6/12/78	7.5	114.0	0.9	3.3	18	4	32.00	3.4E+05	0	48
231	Miyagiken-Okii 2	6/12/78	7.5	114.0	0.9	3.3	8	15	13.61	3.4E+05	0	48
232	Miyagiken-Okii 2	6/12/78	7.5	113.0	2.2	5.3	19	0	27.00	1.9E+05	0	48
233	Miyagiken-Okii 2	6/12/78	7.5	85.0	1.5	8.0	8	15	12.04	2.6E+05	0	48
234	Miyagiken-Okii 2	6/12/78	7.5	112.0	2.5	7.2	15	15	20.32	1.5E+05	0	48
235	Miyagiken-Okii 2	6/12/78	7.5	118.0	4.0	7.3	11	15	14.55	1.1E+05	0	48
236	Miyagiken-Okii 2	6/12/78	7.5	118.0	4.3	7.3	15	15	18.95	1.1E+05	0	48
237	Miyagiken-Okii 2	6/12/78	7.5	118.0	3.4	6.3	21	15	27.55	1.3E+05	0	48
238	Miyagiken-Okii 2	6/12/78	7.5	109.0	8.0	10.0	5	15	6.80	8.3E+04	0	48
239	Miyagiken-Okii 2	6/12/78	7.5	107.0	0.8	6.3	25	15	40.02	2.5E+05	0	48
240	Miyagiken-Okii 2	6/12/78	7.5	95.0	0.0	5.0	15	15	26.71	3.9E+05	0	48

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
241	Miyagiken-Oki 2	6/12/78	7.5	95.0	0.0	4.0	15	15	30.27	4.9E+05	0	48
242	Miyagiken-Oki 2	6/12/78	7.5	100.0	3.4	5.3	13	15	19.90	2.3E+05	0	48
243	Guerrero	6/20/78	6.4	50.0	3.6	5.5	20	8	16.20	1.7E+04	0	50
244	Montenegro	4/15/79	6.9	51.0	1.0	7.0	6	10	6.59	1.3E+05	1	106
245	Imperial Valley	10/16/79	6.7	14.0	1.8	3.7	28	25	50.23	1.1E+06	0	13
246	Imperial Valley	10/16/79	6.7	14.0	1.8	3.7	1	29	6.13	1.1E+06	1	13
247	Imperial Valley	10/16/79	6.7	14.0	1.8	4.3	13	37	26.60	1.0E+06	0	13
248	Imperial Valley	10/16/79	6.7	43.0	0.2	1.8	3	80	11.24	2.9E+05	1	13
249	Imperial Valley	10/16/79	6.7	51.0	1.2	3.8	5	17	11.49	1.0E+05	0	13
250	Imperial Valley	10/16/79	6.7	56.0	2.7	4.0	15	13	23.14	6.1E+04	0	12
251	Imperial Valley	10/16/79	6.7	61.0	2.5	4.0	3	91	9.80	5.4E+04	0	12
252	Imperial Valley	10/16/79	6.7	61.0	2.5	4.0	9	62	19.04	5.1E+04	0	12
253	Imperial Valley	10/16/79	6.7	47.0	2.0	3.5	2	64	8.60	1.0E+05	1	12
254	Imperial Valley	10/16/79	6.7	47.0	2.0	2.3	11	18	18.16	1.3E+05	0	12
255	Imperial Valley	10/16/79	6.7	28.0	1.5	2.3	7	24	14.81	4.3E+05	1	12
256	Imperial Valley	10/16/79	6.7	28.0	1.5	2.3	9	35	19.88	4.3E+05	0	12
257	Imperial Valley	10/16/79	6.7	54.0	4.1	5.0	15	35	26.00	4.8E+04	0	12
258	Imperial Valley	10/16/79	6.7	61.0	2.7	4.3	9	35	19.40	4.9E+04	0	12
259	Mexicali Valley	6/9/80	6.4	38.0	2.0	5.0	5	15	8.16	5.7E+04	1	27
260	Westmoreland	4/26/81	6.0	50.5	1.8	3.7	28	25	50.23	7.7E+03	0	13
261	Westmoreland	4/26/81	6.0	50.5	1.8	3.7	1	29	6.13	7.7E+03	0	13
262	Westmoreland	4/26/81	6.0	50.5	1.8	4.3	13	37	26.60	6.9E+03	0	13
263	Westmoreland	4/26/81	6.0	16.6	0.2	1.8	3	80	11.24	1.7E+05	0	13
264	Westmoreland	4/26/81	6.0	16.6	0.2	5.8	14	18	18.48	6.0E+04	0	13
265	Westmoreland	4/26/81	6.0	10.2	1.2	3.8	5	17	11.49	2.3E+05	1	12
266	Westmoreland	4/26/81	6.0	7.6	2.5	4.0	3	91	9.80	3.1E+05	1	12
267	Westmoreland	4/26/81	6.0	7.6	2.5	4.0	9	5	11.70	2.9E+05	1	12
268	Westmoreland	4/26/81	6.0	15.7	2.0	3.5	2	64	8.60	8.1E+04	1	12
269	Westmoreland	4/26/81	6.0	15.7	2.0	2.3	11	18	18.16	1.0E+05	0	12
270	Westmoreland	4/26/81	6.0	29.0	1.5	2.3	7	24	14.81	3.6E+04	0	12
271	Westmoreland	4/26/81	6.0	29.0	1.5	2.3	9	15	15.49	3.6E+04	0	12
272	Westmoreland	4/26/81	6.0	14.4	4.1	5.0	15	15	20.84	6.0E+04	0	12
273	Westmoreland	4/26/81	6.0	8.0	2.7	4.3	9	15	15.08	2.6E+05	0	12
274	Loma Prieta	10/17/89	6.9	89.8	3.5	8.0	2	35	7.32	2.6E+04	1	71
275	Loma Prieta	10/17/89	6.9	89.8	3.5	4.0	2	35	7.91	4.1E+04	1	71
276	Loma Prieta	10/17/89	6.9	89.8	3.5	7.0	2	33	7.28	2.9E+04	1	71
277	Loma Prieta	10/17/89	6.9	89.8	3.5	5.0	2	33	7.56	3.6E+04	1	71
278	Loma Prieta	10/17/89	6.9	89.8	3.5	7.0	2	33	7.28	2.9E+04	1	71
279	Loma Prieta	10/17/89	6.9	89.8	3.5	5.0	2	33	7.56	3.6E+04	1	71
280	Loma Prieta	10/17/89	6.9	74.1	3.0	7.0	8	11	9.60	4.4E+04	1	71

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
281	Loma Prieta	10/17/89	6.9	74.1	3.0	5.0	8	12	11.29	5.5E+04	1	71
282	Loma Prieta	10/17/89	6.9	74.1	3.0	13.9	16	5	12.66	2.6E+04	0	71
283	Loma Prieta	10/17/89	6.9	74.1	3.0	9.3	17	5	15.85	3.5E+04	0	71
284	Loma Prieta	10/17/89	6.9	74.1	3.0	8.4	18	9	18.31	3.8E+04	0	71
285	Loma Prieta	10/17/89	6.9	74.1	3.0	13.0	26	5	21.17	2.7E+04	0	71
286	Loma Prieta	10/17/89	6.9	74.1	3.0	10.7	28	5	24.69	3.2E+04	0	71
287	Loma Prieta	10/17/89	6.9	74.1	3.0	8.8	29	9	28.65	3.7E+04	0	71
288	Loma Prieta	10/17/89	6.9	74.1	3.0	4.3	24	10	30.91	6.1E+04	0	71
289	Loma Prieta	10/17/89	6.9	74.1	3.0	7.9	31	9	31.85	4.0E+04	0	71
290	Loma Prieta	10/17/89	6.9	74.1	3.0	11.6	41	5	34.99	3.0E+04	0	71
291	Loma Prieta	10/17/89	6.9	74.1	3.0	10.2	41	5	36.85	3.3E+04	0	71
292	Loma Prieta	10/17/89	6.9	74.1	3.0	9.8	42	5	38.36	3.4E+04	0	71
293	Loma Prieta	10/17/89	6.9	74.1	3.0	11.1	58	5	50.39	3.1E+04	0	71
294	Loma Prieta	10/17/89	6.9	74.1	3.0	13.4	71	5	57.07	2.6E+04	0	71
295	Loma Prieta	10/17/89	6.9	74.1	3.0	3.8	48	3	61.08	6.6E+04	0	71
296	Loma Prieta	10/17/89	6.9	74.1	3.0	12.5	89	5	73.64	2.8E+04	0	71
297	Loma Prieta	10/17/89	6.9	74.1	3.0	6.0	7	6	7.74	4.9E+04	1	71
298	Loma Prieta	10/17/89	6.9	74.1	3.0	8.5	8	6	7.80	3.8E+04	1	71
299	Loma Prieta	10/17/89	6.9	74.1	3.0	11.1	17	9	15.27	3.1E+04	0	71
300	Loma Prieta	10/17/89	6.9	74.1	3.0	10.2	18	9	17.15	3.3E+04	0	71
301	Loma Prieta	10/17/89	6.9	74.1	3.0	5.6	17	12	21.55	5.1E+04	0	71
302	Loma Prieta	10/17/89	6.9	74.1	3.0	10.7	27	9	24.90	3.2E+04	0	71
303	Loma Prieta	10/17/89	6.9	74.1	3.0	5.2	20	12	24.91	5.4E+04	0	71
304	Loma Prieta	10/17/89	6.9	74.1	3.0	9.8	44	9	40.96	3.4E+04	0	71
305	Loma Prieta	10/17/89	6.9	74.1	3.0	4.3	35	12	46.11	6.1E+04	0	71
306	Loma Prieta	10/17/89	6.9	74.1	3.0	4.7	50	12	62.99	5.8E+04	0	71
307	Loma Prieta	10/17/89	6.9	74.1	3.0	3.8	50	20	72.30	6.6E+04	0	71
308	Loma Prieta	10/17/89	6.9	74.1	3.0	3.4	50	20	74.56	7.0E+04	0	71
309	Loma Prieta	10/17/89	6.9	71.2	3.0	7.5	10	6	10.18	4.5E+04	1	71
310	Loma Prieta	10/17/89	6.9	71.2	3.0	8.8	12	5	11.43	4.0E+04	0	71
311	Loma Prieta	10/17/89	6.9	71.2	3.0	5.0	10	5	11.72	6.0E+04	1	71
312	Loma Prieta	10/17/89	6.9	71.2	3.0	8.3	14	5	13.64	4.2E+04	0	71
313	Loma Prieta	10/17/89	6.9	71.2	3.0	3.0	10	9	14.40	8.2E+04	0	71
314	Loma Prieta	10/17/89	6.9	71.2	3.0	9.3	19	8	18.27	3.9E+04	0	71
315	Loma Prieta	10/17/89	6.9	71.2	3.0	10.4	21	8	19.30	3.5E+04	0	71
316	Loma Prieta	10/17/89	6.9	71.2	3.0	3.8	17	9	22.56	7.1E+04	0	71
317	Loma Prieta	10/17/89	6.9	71.2	3.0	3.4	19	9	26.06	7.7E+04	0	71
318	Loma Prieta	10/17/89	6.9	71.2	3.0	4.7	23	2	27.40	6.3E+04	0	71
319	Loma Prieta	10/17/89	6.9	71.2	3.0	4.3	26	2	31.98	6.7E+04	0	71
320	Loma Prieta	10/17/89	6.9	71.2	3.0	9.8	45	8	41.92	3.7E+04	0	71

Table 3.1: Continued

No.	Earthquake	Date	M _w	R (km)	Depth of water table (m)	Depth of sand (m)	N- value	FC (%)	(N _{1,60}) _{cs}	S	Liquefied (?)	Ref. No.
321	Loma Prieta	10/17/89	6.9	71.2	3.0	8.0	10	4	9.88	4.3E+04	1	71
322	Loma Prieta	10/17/89	6.9	71.2	3.0	5.3	10	5	11.49	5.8E+04	1	71
323	Loma Prieta	10/17/89	6.9	71.2	3.0	4.8	19	5	22.56	6.2E+04	0	71
324	Loma Prieta	10/17/89	6.9	71.2	3.0	5.3	25	2	28.71	5.8E+04	0	71
325	Loma Prieta	10/17/89	6.9	71.2	3.0	4.4	34	5	41.51	6.6E+04	0	71
326	Loma Prieta	10/17/89	6.9	71.2	3.0	9.4	47	6	43.96	3.8E+04	0	71
327	Loma Prieta	10/17/89	6.9	71.2	3.0	3.5	37	5	48.43	7.6E+04	0	71
328	Loma Prieta	10/17/89	6.9	71.2	3.0	8.5	51	6	49.61	4.1E+04	0	71
329	Loma Prieta	10/17/89	6.9	58.7	3.5	5.0	1	32	6.19	8.4E+04	1	71
330	Loma Prieta	10/17/89	6.9	58.7	3.5	8.0	2	23	6.25	6.1E+04	1	71
331	Loma Prieta	10/17/89	6.9	58.7	5.5	6.7	36	12	36.94	5.9E+04	0	71
332	Loma Prieta	10/17/89	6.9	58.7	5.5	6.2	38	12	39.67	6.1E+04	0	71
333	Loma Prieta	10/17/89	6.9	58.7	5.5	7.6	48	10	45.76	5.4E+04	0	71
334	Loma Prieta	10/17/89	6.9	58.7	5.5	7.1	50	10	48.71	5.7E+04	0	71
335	Loma Prieta	10/17/89	6.9	58.7	5.5	5.8	52	11	54.21	6.4E+04	0	71
336	Kobe	1/17/95	6.9	24.6	3.9	6.2	5	30	11.44	4.0E+05	1	108
337	Kobe	1/17/95	6.9	24.6	3.9	8.1	11	30	18.21	3.3E+05	0	108
338	Kobe	1/17/95	6.9	24.6	3.9	9.3	14	30	21.04	3.0E+05	0	108
339	Kobe	1/17/95	6.9	24.6	3.9	10.3	15	30	21.55	2.8E+05	0	108
340	Kobe	1/17/95	6.9	24.6	3.9	8.1	3	40	8.83	3.3E+05	1	108
341	Kobe	1/17/95	6.9	24.6	3.9	12.0	8	40	13.81	2.5E+05	0	108
342	Kobe	1/17/95	6.9	28.8	4.0	10.6	5	35	10.78	2.0E+05	1	108
343	Kobe	1/17/95	6.9	28.8	4.0	11.4	12	35	18.46	1.9E+05	0	108
344	Kobe	1/17/95	6.9	28.8	4.0	12.3	13	35	19.17	1.8E+05	0	108
345	Kobe	1/17/95	6.9	29.7	4.0	10.9	4	35	9.56	1.8E+05	1	108
346	Kobe	1/17/95	6.9	29.7	4.0	14.4	10	35	15.24	1.5E+05	0	108
347	Kobe	1/17/95	6.9	20.7	1.2	4.1	6	15	12.45	1.0E+06	1	108
348	Chi-Chi	9/21/99	7.6	16.6	0.6	5.3	12	13	18.37	1.6E+07	1	78
349	Chi-Chi	9/21/99	7.6	16.6	0.6	10.8	40	18	43.85	8.3E+06	0	78
350	Chi-Chi	9/21/99	7.6	16.6	0.6	12.1	36	18	37.88	7.5E+06	0	78
351	Chi-Chi	9/21/99	7.6	16.6	0.6	13.7	40	18	39.58	6.7E+06	0	78
352	Chi-Chi	9/21/99	7.6	16.6	1.5	5.8	11	34	20.73	1.3E+07	1	78
353	Chi-Chi	9/21/99	7.6	28.9	1.0	4.8	6	49	14.74	5.5E+06	1	78
354	Chi-Chi	9/21/99	7.6	28.9	0.6	2.8	6	49	17.78	9.5E+06	1	78
355	Chi-Chi	9/21/99	7.6	28.9	0.6	13.4	16	49	21.56	2.2E+06	0	78
356	Chi-Chi	9/21/99	7.6	28.4	2.2	5.9	5	43	11.90	4.1E+06	1	78
357	Chi-Chi	9/21/99	7.6	28.4	2.0	5.7	4	43	10.66	4.3E+06	1	78
358	Chi-Chi	9/21/99	7.6	28.4	2.0	11.8	21	24	24.52	2.4E+06	0	78
359	Chi-Chi	9/21/99	7.6	28.4	2.0	13.3	24	24	26.22	2.1E+06	0	78
360	Chi-Chi	9/21/99	7.6	28.4	2.0	14.9	27	24	27.78	1.9E+06	0	78
361	Chi-Chi	9/21/99	7.6	24.0	1.2	4.3	6	28	14.08	8.4E+06	1	78
362	Chi-Chi	9/21/99	7.6	24.0	1.2	5.8	49	42	77.53	6.6E+06	0	78
363	Chi-Chi	9/21/99	7.6	24.0	1.2	8.8	38	42	51.69	4.6E+06	0	78

Table 3.1: Continued

3.3.1 Corrections for resistance from standard penetration test (SPT)

The standard penetration resistance, N , is the number of blows by a hammer of a standard weight falling a standard height required to drive a standard split spoon sampler 0.3 m into the ground. Unfortunately, the measurement of SPT N -value is subjected to large variation due to the different equipment used, the method of hammering and the size of cased hole in performing the test.

Data compatibility is considered to be critical for the purpose of empirical analysis which is usually the primary application of a database. Therefore, effort has been put into correction or normalization among different cases to make the different type of SPT measurements compatible. There are two main kinds of correction or normalization made to the N -value obtained directly from the SPT. The first is the correction for overburden:

$$N_1 = C_N \times N \quad (3.9)$$

and

$$C_N = 10 \times \sqrt{\frac{1}{\sigma'_v}} \quad (3.10)$$

where C_N = the overburden correction factor according to Liao and Whiteman
(1986)

N = the SPT blows

$$\sigma'_v = \text{the effective vertical stress in kN/m}^2$$

The overburden correction factor by Liao and Whiteman (1986) is adopted here due to its commonly use and an explicit equation is given rather than just a chart (Figure 2.1) as has been given by Seed and DeAlba (1986).

The second correction is to account for the effects of using different sampling equipment and/or practices in performing the SPT:

$$N_{1.60} = C_E \times C_N \times N \quad (3.11)$$

and

$$C_E = \frac{ER}{60} \times C_{ROD} \times C_{LIN} \quad (3.12)$$

where ER is the energy ratio defined as the percent of the theoretical free-fall energy transmitted to the rods from the SPT hammer, estimated in Table 3.2 after Seed et al. (1985), $C_{ROD} = 0.75$ (Seed et al., 1985) for depth of SPT less than 3.0 m, is a correction for the effects of short rod lengths when performing the SPT at shallow depths, $C_{LIN} = 1.1$ to 1.2 (Seed et al., 1985) when tests are performed using a sampler without liners for loose sands, $= 1.2$ for dense sand. For conditions other than that specified above, take C_{ROD} and C_{LIN} equal to 1.0.

For Japanese data, an extra correction, $C_{JAP} = 0.9$ (Seed et al., 1985), has to be used for sands with $N_{1.60} < 20$ to account for different standards of drilling practice in Japan (the

Country	Hammer Type	Hammer release	Estimated rod energy (%)	Correction factor for 60% rod energy
Japan	Donut	Free-fall	78	$78/60 = 1.30$
	Donut	Rope and pulley with special throw release	67	$67/60 = 1.12$
United States	Safety	Rope and pulley	60	$60/60 = 1.00$
	Donut	Rope and pulley	45	$45/60 = 0.75$
Argentina	Donut	Rope and pulley	45	$45/60 = 0.75$
China	Donut	Free-fall	60	$60/60 = 1.00$
	Donut	Rope and pulley	50	$50/60 = 0.83$

Table 3.2: Summary of energy ratios for SPT procedures (after Seed et al., 1985)

frequency of hammer drop and the diameter of the borehole).

3.3.2 Correction for fines content (FC)

It has been recognized for a long time that for a given SPT N-value, sands containing more than 5% fines tend to exhibit greater resistance to liquefaction than clean sands. This increase in liquefaction resistance is a consequence of the general increase of compressibility and decrease of permeability with increased fines content.

The conventional way to deal with this is to develop liquefaction resistance curves for various fines contents of $FC \leq 5\%$, 15% and 35% respectively. The correction for fines content can now be treated in a more systematic way by applying the correction factors as suggested in the guidelines presented in the NCEER Workshop on Evaluation of

Liquefaction Resistance (NCEER, 1997). Consensus was gained in the Workshop that the correction for fines content should be a function of penetration resistance as well as fines content and the following equations were developed for correcting standard penetration resistance determined for silty sands to an equivalent clean sand penetration resistance:

$$(N_{1,60})_{cs} = \alpha + \lambda N_{1,60} \quad (3.13)$$

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

$$\alpha = 0 \quad \text{for } FC \leq 5\% \quad (3.14a)$$

$$\alpha = \exp[1.76 - (190/FC^2)] \quad \text{for } 5\% < FC < 35\% \quad (3.14b)$$

$$\alpha = 5.0 \quad \text{for } FC \geq 35\% \quad (3.14c)$$

$$\lambda = 1.0 \quad \text{for } FC \leq 5\% \quad (3.15a)$$

$$\lambda = [0.99 + (FC^{1.5}/1000)] \quad \text{for } 5\% < FC < 35\% \quad (3.15b)$$

$$\lambda = 1.2 \quad \text{for } FC \geq 35\% \quad (3.15c)$$

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

It is to be noted that the correction for fines content is defined so that sands containing fines with a penetration resistance of $N_{1,60}$ would have the same liquefaction resistance strength as that of the clean sand with a blow count number of $(N_{1,60})_{cs}$.

The above correction for fines content have been applied to all the data in Table 3.1 and the corrected data are listed in the same table under the column $(N_{1,60})_{cs}$.

3.3.3 Analysis of all data

A plot of S vs $(N_{1,60})_{cs}$ based on all the data in Table 3.1 is given in Figure 3.4. Both S and $(N_{1,60})_{cs}$ are shown on a logarithmic scale. Each dot represents a case of liquefaction failure, while each circle represents a nonliquefied site. A straight line is drawn to define the boundary separating the liquefied and the nonliquefied sites. This line, therefore, mathematically corresponds to the equation for the condition of liquefaction and can be written as

$$S = 3.49(N_{1,60})_{cs}^{7.8} \times 10^{-4} \quad (3.16)$$

The assessment of liquefaction occurrence can then be obtained if the value of the seismic energy function, S , at a specific site calculated from the SPT N-value using Equation 3.16 is smaller or greater than the designated value of S calculated by Equation 3.8. It also appears that if the value of the seismic energy function is less than $2.E+0.4$, no liquefaction will occur.

3.4 COMPARISON OF ENERGY METHOD AND STRESS METHOD

Figure 3.4: Plot S vs $(N_{1,60})_{cs}$

The above proposed new energy method is simpler to apply than the stress method by Robertson and Wride (1998). The present method requires the values of M_w and R . For an earthquake that has taken place, these two quantities can now be more reasonably determined than the peak ground surface acceleration. As soon as these quantities are obtained, along with the $(N_{l,60})_{cs}$ values, the liquefaction potential of a site can be examined based on the seismic energy function, S .

The stress method of Robertson and Wride (1998) also requires evaluation of two quantities of an earthquake: peak horizontal acceleration of the bedrock, a_{max} and moment magnitude, M_w . The process of determination of a_{max} and M_w is similar to that of the present method. After evaluation, however, these values cannot be applied directly to examine the liquefaction potential of a site, assuming $(q_{clN})_{cs}$ is known. Two more steps are required. First, the peak horizontal acceleration at the ground surface, a_{s-max} , has to be estimated using a_{max} with due regard to soil amplification as will be discussed in Chapter 5. This change of ground motion characteristics depends on the geometry and properties of the soil layers. Second, this method is based on observations of earthquakes of M_w approximately equal to 7.5. Estimation of liquefaction potential at other magnitudes is based on a laboratory study to yield a curve showing the liquefaction resistance at various numbers of cycles of loading. This curve then provides correction factors for application of values of M_w other than 7.5.

The present energy method is more reliable than the stress method. It should be noted that both methods examining the likelihood of liquefaction failure for equivalent clean

sand based on different fines content of the soil. While the energy method is based on a large database of 363 independent cases of liquefaction and non-liquefaction for soils with different fines content, the stress method is modified from the original method by Robertson and Campanella (1985) based on discussions at the 1996 NCEER workshop in order to be more consistent with the recent increases in the availability of CPT field performance data. Therefore, application of both methods to the database collected in this study will fairly reflect their relative reliability. For the energy method, the success rate of correctly predicting the results (i.e. both for liquefaction occurrence and nonoccurrence) is 87%. For the stress method, the corresponding success rate is 83% by using the correlation between SPT and CPT values as given by the next chapter in order to convert the $(N_{1.60})_{cs}$ in the database to the required $(q_{c1N})_{cs}$ in the stress method. Therefore, the energy method has a higher rate of success in evaluating liquefaction potential.

Chapter 4

A NEW CORRELATION BETWEEN SPT AND CPT VALUES

4.1 INTRODUCTION

In-situ penetration test results represent an important basis for the evaluation of soil liquefaction potential. Both the standard penetration test (SPT) and the cone penetration test (CPT) are widely used to evaluate the liquefaction resistance of saturated soil deposits. This results in a large number of methods based on either one of these test parameters. However, it is not often in practice to perform both tests in the same project. Besides, due to the difficulties of local practice or the availability of equipment, usually just one kind of penetration test can be done at a specific site. As a result, a reliable correlation between SPT and CPT can provide an interchangeable use of all the available data.

Correlations between SPT and CPT results for granular soils have been proposed by several investigators (Robertson et al., 1983, Robertson and Campanella, 1985, Seed and De Alba, 1986, Andrus and Youd, 1989, Kulhawy and Mayne, 1990 and Stark and Olson,

1995). These correlations express the ratio q_c/N_{60} as a function of the mean grain size of the soil, D_{50} . However, quite a large scatter exists between the ratio of q_c/N_{60} and D_{50} , as will be shown in the next section.

Besides, in order to correlate between SPT and CPT for assessing the characteristic of past earthquake that had caused soil liquefaction (as will be developed in Chapter 5), the correlation should be established using data involving only liquefiable soils.

4.2 EXISTING CORRELATIONS BETWEEN SPT AND CPT VALUES

The results of SPT and CPT tests have conventionally been related by the ratio q_c/N where N is the standard penetration resistance blow-count and q_c is the tip resistance of the standard cone in MPa. A relationship of this type was first proposed by Meyerhof (1956), and as the use of the cone penetration test has intensified, it has been progressively extended and refined. Kruizinga (1982), compiled the existing correlation data published by several investigators and presented them in a table of the q_c/N ratio for different soil types (Table 4.1). The data show that the scatter in sand appears especially large from 0.3 to 1.0. This wide range of q_c/N ratios, as a result, leads to even more confusions. The reason for this scatter was later found by Robertson et al. (1983) to be due to the fact that different grain sizes of the sand particles have different q_c/N ratios and the variations in published q_c/N ratio can be rationalized somewhat by reviewing the derived q_c/N ratios as a function of mean grain size, D_{50} , as shown in Figure 4.1. It is observed that the q_c/N ratio increases with increasing grain size. Since then, there seems

Soil type	q_c/N	Author
A	0.8-1.0	Schmertmann (1970)
A	1.8	Meigh-Nixon (1961)
B	0.5-0.6	Schmertmann (1970)
C	0.8	Meigh-Nixon (1961)
C		Y. Lacroix (1971)
C	1.0	De Alencar Velloso (1959)
C	0.4	Meyerhof (1956)
D	0.3-0.4	Schmertmann (1970)
D	0.6	De Alencar Velloso (1959)
E	0.3-0.5	De Alencar Velloso (1959)
F	0.2	De Alencar Velloso (1959)
F	0.2	Schmertmann (1970)
F	0.4-0.5	Franki *
G	0.3-0.5	De Alencar Velloso (1959)
G	0.2-0.3	Franki *

* year not given in the original reference (Kruizinga, 1982)

q_c in MPa

N in blows/300 mm

A = sandy gravels and gravels

B = coarse sands and sands with little gravel

C = sand

D = clean, fine to medium sands and slightly silty sands

E = sandy silt

F = sandy clay, silty sand, cohesive silt-sand mixtures

G = clay, silty clay, clayey silt

Table 4.1: SPT and CPT correlations (after Kruizinga, 1982)

Figure 4.1: Variation of q_c/N ratio with mean grain size (after Robertson et al., 1983)

to be general agreement that the value of q_c/N varies with the grain size of the soil characterized by the mean grain size, D_{50} , as subsequent investigators (Robertson and Campanella, 1985, Seed and De Alba, 1986, Andrus and Youd, 1989, Kulhawy and Mayne, 1990 and Stark and Olson, 1995) published similar plots with the value of q_c/N varying with the mean grain size, D_{50} , as shown in Figure 4.2. The q_c/N_{60} data presented in Figure 4.2 include those by Seed and De Alba (1986), Robertson and Campanella (1985), Youd and Bennett (1983), Bennett (1989), Bennett (1990) and Kayen et al. (1992). The data exhibit a large variation in the ratio of q_c/N_{60} for a particular value of D_{50} . All SPT data were corrected to a SPT hammer energy of 60% and the data in Figure 4.2 are average values of q_c/N_{60} reported by the investigators for subsurface layers where SPTs and CPTs were conducted adjacent to one another. These data cover both liquefied and non-liquefied soils.

The SPT and CPT correlation suggested by Robertson and Campanella (1985) is based on the mean grain size and an average energy ratio of 55% for the SPT N-values. For consistency, the Robertson and Campanella (1985) correlation and data had been converted to an energy ratio of 60%. Their SPT and CPT correlation indicates that the value of q_c/N_{60} increases with the increasing values of D_{50} .

Seed and De Alba (1986) also proposed a SPT and CPT correlation based on the mean grain size. This correlation, however, remains approximately constant for D_{50} values greater than approximately 0.5 mm. This was the consequence of considering additional data provided by Muromachi and Kobayashi (1982), who presented data covering a wide

Figure 4.2: Conversion of SPT N -values to CPT q_c -values using median grain diameter (after Stark and Olson, 1995)

range of grain sizes, and by Jamiolkowski et al. (1985), whose data points were obtained from mean values of test results on Po Valley sands.

Andrus and Youd (1989) developed a SPT and CPT correlation by extending the Seed and De Alba (1986) correlation to account for values of D_{50} up to 40 - 45 mm. The data used to extend the SPT and CPT correlation were based on gravelly soils involved in the 1983 Borah Peak earthquake. The values of D_{50} used were obtained from the 127-mm auger samples as the diameter of the split spoon sampler was too small to obtain a representative sample of the gravelly soil.

The correlation proposed by Kulhawy and Mayne (1990) is based on statistical analysis of q_c , N_{60} data from 197 cases, with the values of D_{50} ranging from 0.001 mm to 10 mm. This database included the data from Robertson and Campanella (1985) and Seed and De Alba (1986). For values of D_{50} greater than 1 mm, however, the data used by Kulhawy and Mayne (1990) are limited and do not include several of the cases from Andrus and Youd (1989).

The SPT and CPT correlation proposed by Stark and Olson (1995) was developed indirectly by determining the q_c/N_{60} ratios that yielded the best agreement between SPT liquefaction case histories and their CPT-based liquefaction potential relationships. The SPT and CPT correlation obtained is intermediate to the Seed and De Alba (1986) and Robertson and Campanella (1985) correlations. It coincides with the Robertson and Campanella (1985) correlation for values of D_{50} greater than 0.3 mm and with Seed and

De Alba (1986) for values of D_{50} less than 0.08 mm. However, there is a lack of agreement between the Stark and Olson (1995) correlation and the other correlations for values of D_{50} between 0.08 mm and 0.3 mm. It actually moves from the Seed and De Alba (1986) correlation to the Robertson and Campanella (1985) correlation for values of D_{50} between 0.08 and 0.3 mm.

In view of the discrepancy between the correlations on this type of plot, it would now seem desirable to seek a better way of relating SPT and CPT data. The next section presents a new SPT and CPT correlation using the fines content to characterize the soil.

4.3 PROPOSED CORRELATION BETWEEN SPT AND CPT VALUES

In order for the new correlation between the SPT and the CPT values to be applicable for assessing the characteristics of past earthquakes that had caused soil liquefaction, the following two criteria are of utmost importance:

- 1) The relationship should be able to reflect the differences for different types of soil
- 2) The relationship should be developed specifically on the basis of liquefiable materials only

For the first criterion, the reliability on the mean grain size, D_{50} , characteristics of the soil has already been demonstrated to be inadequate in the previous section. Instead of using D_{50} , the concept of “*equivalent clean sand*” introduced in Chapter 3 as presented in the NCEER Workshop on Evaluation of Liquefaction Resistance (NCEER, 1997) is used

here in developing the new correlation. The equation for correcting standard penetration resistance determined for silty sands to an equivalent clean sand penetration resistance is as below:

$$(N_{1,60})_{cs} = \alpha + \lambda N_{1,60} \quad (4.1)$$

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

$$\alpha = 0 \quad \text{for } FC \leq 5\% \quad (4.2a)$$

$$\alpha = \exp[1.76 - (190/FC^2)] \quad \text{for } 5\% < FC < 35\% \quad (4.2b)$$

$$\alpha = 5.0 \quad \text{for } FC \geq 35\% \quad (4.2c)$$

$$\lambda = 1.0 \quad \text{for } FC \leq 5\% \quad (4.3a)$$

$$\lambda = [0.99 + (FC^{1.5}/1000)] \quad \text{for } 5\% < FC < 35\% \quad (4.3b)$$

$$\lambda = 1.2 \quad \text{for } FC \geq 35\% \quad (4.3c)$$

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

On the other hand, the equation to obtain the equivalent clean sand normalized CPT penetration resistance, $(q_{c1N})_{cs}$, is a function of both the measured penetration resistance, q_{c1N} , and the grain characteristics of the soil shown as follows:

$$(q_{c1N})_{cs} = K_c q_{c1N} \quad (4.4)$$

where K_c is a correction factor that is a function of the grain characteristics of the soil, as described in Chapter 2.

After obtaining these corrections of FC for silty materials, it is no longer necessary to plot the SPT-CPT ratio along with different D_{50} in order to account for different types of soil. Instead, a more direct way of presenting the correlation can be obtained as shown later in this section.

To fulfill the second criterion, SPT and CPT data obtained only from liquefiable materials are considered. Table 4.2 summarizes some of the available data in the literature derived from liquefiable sites. Shown in the table are SPT and CPT test data from 13 test sites together with the FC of the soil samples. Also included in the table are the “equivalent clean sand” values for the SPT and the CPT data using Equation 4.1 and Equation 4.4 respectively. The values of all the data pairs are plotted on a graph of $(N_{1.60})_{cs}$ versus $(q_{c1})_{cs}$ in Figure 4.3. From this plot, it appears that there is reasonable correlation between $(N_{1.60})_{cs}$ and $(q_{c1N})_{cs}$, despite some scatter due to experimental errors. The correlation can be described by a linear function of the form $(N_{1.60})_{cs} = L \cdot (q_{c1N})_{cs}$ where the value of L is found to be equal to 0.12. This relationship implies that the $N_{1.60}/q_{c1N}$ ratio is a constant. As such, it is in apparent conflict to existing experience that states the ratio is not a constant but a function of the mean grain size, D_{50} . To resolve this conflict, the following is considered:

Site	Depth of water table (m)	Depth of sand (m)	σ'_v (kN/m ²)	q_c (MPa)	N-value	FC (%)	$(q_{c1N})_{cs}$	$(N_{1.60})_{cs}$	Reference No.
Borah Peak	1.0	2.7	34.04	5	8	2	80.69	12.00	7
	1.0	4.2	49.44	16	20	2	221.76	26.00	7
	1.0	2.4	32.00	5	6	3	95.35	11.00	7
	1.0	4.5	51.62	16	22	0	217.02	32.00	7
	1.0	1.9	26.91	6	7	0	109.64	14.00	7
	1.0	4.1	47.99	18	21	0	253.40	30.00	7
	1.0	3.6	43.63	7	8	0	112.84	12.00	7
	1.0	6.0	66.89	17	20	0	213.45	25.00	7
	1.0	3.8	45.52	19	16	3	280.54	25.00	7
	1.0	2.9	35.93	6	8	21	175.82	16.81	7
	1.0	5.9	65.87	24	25	15	420.10	38.13	7
	1.0	10.7	112.38	20	25	11	228.88	20.71	7
Loma Prieta	3.5	4.0	67.85	2	2	35	79.69	7.91	66
	3.5	7.0	96.92	2	2	33	61.72	7.28	66
	3.5	7.0	96.92	2	2	33	61.72	7.28	66
	3.0	5.0	73.38	5	8	12	74.03	11.29	66
	3.0	7.0	92.76	6	8	11	74.72	9.60	66
	3.0	5.0	73.38	9	10	5	105.06	11.72	66
	3.0	7.5	97.61	10	10	6	101.22	10.18	66
	3.0	5.3	76.29	8	10	5	91.59	11.49	66
	3.0	8.0	102.45	10	10	4	98.80	9.88	66
Niigata	2.0	2.2	46.09	3	3	0	45.16	5.06	41
	2.0	4.4	111.81	2	2	0	21.29	2.38	41
	2.0	13.3	272.67	8	10	0	62.89	8.80	41
	2.0	14.2	281.50	10	10	0	78.61	8.80	41
Tokyo Bay	1.0	6.9	126.53	4	5	30	129.96	12.51	42
	1.0	12.9	201.08	2	3	35	71.48	9.39	42
	1.0	15.0	221.68	2	4	35	71.48	10.85	42
Vancouver	1.0	4.8	89.26	3	6	35	133.94	14.99	77
	1.0	6.3	118.68	8	12	0	96.07	14.69	77
	1.0	9.4	180.47	12	18	0	118.84	18.18	77
	1.0	12.4	238.34	16	17	0	139.51	15.11	77
	1.0	14.0	267.77	5	11	35	135.90	16.15	77

Table 4.2: Available SPT and CPT data

Figure 4.3: Plot $(N_{1,60})_{cs}$ vs $(q_{c1N})_{cs}$

- 1) The L factor in the above proposed relationship is equal to 0.12 and is applicable to clean sand only. Existing correlations for medium to coarse sand with little or zero fines content (i.e. clean sand) give a similar value. Therefore this proposed relationship is consistent to current experience.
- 2) The proposed constant ratio forms a reference for calculating a different factor for sands with a non-zero fines content. Hence this approach will give different ratios for sands with different fines content. Therefore, there is no conflict with existing experience.
- 3) Now let us compare the reliability of the proposed relationship and that of the existing ones as represented by the one proposed by Andrus and Youd (1989). This is done by examining the root mean square error of the predicted value from the actual value as expressed by $\left[\sqrt{\sum \left(\frac{\text{Actual} - \text{Predicted}}{\text{Predicted}} \right)^2} / n \right]$, where n is the total number of data points. Based on the data set listed in Table 4.2, the root mean square error for the proposed relationship is 0.05, while that for the Andrus and Youd (1989) correlation is 0.14. Hence, the proposed relationship is superior to the existing ones.

Finally the proposed linear correlation between SPT and CPT is easier to apply than the existing methods and is especially useful when an explicit equation is required for programming applications. Furthermore, this new correlation allows the conversion from CPT to SPT data without the requirement of additional index testing in order to determine

the grain characteristics of the soil. Thus the requirement of grain size information through soil sampling is eliminated. Consequently, not only will this save significant time and resources for such correlation, but it will also enable us to correlate CPT-SPT data for which the grain size information is missing. In the past missing the grain size information would disqualify the CPT-SPT data for meaningful correlation. The next section will present the validation of the proposed relationship via other correlations between different soil parameters.

4.4 VALIDATION VIA OTHER SOIL PARAMETERS

The standard penetration test (SPT) is firmly established in the geotechnical engineering practice as a routine procedure for in-situ investigation of soil deposits. Due to the longtime and worldwide use of this test, comprehensive data have been accumulated in the past, resulting in a number of SPT-based empirical correlations for soil characterization. One of the well-known correlations between the SPT N-value and relative density, D_r , for sands was proposed by Meyerhof (1957) based on the calibration chamber (CC) test data of Gibbs and Holtz (1957). This widely used correlation suggests that the penetration resistance increases with the square of the relative density and the effective overburden pressure of the sand as

$$N = 0.23D_r^2(\sigma'_v + 70) \quad (4.5)$$

where N = SPT blow count in blows/300 mm, σ'_v = effective overburden stress in kN/m^2 acting at the depth of the SPT test and D_r is the relative density expressed as a ratio. This expression is conventionally written as

$$D_r = \left(\frac{N}{0.23\sigma'_v + 16} \right)^{0.5} \quad (4.6)$$

for ease of use.

Re-analysis of the available SPT data of laboratory and field tests by Skempton (1986) shows that wide variations in N values can occur between different sands, even at a given overburden pressure and constant relative density. This variation appears to be largely due to the different energy ratio (ER) resulting from the use of different procedures in the test and can be reduced by normalizing the results to a standard rod energy ratio of 60%. This becomes the corrected blow count, N_{60} . A relationship between the corrected blow count, relative density and effective overburden pressure in a given normally consolidated sand proposed by Skempton (1986) is as follows:

$$D_r = \left(\frac{N_{60}}{c_1 \sigma'_v + c_2} \right)^{0.5} \quad (4.7)$$

where the parameters c_1 and c_2 are constants and σ'_v is expressed in kN/m^2 . By putting $\sigma'_v = 100 \text{ kN/m}^2$ into the above equation, the correlation between D_r and the corrected normalized SPT N-value, $N_{1.60}$, can be obtained as

$$D_r = \left(\frac{N_{1.60}}{c_3} \right)^{0.5} \quad (4.8)$$

where c_3 is a constant with values ranging from 34 to 44 for sand.

Then, in order to develop an empirical relationship between SPT and CPT, the next step is to find another correction which can relate CPT and D_r together so as to use D_r as the linkage between SPT and CPT. This goal can be achieved indirectly by using two of Meyerhof's (1976) correlations between the friction angle of sand, ϕ , and the measured cone resistance, q_c , and the friction angle and the relative density of sand respectively as stated in Chow et al. (1992). These two correlations are

$$\phi = 29^\circ + 2.5 \sqrt{q_c} \quad (4.9)$$

and

$$\phi = 28^\circ + 15^\circ D_r \quad (4.10)$$

where q_c is the measured CPT resistance in MPa and D_r is the relative density of sand expressed as a ratio. Combining Equation 4.9 and Equation 4.10 together leads to:

$$D_r = \frac{1 + 2.5\sqrt{q_c}}{15} \quad (4.11)$$

The correlation of D_r with the truly normalized cone penetration resistance corrected for overburden stress, q_{c1N} , can then be obtained by using Equation 2.10 with the value of σ'_v equal to 100 kN/m², hence $C_Q = 1$. Equation 4.11 now becomes:

$$D_r = \frac{1 + 2.5\sqrt{0.1q_{c1N}}}{15} \quad (4.12)$$

where q_{c1N} is dimensionless.

By combining Equation 4.8 and Equation 4.12, the empirical correlation between SPT and CPT can be obtained as:

$$N_{1.60} = \left(\frac{1 + 2.5\sqrt{0.1q_{c1N}}}{15} \right)^2 \cdot c_3 \quad (4.13)$$

where $N_{1.60}$ is the corrected normalized SPT N-value and q_{c1N} is the normalized corrected CPT resistance. Figure 4.4 shows this correction in graphical form using a value of $c_3 = 38$ at intervals of 10 q_{c1N} . It can be seen from the figure that it represents an almost straight line passing through the origin with slope 0.121 which is very close to the

Figure 4.4: Plot theoretical clean sand $N_{1,60}$ vs q_{c1N} - Validation 1

proposed correlation in Section 4.3. Other values of c_3 within the normal range give similar results.

Besides, not only the above mentioned well-known correlations which are developed a few decades ago can be used to validate the proposed SPT and CPT correlation, a recent finding by Cubrinovski and Ishihara (1999) shows equally good results. Cubrinovski and Ishihara (1999), in an attempt to establish an improved empirical correlation between the SPT N-value and the relative density which is applicable to sandy soils with various grain size, introduced a new parameter, the *void ratio range* ($e_{\max} - e_{\min}$), to allow for the effects of grain size and grain size distribution of soils. By using more reliable data from high-quality undisturbed soil samples, a correlation is obtained as:

$$D_r = \left\{ \frac{N(e_{\max} - e_{\min})^{1.7}}{9} \left(\frac{98}{\sigma'_v} \right)^{1.2} \right\}^{1/2} \quad (4.14)$$

where D_r is defined as a ratio, σ'_v is given in kPa, N is the SPT N-value corresponds to an SPT energy rod ratio of about 78% from the theoretical free-fall energy and $(e_{\max} - e_{\min})$ is the void ratio range with values ranging from 0.30 – 0.70 for sandy materials. Converted to the corrected normalized SPT N-value, $N_{1,60}$, Equation 4.14 becomes:

$$D_r = \left\{ \frac{N_{1,60} \times \left(\frac{60}{78} \right) \times (e_{\max} - e_{\min})^{1.7}}{9} \left(\frac{98}{100} \right)^{1.2} \right\}^{1/2} \quad (4.15)$$

Similar to the previous case, by combining Equation 4.8 and Equation 4.15, the empirical correlation between SPT and CPT is obtained as:

$$N_{1.60} = 70.91 \left(\frac{1 + 2.5\sqrt{0.1q_{c1N}}}{15} \right)^2 \cdot (e_{\max} - e_{\min})^{-1.7} \quad (4.16)$$

where $N_{1.60}$ is the corrected normalized SPT N-value, q_{c1N} is the normalized corrected CPT resistance and $(e_{\max} - e_{\min})$ is the void ratio range. Figure 4.5 shows this correction in graphical form using a void ratio range of 0.5 at intervals of 10 q_{c1N} . It can be seen from the figure that it also represents an almost straight line passing through the origin with slope of 0.122 which once again is very close to the proposed correlation in Section 4.3. Other values of $(e_{\max} - e_{\min})$ within the normal range give similar results.

It may, therefore, be concluded with Figure 4.4 and Figure 4.5 that the new correlation between SPT and CPT proposed in this study shows good agreement with the empirical correlations which are obtained indirectly by using correlations that are established through independent sources of in-situ data and back-analyses by different investigators.

4.5 COMPARISON WITH EXISTING CORRELATIONS

The new correlation between SPT and CPT values proposed above is a better way to account for different types of soil. In order to test the reliability of the newly proposed

Figure 4.5: Plot theoretical clean sand $N_{1,60}$ vs q_{c1N} - validation 2

correlation with due regards to those exiting correlations, all the data form Table 4.2 are plotted together with the five correlations discussed previously in Section 4.2 and is shown in Figure 4.6. Figure 4.6 is in the same format as Figure 4.2 for the sake of easy comparison. From the figure, it can be observed that all the five correlations did cover certain parts of the data points accordingly. However, none of them can completely describe the whole range of data points. This is due to the fact that large variations occur for a certain value of D_{50} . This also suggested that using D_{50} is not a good way to account for different types of soil as it is rather insensitive to the change in q_c/N ratio. As in many cases, the same D_{50} value gives different penetration values even from the same borehole. On the other hand, using equivalent clean sand penetration values in the new correlation can overcome this problem as all the data are corrected separately for soil types before comparing the SPT to CPT values. As a result, it can be concluded that the new correlation is a more reliable correlation which can describe better for different types of soil. Within the range of scatter (i.e. ± 0.03 as shown by the dotted lines in Figure 4.3), the calculated value of M_w using Equation 3.8 and Equation 3.16 with the values of σ'_v and R equal to 100 kPa and 100 km respectively, are within an acceptable range of ± 0.5 .

Figure 4.6: SPT and CPT relationships

Chapter 5

A NOVEL METHOD TO ASSESS PAST EARTHQUAKE CHARACTERISTICS

5.1 INTRODUCTION

The possibility of obtaining an objective, quantitative measure of the magnitude of an earthquake came about with the development of modern instrumentation for measuring ground motion during earthquakes. In the past 60 years, the development of seismic instruments, and our understanding of the quantities they measure, have increased dramatically. Seismic instruments allow an objective, quantitative measurement of earthquake size called *earthquake magnitude* to be made. Before that, only qualitative records had been made which date back many centuries. Written records of earthquakes in China date as far back as 3000 years. Japanese records and records from the eastern Mediterranean region go back nearly 1600 years. In the United States the historical record of earthquakes is much shorter, about 350 years. On the seismically active west coast of the United States, earthquake records go back only about 200 years. Nevertheless, compared with the millions of years over which earthquakes have been occurring, humankind's experience with earthquakes is still very brief. Earthquakes have

occurred for millions of years and will continue in the future as they have in the past. Thus, a more accurate quantification of past earthquakes will improve the accuracy of the seismic zoning maps and enhance our ability to assess potential seismic hazards.

Besides, other than those pre-instrumental earthquakes, better quantification is also needed for some recent earthquakes that happened in remote areas where neither human nor instrumentation exists. This is due to the ever increasing demand of lands for building projects especially those important one just like nuclear plants. As a result, a way to quantify unrecorded earthquakes is required.

In order to develop a method to assess past earthquake characteristics, well documented earthquakes are required. The 1988 Saguenay event, for instance, is the first earthquake in Eastern North America (ENA) in which both the instrumental ground motion characteristics and liquefaction failure have been documented. This provides a good opportunity for the investigation of the potential that liquefaction failures, interpreted with geotechnical methods and seismological principles may be used to assess the magnitude, hypocentral distance and/or peak ground motion acceleration of past earthquakes. The following sections describe the basis for the development of a novel method to assess past earthquakes that had caused soil liquefaction failures.

5.2 CHARACTERISTICS OF SOIL AFTER LIQUEFACTION

The development of a method which correlate in-situ test results of liquefied sand deposits during a past earthquake to the earthquake characteristics requires a good understanding of how these tests are influenced by factors such as the aging and the stress and strain history of the soil. For aging, both field evidence and laboratory test results by several investigators have shown that the age has certain influence on the cyclic liquefaction resistance of cohesionless soils. For example, Seed (1979) observed significant increases in liquefaction resistance with the 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 about 25 percent (Figure 5.1) 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) noted that liquefaction resistance increases according to 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.

However, the influence of aging to the penetration test result is not the same. Laboratory tests by Daramola (1980) indicated that the mechanics of the process is essentially an increase in stiffness. In drained triaxial tests on a river sand, with $D_{50} = 0.3$ mm, he

Figure 5.1: Influence of period of sustained pressure on strength (after Seed, 1979)

found that the secant modulus increased by 60% and 100% respectively in tests made after 30 days and 150 days consolidation under 400 kPa, compared with the results of freshly prepared samples, all at virtually the same relative density of 67%. The strains to failure were correspondingly reduced, from 5.5%, to 3.5% and 2.5% at 30 days and 150 days, but interestingly the angle of internal friction was barely changed (at $\phi = 39-39.5^\circ$), the interparticle bonds presumably having been broken on the approach to failure (Skempton, 1986). This implies that the penetration test values obtained will remain unchanged unless chemical or thermal aging had been occurred.

For the influence of the stress and strain history of the soil, Biegansky and Marcuson (1976) presented the results of twelve SPT tests carried out in the calibration chamber (CC) on Reid Bedford Model sand specimens with an OCR equal to 3. Table 5.1 compares SPT-N resistances measured on OC specimens with SPT-N resistances obtained on NC specimens of the same sand. The comparison shows that the SPT seems to be quite insensitive to soil stress history. Lambrecht and Leonards (1978) postulated, on the basis of small-scale laboratory tests, that cone penetration test resistance, q_c , is completely insensitive to past strain history. Marchetti (1982) and Baldi et al. (1982) documented by means of numerous large-scale laboratory CC tests on Ticino sand, that q_c is only slightly influenced by the strain history of the soil specimen. Later, Jamiolkowski et al. (1985) on the basis of extensive CC tests performed on Ticino and Høksund sand showed that q_c in Ticino sand is almost independent of strain history (Figure 5.2) when the specimen is subjected to cyclic loading along the K_0 -line.

Dr (%)	σ'_v (kN/m ²)	SPT N-value (blows/0.3 m)	
		NC	OCR = 3
58	70	11 to 13	12
58	280	22 to 23	25 to 27
58	560	-	35 to 38
40	70	6 to 9	4 to 5
40	280	7 to 15	16
40	560	25 to 31	27 to 30

Table 5.1 Influence of overconsolidation on laboratory SPT N-value (after Bieganousky and Marcuson, 1976)

Therefore, it appears that both N and q_c are influenced probably only to a limited extent by both aging and past stress and strain history of the soil. As a result, both N and q_c values can be used in the novel method as a good assessment of past earthquake characteristics with respect to the influence of aging and past history of the soil being tested.

Besides the influence of the age, and past stress and strain history on the in-situ test results, whether the N and q_c values will change after the earthquake induced liquefaction had occurred is another issue that needs to be confirmed before the use of them is granted for the assessment of past earthquake characteristics. This concern is due to the fact that soil investigations, if any, are often done after the earthquakes, there is a possibility that these soil properties are different from those before the earthquakes.

Figure 5.2: Influence of strain history on q_c (after Jamiolkowski et al., 1985)

Right from the first liquefaction potential assessment, soil properties have been assumed to remain the same before and after the occurrence of liquefaction as they were all based on penetration test values measured after earthquakes. However, it was not until after the recent 1995 Hyogoken-Nambu (Kobe) earthquake of Japan that Inagaki et al. (1996) showed actual field data which concretely proved that this is true. Figure 5.3 shows the SPT N-values of the landfill at one of the most severe liquefied site, Kobe Port, before and after the earthquake. These results show that the SPT N-values of landfill before and after the earthquake range from about 5 to 10 and there is not a large difference between the SPT N-values of landfill before and after the earthquake. Since the SPT N-values are good indices of many of the soil properties relevant to earthquake response and liquefaction studies, the results obtained by the geotechnical investigations after the earthquake can now be considered good approximations of those before the earthquake.

5.3 A NOVEL METHOD TO ASSESS PAST EARTHQUAKE CHARACTERISTICS

A novel method to assess past earthquake characteristics essentially involves the development of four independent equations governing the relationship between soil resistance, earthquake magnitude, ground accelerations, hypocentral distance and attenuation characteristics of seismic energy. These equations will then be solved simultaneously to obtain the required unknowns. The following paragraphs describe step by step of how the method is established.

Figure-5.3: SPT N-values before and after the Kobe earthquake (after Inagaki et al., 1996)

As mentioned at the beginning of this chapter, a better quantification of past earthquake characteristics starts with a better estimation of the earthquake magnitude. That means the soil test results obtained from a liquefied site have first to be related with the earthquake magnitude by some means. For that, both the stress method and the energy method equations described in previous chapters can be utilized.

From the stress method, the functional form of Robertson and Wride (1998) equation for assessing liquefaction potential can be written as:

$$\theta = F_l (a_{s-\max}, M_w, z, \sigma_v, \sigma'_v) \quad (5.1)$$

where θ = in-situ properties of the soil that liquefied under the given earthquake

F_l = a function of the factors within the brackets

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

M_w = moment magnitude of earthquake

z = depth of soil at the level of liquefaction

σ_v = total vertical in-situ stress

σ'_v = effective vertical in-situ stress

For any given liquefied site θ , z , σ_v , σ'_v can be measured while $a_{s-\max}$ and M_w are unknowns. As a result, Equation 5.1 becomes the first equation in the novel method.

Then from the new energy approach for assessing liquefaction potential as described in Chapter 3, the functional form of a second equation in the novel method can be expressed as:

$$\theta = F_2 (M_w, R) \quad (5.2)$$

where F_2 = a function of the factors within the brackets

M_w = moment magnitude of earthquake

R = the hypocentral or closest distance to the fault

Once again, for any given liquefied site, θ can be measured while M_w and R are unknowns.

These two equations, although both relating soil properties with earthquake characteristics, are two independent equations. The reason is due to the fact that they are established from two completely different aspects of an earthquake. The first is the peak ground acceleration and the other is the energy or the peak ground velocity of the earthquake.

Consequently, two equations with three unknowns (a_{s-max} , M_w and R) have been coming out. In order to solve them, another equation with a functional form below is required:

$$a_{s-max} = F_3 (M_w, R) \quad (5.3)$$

This can be achieved by using the ground motion models as discussed in Section 2.4. From that, the simple functional form of the ground motion model developed by Atkinson and Boore (1995) (i.e. Equation 2.25) for Eastern North American and Atkinson and Silva (2000) (i.e. Equation 2.26) for California, respectively, is:

$$a_{\max} = F_4 (M_w, R) \quad (5.4)$$

where $a_{\max}$ = the maximum peak ground acceleration (PGA) on rock

F_4 = a function of the factors within the brackets

M_w = moment magnitude of earthquake

R = the hypocentral or closest distance to the fault

and are all unknowns for a particular past earthquake.

The values of $a_{s-\max}$ in Equation 5.3 and $a_{\max}$ in Equation 5.4 are related to each other by an amplification factor A as follows:

$$a_{s-\max} = A \cdot a_{\max} \quad (5.5)$$

As the seismic waves travel through the compressible soil layers from the bedrock, the maximum peak horizontal acceleration at the ground surface will be modified. The magnitude of amplification of acceleration at the ground surface depends on many factors including the thickness of soil, the compressional characteristics of the soil compared to

that of the bedrock, the shape of bedrock, other local geological features and the frequency contents of the ground motion. Therefore A is a site-specific parameter. Generic relations determined from strong motion database have also been published by some investigators such as Abrahamson and Silva (1997) and Boore and Joyner (1997). On the other hand, Atkinson and Silva (2000), by adapting the soil response factors determined by Abrahamson and Silva (1997), produced a relationship (Figure 5.4) showing how the total amplification effects for generic California crust vary with frequency including rock sites and soil sites at weak and strong levels of input rock motion.

A similar amplification factor called the foundation factor, F , is incorporated in the National Building Code of Canada (NBCC, 1990) design base shear formula to account for site effects of the amplification of bedrock motions as these propagate upwards through the local soil deposit (Heidebrecht et al. 1991 and Hosni and Heidebrecht, 1994). The value of F is specified as a function of the local soil type and depth in accordance with Table 5.2. From that, the range of amplification is from 1.0 to 2.0 for rock to very soft soils with depth greater than 15 m.

Category	F	Type and depth* of soil deposit
1	1.0	Rock, dense and very dense coarse-grained soils, very stiff and hard fine-grained soils; compact coarse-grained soils and firm and stiff fine-grained soils from 0 to 15 m deep

Table 5.2: Foundation factor categories (after NBCC, 1990)

Figure 5.4: Total amplification effects for generic California crust (after Atkinson and Silva, 2000)

2	1.3	Compact coarse-grained soils, firm and stiff fine-grained soils with a depth greater than 15 m; very loose and loose coarse-grained soils and very soft and soft fine-grained soils from 0 to 15 m deep
3	1.5	Very loose and loose coarse-grained soils with depth greater than 15 m
4	2.0	Very soft and soft fine-grained soils with depth greater than 15 m

*Depth is measured from the foundation or pile cap level.

Table 5.2: Continued

The mathematical form of the four established equations (i.e. Equations 5.1, 5.2, 5.4 and 5.5) can now be summarized as:

$$a_1 \left[\frac{(q_{clN})_{cs}}{1000} \right]^{a_2} + a_3 = 0.65 \left(\frac{a_{s-max}}{g} \right) \left(\frac{\sigma_v}{\sigma'_v} \right) (a_4 - a_5 z) \left(\frac{M_w}{7.5} \right)^{2.56} \quad (5.6a)$$

$$\frac{10^{1.5M_w}}{\sigma'_v R^2} = 3.49 (N_{1.60})_{cs}^{7.81} \times 10^{-4} \quad (5.6b)$$

$$a_{s-max} = a_6 \cdot a_{max} \quad (5.6c)$$

$$\log a_{max} = a_7 + a_8 (M_w - 6) + a_9 (M_w - 6)^2 - \log R - a_{10} R \quad (5.6d)$$

where a_1 to a_{10} are constant for a given geological condition.

There are therefore a total of four independent equations with four unknowns: M_w , R , a_{s-max} and a_{max} . Together with the correlation between $(N_{1,60})_{cs}$ and $(q_{c1N})_{cs}$ developed in Chapter 4, the four equations can be solved simultaneously by the graphical method (Saaty and Bran, 1964) to yield the values of the four unknowns. The other quantities can be measured at the site where liquefaction is known to have taken place.

Four case studies will be given in Chapter 6 in order to show the applicability of this novel method to some recent earthquakes at different parts of the world.

Chapter 6

CASE STUDY ON RECENT EARTHQUAKES

6.1 INTRODUCTION

The aim of the past earthquake characteristics assessment method developed in this research is to assess the characteristics of past earthquakes that had caused soil liquefaction at the time when it occurred by using the in-situ soil properties of the liquefied site. As such, in order to verify this novel method, recent earthquakes that are well documented and had liquefaction failures occurred are suitable for the purpose of comparison.

The following four sections describe case studies on four recent earthquakes: the 1988 Saguenay earthquake of Canada, the 1989 Loma Prieta earthquake of USA, the 1995 Hyogoken-Nambu (Kobe) earthquake of Japan and the 1999 Chi-Chi earthquake of Taiwan. These earthquakes are purposely chosen from different parts of the world in order to show the wide applicability. Of these earthquakes, the moment magnitude, M_w , the hypocentral or closest distance to the fault, R , and the peak horizontal acceleration at

the ground surface, a_{s-max} , have been obtained and are used to compare with the calculated values from the analysis.

6.2 THE 1988 SAGUENAY EARTHQUAKE

6.2.1 Background information

On Friday November 25, 1988, at 18:46 Eastern Standard time (EST), the Saguenay earthquake, the most significant earthquake to take place in Eastern North America in over fifty years, occurred close to the northern border of the Laurentides Provincial Park in the province of Quebec. The epicenter was 36 kilometers south of the towns of Chicoutimi and Jonquiere on the southern shore of the Saguenay River and was located at 48.117°N latitude and 71.186°W longitude at a depth of 29 kilometers (Munro and Weichert, 1989). This moment magnitude 5.8 (M_w) earthquake was felt over a very wide area, from Goose Bay, Labrador, in the east to Thunder Bay, Ontario, in the west and from Kuujjuarapik on Hudson's Bay, in the north to Washington, D.C., in the south; the felt area was in excess of 3.5 million km². Although there was no major structural damage, the estimated damage to residential buildings amounted to \$7 million just in the Province of Quebec alone (Law, 1990). This amount represented only a small fraction of the total damage because it did not include damage to public buildings, hospital and transportation routes.

Damage during this earthquake that is of geotechnical interest includes landslides in nature slope, highway and railway embankment failure, some rockfalls, foundation

settlement, and liquefaction failures and its associated damage. This earthquake is the first case in Eastern North America in which both soil liquefaction and ground motions have been instrumentally recorded.

6.2.2 Description of liquefied sites

Piezcone penetrometer tests (CPT) were conducted at six sites in Ferland and Boilleau by Law (1990) as shown in Figure 6.1. For each site, liquefaction failure has been reported. Each piezocone profile reached a hard or clay layer at a depth beyond the liquefied soil. A GMF 50 kN piezocone penetrometer was used in the testing program. The cone has a 60° conical tip with 10 cm^2 cross sectional area. The soil type was determined by visual examination and sampling for grain size tests in the laboratory. Soil samples were obtained using the 152 mm diameter augers. Table 6.1 below shows the hypocentral distances respectively of the six sites together with the depth of water table and the depth of liquefied layers. The maximum surface acceleration, $a_{s-\max}$, is likely to be in the range of 0.25 to 0.30 g at these sites (Law, 1990).

Site	Hypocentral distance (km)	Depth of water table (m)	Depth of liquefied layers (m)
1	38.68	1.8	4.4 to 4.7
2	38.62	0.4	1.7 to 2.5
3	38.29	2.0	4.1 to 4.5
4	39.15	2.1	2.8 to 3.3
5	39.15	1.8	3.5 to 7.5
6	37.07	0.1	1.7 to 3.0

Table 6.1: Data of the six sites – Saguenay

Figure 6.1: Location of field tests and recorded ground failure (after Law, 1990)

Site 1 is located just south of the centre of Ferland west of Rt. #381. A piezocone profile was obtained at about 11 m away from a house which suffered damage due to liquefaction failure. The soil profile is composed of a 1.5 m top soil and fill of gravelly sand and clayey silt. The fill is underlain by a silty sand of loose to medium dense compactness. This is followed by a 2 m thick clay deposit below which a hard gravelly layer is encountered. The record of drilling a well at this site shows that bedrock lies at a depth of about 30 m.

A sand boil deposit of gray sandy silt, about 1 m in diameter and 7.5 cm thick was observed in the yard about 6 m west of the house and 5 m east of the piezocone profile. In addition, the earthquake also caused large quantities of gray sand to come up from the bottom of the creek, 12 m south of the house. All basement walls of the house were cracked, presumably as a result of settlement of the footing over a liquefied layer. Large quantities of sand and water flowed into the home through the bathtub and the toilet in the basement. The tenant had to remove a 22 cm thick layer of sand from the tub afterwards. The chimney was tilted a few degrees.

Site 2 is about 55 m to the west of site 1. There a large sand boil deposit of gray silty sand to sandy silt is located. It has a diameter of 10 m and thickness of 22 cm. The soil profile is composed of a thin layer of top soil followed by 0.3 m of medium to coarse brown sand. This sand becomes lighter in colour and finer in grain size with increasing depth. A layer of stones is found at about 2 m depth. Below this layer at 2.3 m depth gray silty sand to sandy silt similar to the material of the sand boil is found. The

compactness of this layer increases with depth and at 5.8 m depth, a clay layer is encountered.

Site 3 is located on Rue Paquet of Rt. #381. Here gray silty sand vented through fissures along the asphalt pavement and shoulder. Off the pavement on the west side, patches of similar sand were noted to cover the natural ground. The soil profile at this site is composed of a top 2.5 m thick brown medium to coarse sand. Beyond this depth the sand turns lighter in colour and finer in grain size with depth. At 4.2 m depth, it becomes a gray silty sand similar to that vented on the surface. This layer terminates at a depth of 5.5 m where a 0.3 m clay layer follows. This is underlain by a medium dense sand at 5.8 m depth.

Site 4 is located north of the centre of Ferland on Rt. #381. A piezocone profile was conducted on the gravel driveway. The soil profile consists of a 2 m fill of mixture of loose sand, organic material and wood chips. Below this to 3 m depth is a mixture of sand and stones. This is followed by a gray silty sand with rapidly increasing compactness. At 4.5 m depth, it becomes a medium dense to dense sand. Two sand boil deposits on this property were reported by the owner. He also identified that the material from about 3 m depth was similar to the boil material. Sand and water also entered the basement of the house on this site, depositing a layer of 20 to 30 cm thick sand layer on the basement floor. Cracks on the wall indicative of differential settlement were found.

Site 5 is located in Boilleau on the east side of Rt. #381. A piezocone profile was conducted on the unpaved part of the driveway of the house at the site. The soil profile consists of a 1 m of compact fill. This is followed by a dark brown organic silt to a depth of 3.5 m. Between this depth to 7.5 m depth lies a layer of gray silty sand with occasional stones. From 7.5 m onward, the soil becomes more clayey and the profiling stopped at 10.3 m depth in the clay layer.

According to the owner, numerous sand boils of gray silty sand were observed around his property during the earthquake. Some came through fissures in the asphalt pavement and others through natural ground. The basement floor sustained extensive cracks through which the gray silty sand and water entered the basement. Differential settlement of at least 4 cm was noted between the basement wall and the centre of the house. Aluminum sidings near the bottom of an exterior wall were buckled due to the differential settlement and the stainless steel chimney was broken at the elbow joint coming out of the exterior wall.

Site 6 is located at a creek on the east side of Rang Edmour-Lavoie. The creek runs along the foot of a hill. Numerous gray medium sand boils were noted on a nearby terrace within 15 m of the creek. Here some small artesian pressure was observed. The soil profile is composed of a top 1.7 m thick layer of brown medium to coarse sand with organic content. Loose to compact sand with stones is found between 1.7 m to 2.3 m. This is followed by a gray medium sand to a depth of 3 m, similar to the boil material.

Underlying this is a layer of gray clayey silt to a depth of 4.7 m beyond which a dense sand layer is located.

The measured piezocone tip resistance, q_c , and sleeve friction, f_s , profiles for the six sites are shown in Figure 6.2 to Figure 6.13 respectively.

6.2.3 Case analysis

To perform an assessment of earthquake characteristics, as has been described in the last chapter, representative in-situ soil properties of the liquefied site induced by the earthquake have to be obtained. For the same magnitude of an earthquake, liquefaction will occur at places where q_c values are equal to or lower than the triggering value that soil liquefaction can take place. Therefore, to improve the reliability of the analysis, the cone penetration values at the beginning and the end of a liquefied layer are used for the analysis. These are normally the largest values within the liquefied layer and closely reflect the threshold values at which liquefaction just starts or terminates. Table 6.2 shows the q_c values (Law, 1990) together with the corresponding sleeve friction, f_s , at levels where liquefaction begins and ended for site 1 to site 4 respectively. Also shown in Table 6.2 are the total vertical in-situ stress, σ_v , and the effective vertical in-situ stress, σ'_v , at each level.

Figure 6.2: CPT q_c profile of site 1 (after Law, 1990)

Figure 6.3: CPT f_s profile of site 1 (private communication with Law, 2001)

Figure 6.4: CPT q_c profile of site 2 (after Law, 1990)

Figure 6.5: CPT f_s profile of site 2 (private communication with Law, 2001)

Figure 6.6: CPT q_c profile of site 3 (after Law, 1990)

Figure 6.7: CPT f_s profile of site 3 (private communication with Law, 2001)

Figure 6.8: CPT q_c profile of site 4 (after Law, 1990)

Figure 6.9: CPT f_s profile of site 4 (private communication with Law, 2001)

Figure 6.10: CPT q_c profile of site 5 (after Law, 1990)

Figure 6.11: CPT f_s profile of site 5 (private communication with Law, 2001)

Figure 6.12: CPT q_c profile of site 6 (after Law, 1990)

Figure 6.13: CPT f_s profile of site 6 (private communication with Law, 2001)

Site	Depth of sand (m)	σ_v (kPa)	σ'_v (kPa)	q_c (MPa)	f_s (kPa)
1	4.4	83.18	57.18	3.8	40
	4.7	89.03	60.09	4.5	40
2	1.7	32.55	19.80	3.0	25
	2.5	48.15	27.55	3.5	25
3	4.1	76.95	56.35	4.0	40
	4.5	84.75	60.23	6.0	25
4	2.8	51.45	44.58	3.0	75
	3.3	61.20	49.43	3.0	60

Table 6.2: Results of test at liquefied layers – Saguenay

Site 5 and site 6 are not included in the analysis due to the fact that liquefaction had occurred throughout the whole layer of sand deposit that can be liquefied. This happens due to the extremely low q_c value of the soil and therefore no upper limiting q_c values for the site can be determined. However, they did serve as a good example of field data records that are not suitable for this kind of liquefaction analysis. Also to be noted is that, as the determination of the beginning and the end of a liquefied layer may not be exact, the q_c values chosen for the other site may not be accurate as well.

The next input parameter for analyzing an earthquake is the value of amplification, A , at the liquefied sites. This amplification is the ratio of the peak acceleration at the ground surface to the peak acceleration at the bedrock. For the Saguenay earthquake, this can be estimated by comparing the strong motion data recorded during the earthquake on top of bedrock to those on top of soil. Figure 6.14 shows the location of the Eastern Canada Strong Motion Seismograph Network stations (Munro and Weichert, 1989) that were triggered by the main shock of November 25. The recorded peak horizontal accelerations

Figure 6.14: Eastern Canada Strong Motion Seismograph Network; October 1988 (after Munro and Weichert, 1989)

are listed in Table 6.3 and are plotted against the distance from epicenter in Figure 6.15. It can be seen that the only station on top of an alluvium layer is the Baie-St-Paul station. The peak horizontal acceleration observed at this station is about 1.4 times those observed at the two nearby stations that are on bedrock (i.e. Les Eboulements and La Malbaie stations). Hence the amplification, A , can be estimated to be about 1.4.

	Station	Distance (km)	Azimuth (°)	Hoiz. Acc. (% g)	Foundation
16	Chicoutimi-Nord	43.2	17.6	13.1	Bedrock
17	St-Andre	63.6	291.3	15.6	Bedrock
20	Les Eboulements	90.4	134.2	12.5	Bedrock
7	Baie-St-Paul	91.0	145.6	17.4	Alluvium
8	La Malbaie	93.0	123.4	12.4	Bedrock
5	Tadoussac	109.2	88.1	2.7	Bedrock
1	St-Ferreol	113.8	166.1	12.1	Bedrock
10	Riviere-Ouelle	114.4	128.3	5.7	Bedrock
9	St-Pascal	122.7	122.1	5.6	Bedrock
2	Quebec	149.3	182.5	5.1	Bedrock
14	Ste-Lucie-de-Beauregard	176.8	149.6	2.3	Bedrock

Table 6.3: Peak horizontal acceleration at triggered stations – Saguenay

(Munro and Weichert, 1989)

As the Saguenay earthquake was located in the Province of Quebec which is in Eastern Canada, the ground motion model developed by Atkinson and Boore (1995) for the Eastern North American may be used in the analysis. Accordingly, the theoretical values of A at the above described six sites obtained using the Atkinson and Boore (1995) model are in the range of 1.91 to 2.03. As a result, the analysis has been carried out with values of A equal to 1.5 and 2.0.

Figure 6.15: Measured peak horizontal acceleration vs epicentral distance – Saguenay (after Law, 1990)

Having all this information determined, the Saguenay earthquake can then be analyzed using the system of equations developed for the novel method in Chapter 5 to back-calculate the earthquake characteristics. The results obtained are tabulated in Table 6.4 and Table 6.5 for $A = 1.5$ and $A = 2.0$ respectively.

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
1	4.4	6.0	35.08	0.24
	4.7	6.1	37.19	0.24
2	1.7	5.8	29.22	0.26
	2.5	5.9	34.81	0.23
3	4.1	5.9	28.38	0.28
	4.5	5.8	27.50	0.27
4	2.8	6.6	30.75	0.40
	3.3	6.4	34.66	0.32

Table 6.4: Results of analysis using $A = 1.5$ – Saguenay

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
1	4.4	6.3	58.89	0.22
	4.7	6.4	62.44	0.21
2	1.7	6.1	49.05	0.23
	2.5	6.2	58.43	0.20
3	4.1	6.3	56.63	0.23
	4.5	6.2	54.87	0.23
4	2.8	6.9	51.63	0.35
	3.3	6.7	58.18	0.28

Table 6.5: Results of analysis using $A = 2.0$ – Saguenay

To have an overall picture of the results, the range and the average values for the calculated M , R and a_{s-max} are tabulated in Table 6.6 and Table 6.7 respectively, together with the actual values.

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	5.8	38.29 – 39.15	0.25 – 0.30*
Calculated (range)	5.8 – 6.6	27.50 – 37.19	0.23 – 0.40
Calculated (average)	6.1	32.20	0.28

*deduced by Law (1990)

Table 6.6: The range and average of the results ($A = 1.5$) – Saguenay

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	5.8	38.29 – 39.15	0.25 – 0.30*
Calculated (range)	6.1 – 6.9	49.05 – 62.44	0.20 – 0.35
Calculated (average)	6.4	56.27	0.24

*deduced by Law (1990)

Table 6.7: The range and average of the results ($A = 2.0$) – Saguenay

In view of the facts that the measured q_c may not be exactly the threshold value for liquefaction failure triggered in this earthquake and scattering exists on all the estimated equations, the average calculated values of the earthquake magnitude, hypocentral distance and maximum peak horizontal acceleration at the ground surface by using an amplification factor of 1.5 as listed in Table 6.6 are considered reasonably close to the actual values. It can, therefore, show the applicability of the novel assessment method to the earthquake in Eastern Canada. For those earthquakes from other parts of the world, analysis will be done immediately in the following sections in order to fully verify the method.

6.3 THE 1989 LOMA PRIETA EARTHQUAKE

6.3.1 Background information

On October 17, 1989 at 5:04 p.m., the San Francisco Bay region of California was shaken by the Loma Prieta earthquake (Mitchell et al., 1994). The Richter magnitude 7.1 (M_w 6.9) earthquake was the largest in the area since the 1906 San Francisco earthquake. The main shock occurred at latitude 37.040° N., longitude 121.880° W., at 17 km depth (Dietz and Ellsworth, 1990), southwest of the southern Santa Cruz Mountains section of the San Andreas fault. The earthquake was felt from the city of Los Angeles, California to the Oregon-California state border in the north, and to the western part of the state of Nevada in the east covering an area of approximately 1,024,000 square kilometers (400,00 square miles).

This earthquake has been estimated as one of the largest natural disasters in U.S. history, with \$10 billion economic losses, 64 confirmed deaths, and more than 3700 injuries. It caused severe damage to a number of engineered structures such as the Cypress Street viaduct of the interstate highway I-880 (also called Nimitz Freeway), where dozens of motorists were killed. It also caused displacement of 18 cm of the section of the San Francisco-Oakland Bay Bridge on the Oakland side of the bridge and heavily damaged a number of bridges and highways. A number of cities were hit, including San Francisco and Oakland, but the major areas of destruction were limited to several pockets associated with soft soil deposits, especially fill areas. The earthquake caused ground

failures in many areas, including soil liquefaction, landslides, soil lateral spreads, and ground cracks (Rainer et al. 1990).

6.3.2 Description of liquefied sites

Electronic cone penetrometer testing was performed at four sites which showed evidence of liquefaction during the Loma Prieta earthquake (Mitchell et al., 1994). The four sites are the Port of Richmond, San Francisco-Oakland Bay Bridge Toll Plaza, the Port of Oakland and Alameda Bay Farm Island and are located as shown in Figure 6.16.

The Port of Richmond

The Port of Richmond is located approximately 88 kilometers ($R = 89.63$ km) north of the Loma Prieta earthquake fault rupture. The present site was created by placing hydraulic fill of silty sand over soft normally consolidated clay, known locally as Young San Francisco Bay Mud. The Port of Richmond overlies a historic stream channel, and the cohesive deposits underlying the site are much deeper than the adjacent cohesive deposits. The deeper cohesive deposits are believed to have amplified the levels of acceleration and, therefore, contributed not only to ground liquefaction but also to considerable structural damage to a warehouse located nearby. Evidence of liquefaction at this site was seen in the form of sand boils, minor settlements, and lateral spreading on the order of magnitude of 2.5 to 8 centimetres. The estimated maximum horizontal ground surface acceleration, $a_{s-\max}$, was approximately 0.13 to 0.18 g (Kayen et al., 1992). The watertable is approximately 3.5 metres deep.

Figure 6.16: Site locations for liquefaction study along the eastbay shoreline of San Francisco Bay (after Mitchell et al., 1994)

At three locations, POR-2, POR-3 and POR-4, shown in Figure 6.17, cone penetration were performed. At the POR-2 location, the soil which is likely to have liquefied is a fine silt of low plasticity, classified as (ML) in the Unified Soil Classification System. This soil lies approximately between 4- and 8-metre depths. At the POR-3 and POR-4 locations, the soil which is likely to have liquefied is a fine silty sand that is classified as (SM) in the Unified Soil Classification System. The soil lies between 5- and 7-metre depths at the POR-3 and POR-4 locations.

San Francisco-Oakland Bay Bridge Toll Plaza

The Bay Bridge Toll Plaza, is located approximately 72 kilometers ($R = 73.98$ km) north of the zone of fault rupture. The soil profile at this site consists of dumped fill material over soft San Francisco Bay Mud. The fill material ranges from gravelly sand to clean sand. Evidence of soil liquefaction at this site was seen in the form of sand boils, settlement and lateral spreading.

Along the highway which approaches the Toll Plaza in both east and west bound directions, large, longitudinal fissures opened up in the ground, and sand was exuded onto the pavement surface. The width of the fissures was between 2.5 and 7.6 centimetres. The lengths varied with the longest one being more than 91 metres.

Along the peninsula, settlements of as much as 41 centimetres occurred. In some cases differential settlements produced an uneven, "hummocky" pavement surface with permanent "waves" of up to 15 centimetres in amplitude. Pile supported structures

Figure 6.17: Locations of borings POR-2, POR-3 and POR-4 at the Port of Richmond site (after Mitchell et al., 1994)

located at the Bay Bridge Toll Plaza did not experience settlements; however, the foundation material surrounding these buildings settled appreciably. The highway adjacent to the Toll Plaza in the eastbound direction also experienced settlement and had to be repaved in order to bring the settled portion back up to grade. The maximum horizontal ground surface acceleration, a_{s-max} , was estimated to be 0.16 g. This a_{s-max} measurement was obtained at a strong motion recording station located at the Treasure Island which is underlain by natural, soft bay sediments and is about 72 km north of the fault rupture surface (Kramer, 1996 and Stewart, 1997).

Figure 6.18 shows the locations of the borings at this site. Based on standard penetration resistance logs and surface evidence of sand boils, liquefaction is likely to have occurred at depths between approximately 5 and 7 metres at the SFOBB-1 location and 6 and 8.5 metres at the SFOBB-2 location. The soils which are likely to have liquefied at the SFOBB-1 and SFOBB-2 locations are classified as (SPSM) and (SM) in the Unified Soil Classification System. The watertable is at approximately 3-metre depth.

The Port of Oakland

The Port of Oakland is located approximately 69 kilometers ($R = 71.06$ km) north of the fault rupture. The site was developed by dumping and/or placing sandy hydraulic fill over San Francisco Bay Mud. The cohesive deposits are believed to have amplified the levels of acceleration at the site and thus helped to contribute to the observed liquefaction of the fill.

Figure 6.18: Locations of borings SFOBB-1 and SFOBB-2 at the Bay Bridge Toll Plaza site (after Mitchell et al., 1994)

Evidence of liquefaction at this site, located as shown in Figure 6.19, appeared in the form of settlement, lateral spreading and pavement cracking. At the southwest end of the 7th Street Terminal, a small recreational park, Portview Park, suffered extensive damage. During the earthquake, lateral spreading caused the foundation soils at the park to move several feet towards the bay. Large rip-rap was dumped there to temporarily stabilize the weakened soils. After the earthquake, the park was closed permanently because it suffered extensive damage.

In addition, some large cranes which traveled along two rails, and which are used to load and unload container cargo from trucks and ships, suffered damage due to differential settlement of one of the rails. One of the rails along which the cranes operated was situated on a pile-supported concrete wharf while the other rail was situated on fill. During the earthquake, the fill-supported rail settled, but the rail which had been situated on the pile supported concrete wharf did not. This differential settlement rendered some of the cranes temporarily inoperable. At the same site, cranes which had both of their rails located on the concrete wharf, did not suffer any damage and remained fully operational.

Cone penetration testing was performed at the POO7-1, POO7-2 and POO7-3 locations, as shown in Figure 6.19. The soils at POO7-1 were classified as poorly graded sands (SP), and poorly graded sands with silt binder (SPSM), using the Unified Soil Classification System and liquefaction is believed to have occurred between 6- and 8-metre depths. At POO7-2, liquefaction is believed to have occurred between the depths

Figure 6.19: Locations of borings POO7-1, POO7-2 and POO7-3 at the Port of Oakland site (after Mitchell et al., 1994)

of 5 and 7.5 metres while at POO7-3, liquefaction is believed to have occurred between the depths of 5.3 and 8 metres. Borings POO7-1 and POO7-2 are approximately 7.6 metres apart; and Boring POO7-2 and POO7-3 are approximately 15.2 metres apart. The maximum horizontal ground surface acceleration, a_{s-max} , was estimated to be 0.16 g. This a_{s-max} measurement was obtained at a strong motion recording station located at the Treasure Island which is underlain by natural, soft bay sediments and is about 72 km north of the fault rupture surface (Kramer, 1996 and Stewart, 1997). The watertable is at approximately 3-metre depth.

Alameda Bay Farm Island

Bay Farm Island is located approximately 56 kilometers ($R = 58.52$ km) north of the zone of fault rupture. Considerable liquefaction occurred at the northwest corner and at points along the western edge of Bay Farm Island. Numerous sand boils, many of them relatively large, occurred in this area. Most of Bay Farm Island consists of sandy hydraulic fill, underlain by Bay Mud and deeper stiffer alluvium. Soil liquefaction, however, was largely confined to undeveloped, open lands. Most of the rest of the island has been developed for residential housing and light commercial use, and the fill in the developed areas was densified by vibroflotation prior to the construction of buildings. This densification, in all areas, successfully prevented soil liquefaction from damaging structures, though light damage to roadway and parking lot pavements occurred at the edge of one developed area.

Surface evidence of liquefaction in the form of sand boils was seen at the South Loop Road location. There was no surface evidence of liquefaction along the dike and based on standard penetration resistance measurements taken at this location, liquefaction was not likely to have occurred. Figure 6.20 shows the locations of the two borings DIKE and HBI-P6, for the dike and South Loop Road locations, respectively. Soil at DIKE was classified as a poorly graded sand with fines. The maximum horizontal ground acceleration, a_{s-max} , from strong motion recordings, was measured to be 0.30 g (Kayen et al., 1992). The watertable is at approximately 3-metre depth.

The measured cone penetration resistance, q_c , and friction ratio, F , profiles for all the sites are shown in Figure 6.21 to Figure 6.29 respectively.

6.3.3 Case analysis

The Loma Prieta earthquake was located at the San Francisco Bay region of California which, as a result, can be well addressed by the California ground motion model developed by Atkinson and Silva (2000). As for the same reason given for the case of the Saguenay earthquake, the cone penetration values at the beginning and the end of a liquefied layer have been determined and are tabulated in Table 6.8. Together in the table are the friction ratio, F , the total vertical in-situ stress, σ_v , and the effective vertical in-situ stress, σ'_v , at each level.

Figure 6.20: Locations of borings DIKE and HBI-P6 at Alameda Bay Farm Island site
(after Mitchell et al., 1994)

Figure 6.21: CPT q_c and F profiles of POR-2 (after Mitchell et al., 1994)

Figure 6.22: CPT q_c and F profiles of POR-3 (after Mitchell et al., 1994)

Figure 6.23: CPT q_c and F profiles of POR-4 (after Mitchell et al., 1994)

Figure 6.24: CPT q_c and F profiles of SFOBB-1 (after Mitchell et al., 1994)

Figure 6.25: CPT q_c and F profiles of SFOBB-2 (after Mitchell et al., 1994)

Figure 6.26: CPT q_c and F profiles of POO7-1 (after Mitchell et al., 1994)

Figure 6.27: CPT q_c and F profiles of POO7-2 (after Mitchell et al., 1994)

Figure 6.28: CPT q_c and F profiles of POO7-3 (after Mitchell et al., 1994)

Figure 6.29: CPT q_c and F profiles of HBI-P6 (after Mitchell et al., 1994)

Site	Depth of sand (m)	σ_v (kPa)	σ'_v (kPa)	q_c (MPa)	F (%)
SFOBB-1	5.0	93.00	73.38	5	0.6
	7.0	132.00	92.76	6	0.5
SFOBB-2	6.0	112.50	83.07	9	0.4
	8.5	161.25	107.30	10	0.4
POO7-1	6.0	112.50	83.07	11	0.4
	8.0	151.50	102.45	12	0.4
POO7-2	5.0	93.00	73.38	9	0.4
	7.5	141.75	97.61	10	0.4
POO7-3	5.3	98.85	76.29	8	0.3
	8.0	151.50	102.45	10	0.2

Table 6.8: Results of test at liquefied layers – Loma Prieta

The Port of Richmond site and Alameda Bay Farm Island site are excluded in the analysis due to the same reason as site 5 and site 6 of the Saguenay earthquake case that liquefaction had occurred throughout the whole layer of sand deposit within which the extremely low measured q_c value of the soil is far from the threshold q_c values for the occurrence of liquefaction.

Figure 6.30 is similar to Figure 6.15 showing the peak horizontal acceleration against the distance from epicenter. From this plot, the amplification, A , at Oakland located on top of a layer of Bay mud seems to range from about 2 to 3 (Rainer et al., 1990). This range is in agreement with the theoretical values (2.68 to 2.85) as obtained by the Atkinson and Silva (2000) ground motion model. As a result, values of A equal to 2.0, 2.5 and 3.0 have been used in the analysis in order to show the effects of the amplification factor on

Figure 6.30: Measured peak horizontal acceleration vs epicentral distance – Loma Prieta (after Rainer et al., 1990)

the calculated values of the magnitude, hypocentral distance and peak acceleration at the ground surface. The results obtained are tabulated in Table 6.9 to Table 6.11 and the summaries are shown in Table 6.12 to Table 6.14 respectively.

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
SFOBB-1	5.0	5.1	8.25	0.40
	7.0	5.6	16.79	0.28
SFOBB-2	6.0	6.6	33.58	0.27
	8.5	6.7	38.35	0.24
POO7-1	6.0	7.1	36.37	0.34
	8.0	7.2	41.73	0.30
POO7-2	5.0	6.6	28.04	0.33
	7.5	6.8	39.73	0.25
POO7-3	5.3	6.2	23.55	0.31
	8.0	6.7	35.86	0.26

Table 6.9: Results of analysis using $A = 2.0$ – Loma Prieta

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
SFOBB-1	5.0	5.5	16.47	0.33
	7.0	5.9	28.18	0.25
SFOBB-2	6.0	6.8	47.43	0.25
	8.5	6.9	54.18	0.22
POO7-1	6.0	7.3	51.37	0.30
	8.0	7.4	58.95	0.27
POO7-2	5.0	6.8	39.61	0.31
	7.5	7.0	56.12	0.22
POO7-3	5.3	6.4	33.27	0.29
	8.0	6.9	50.66	0.24

Table 6.10: Results of analysis using $A = 2.5$ – Loma Prieta

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
SFOBB-1	5.0	5.8	27.64	0.28
	7.0	6.1	39.81	0.23
SFOBB-2	6.0	6.9	56.37	0.25
	8.5	7.0	64.39	0.22
POO7-1	6.0	7.4	61.06	0.30
	8.0	7.5	70.06	0.26
POO7-2	5.0	7.0	55.95	0.27
	7.5	7.1	66.69	0.22
POO7-3	5.3	6.6	47.00	0.26
	8.0	7.1	71.56	0.20

Table 6.11: Results of analysis using $A = 3.0$ – Loma Prieta

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	6.9	71.06 – 73.98	0.16
Calculated (range)	5.1 – 7.2	8.25 – 41.73	0.24 – 0.40
Calculated (average)	6.5	30.23	0.30

Table 6.12: The range and average of the results ($A = 2.0$) – Loma Prieta

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	6.9	71.06 – 73.98	0.16
Calculated (range)	5.5 – 7.4	16.47 – 58.95	0.22 – 0.33
Calculated (average)	6.7	43.62	0.27

Table 6.13: The range and average of the results ($A = 2.5$) – Loma Prieta

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	6.9	71.06 – 73.98	0.16
Calculated (range)	5.8 – 7.5	27.64 – 71.56	0.20 – 0.30
Calculated (average)	6.9	56.05	0.25

Table 6.14: The range and average of the results ($A = 3.0$) – Loma Prieta

The results in Table 6.12 to Table 6.14 suggest that an amplification factor of 3 was likely operating during the earthquake. It has also been noted that a change of the amplification factor by 0.5 will only change the calculated value of M_w by about 0.2 in magnitude and a_{s-max} by about 0.02 g while the value of R is more sensitive and shows a change of more than 10 km.

6.4 THE 1995 HYGOKEN-NAMBU (KOBE) EARTHQUAKE

6.4.1 Background information

At 5:46 a.m. local time on Tuesday, January 17, 1995, an earthquake of Richter magnitude 7.2 ($M_w = 6.8$), with its epicenter at the northern part of Awaji Island, occurred (Kimura, 1996). The earthquake was a right-lateral strike-slip type and cause violent shaking in urban areas. As a result, urban facilities around Kobe city experienced serious damage. The earthquake was named by Japan Meteorological Agency (JMA) as “the 1995 Hygoken-Nambu Earthquake”. In Japanese “ken” and “nambu” mean a prefecture and southern part, respectively. It is also referred to as “the Great Hanshin-Awaji Earthquake Disaster” by Japanese Government as a reference to the disaster caused from the earthquake.

The earthquake’s epicenter was 15 km north of Awaji Island, and the hypocenter depth (the seismic source) was 14.3 km below sea level. The duration of the main seismic shaking was rather short, approximately 5 to 50 seconds (Shibata et al. 1996).

This earthquake is among the most severe natural disasters in this century. Nearly 5,500 people were killed; about 35,000 were injured; 180,000 buildings were destroyed or badly damaged, 300,000 people were rendered homeless on the night of the earthquake. The economic loss is the worst in history. Damage was reported over an area within 100 km from the epicenter, including the cities of Kobe, Osaka and Kyoto. The estimated direct damage is US\$147 billion (EQE, 1995).

6.4.2 Description of liquefied sites

One of the main causes of extensive damage in the Kobe earthquake was ground failure due to soil liquefaction as a large number of buildings settled and/or tilted, with their superstructures remaining intact. Figure 6.31 shows the distribution of buildings with foundation problems in Hyogo prefecture. The circles in the figure correspond to pile foundations, and triangles denote spread foundations. Most of the foundation problems were found to be concentrated in the reclaimed lands which indicates that soil liquefaction was the primary cause of the foundation distress. Typically, the traditional Japanese wooden houses with low foundation rigidity almost collapsed due to differential settlement. The engineered buildings and modern wooden houses with rigid foundation beams, by contrast, only settled and/or tilted with their superstructures remaining intact or with little damage. Standard penetration tests (SPT) were conducted at five locations where foundation failures due to liquefaction have been reported at Rokko Island,

Ashiyahama and Kobe City (Tokimatsu et al., 1996) respectively as shown in Figure 6.31.

Rokko Island

Rokko Island is a reclaimed island which was constructed between 1972 and 1990. It has a 3.4 by 2 km rectangular shape, covering a 580 ha area. The maximum ground surface subsidence was reported to be 3 m just behind the quay wall, which itself had moved 2 m seaward. There were many deep ground cracks parallel to the wall. Settlement between the foundation pile of the Rokko Liner new transit system and the surrounding ground level was observed. The ground surface in many of the liquefied areas settled over 30 cm, similar in magnitude to that observed in the 1964 Niigata earthquake.

Figure 6.32 and Figure 6.33 show the SPT N-value profiles at sites G and B in Rokko Island, respectively. The depth of watertable is 3.9 m and the hypocentral distance, R , is 24.59 km for these sites. The maximum horizontal ground surface acceleration, a_{s-max} , was estimated to be 0.33 g.

Ashiyahama

Shiomi Junior High School (N in Figure 6.31) is situated at the south end of Ashiyahama, and faces the shore. Figure 6.34 shows the SPT N-value profile at the site. Several fissures occurred with many sand boils parallel to the shoreline. The ground distress might have been contributed by the adjacent south quay walls that shifted towards the sea by 1.5 metres.

Figure 6.31: Distribution of buildings with foundation problems (after Tokimatsu et al., 1996)

**Figure 6.32: SPT N-value at various depth (site G)
(after Tokimatsu et al., 1996)**

**Figure 6.33: SPT N-value at various depth (site B)
(after Tokimatsu et al., 1996)**

**Figure 6.34: SPT N-value at various depth (site N)
(after Tokimatsu et al., 1996)**

The school buildings supported on prestressed concrete piles 30 metres long tilted to the southeast. The passageways connecting school buildings tilted to the south. Consequently, large horizontal openings and vertical gaps occurred at the connections between the school buildings and the passageways.

The total horizontal opening amounts to about 1 metre, which is also consistent with the differential ground displacements detected by an aerial photographic survey. Although the excavated pile-heads appeared sound, integrity sonic tests suggested that the piles had been damaged at depths. Therefore, the school buildings including the pile foundations might have shifted laterally with the liquefied fills. The depth of watertable is 4.0 m and the hypocentral distance, R , is 28.80 km for the site. The maximum horizontal ground surface acceleration, a_{s-max} , was also estimated to be 0.33 g.

Waste disposal facilities with a four-story building are located at the southeast corner of Ashiyahama (O in Figure 6.31, $R = 29.67$ km). This building (under construction) is supported on cast-in-place concrete piles 1.0 – 1.2 metres in diameter and 30 metres long with enlarged bases. Figure 6.35 shows the SPT N-value profile at the site. Ground fissures and settlements of 20 – 100 cm occurred around the building, with many sand boils on the north. Several large horizontal cracks appeared at a depth of about 10 metres as well as near the pile heads. This suggests that the fill down to a depth of about 10 metres might have liquefied and slid towards the sea, causing the piles to fail.

**Figure 6.35: SPT N-value at various depth (site O)
(after Tokimatsu et al., 1996)**

Kobe city

Many sand boils and ground fissures were obvious at American Park, Chuo-Ku, Kobe city (R in Figure 6.31), facing the Port of Kobe. Figure 6.36 shows the SPT N-value profile at the site. On the pier at the park, a 14-story building under construction was supported on cast-in-place concrete piles 2.5 metres in diameter and 33 metres long. During the earthquake, the quay walls on the west, south, and east of the building moved horizontally by 1 m, 2 m, and 0.5 – 0.6 m; and settled by 0.4 – 0.6 m, 0.5 – 0.7 m, and 0.2 – 0.3 m; respectively. The depth of watertable is 1.2 m and the hypocentral distance, R, is 20.72 km for the site. The maximum horizontal ground surface acceleration, a_{s-max} , was estimated to be 0.51 g.

6.4.3 Case analysis

The Kobe earthquake belongs to the category of shallow crustal earthquakes, and thus can be analyzed using the ground motion model by Atkinson and Silva (2000) developed for similar shallow crustal conditions in California. The in-situ soil parameter that can be obtained from this earthquake, however, is different from that of the Saguenay and the Loma Prieta earthquakes as only SPT N-values are available. This is due to the fact that CPT is seldom done in Japan (private communication with Tokimatsu, 2001). As a result, instead of determining the limiting values at the starts and ends of the liquefied layer due to the continuous nature the CPT, a representative N-value within the liquefied layer are determined for the purpose of this analysis. Table 6.15 shows the N-values

**Figure 6.36: SPT N-value at various depth (site R)
(after Tokimatsu et al., 1996)**

together with the fines content (FC), the total vertical in-situ stress, σ_v , and the effective vertical in-situ stress, σ'_v , of the liquefied layer at each site.

Site	Depth of sand (m)	σ_v (kPa)	σ'_v (kPa)	N-value	FC (%)
G	6.2	114.05	92.17	5	30
B	8.1	151.68	110.87	3	40
N	10.6	199.76	135.30	5	35
O	10.9	207.36	139.08	4	35
R	4.1	78.93	50.09	6	15

Table 6.15: Results of test at liquefied layers – Kobe

For the amplification value A of the peak acceleration at the ground surface, a plot showing the peak horizontal ground acceleration against the closest distance from the fault line was obtained as shown in Figure 6.37. From the figure, it can be observed that the peak horizontal ground acceleration on top of normal soil and soft soil to that of rock and stiff soil exists an amplification factor around 1.5 to 2.0 at a distance of about 20 km from the fault line. This range is close to the theoretical values (1.39 to 1.75) as obtained by the Atkinson and Silva (2000) ground motion model. As a result, values of A equal to 1.5 and 2.0 have been used in the analysis in order to see whether a change of 0.5 in the amplification factor will have more or less the same responses as previous cases to the change in the calculated magnitude, hypocentral distance and peak ground acceleration. The results obtained are tabulated in Table 6.16 and Table 6.17 and the summaries are shown in Table 6.18 and Table 6.19 respectively.

Figure 6.37: PHGA attenuation for each subsurface condition of the Kobe earthquake

(after Ejiri et al., 1996)

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
G	6.2	6.0	13.15	0.38
B	8.1	5.0	5.86	0.39
N	10.6	6.3	22.51	0.26
O	10.9	5.8	14.83	0.29
R	4.1	6.2	18.15	0.31

Table 6.16: Results of analysis using $A = 1.5$ – Kobe

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
G	6.2	6.4	26.25	0.31
B	8.1	5.6	16.52	0.29
N	10.6	6.6	37.79	0.23
O	10.9	6.2	29.59	0.23
R	4.1	6.6	36.22	0.24

Table 6.17: Results of analysis using $A = 2.0$ – Kobe

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	6.8	20.72 – 29.67	0.33 – 0.51
Calculated (range)	5.0 – 6.3	5.86 – 22.51	0.26 – 0.39
Calculated (average)	5.9	14.90	0.33

Table 6.18: The range and average of the results ($A = 1.5$) – Kobe

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	6.8	20.72 – 29.67	0.33 – 0.51
Calculated (range)	5.6 – 6.6	16.52 – 37.79	0.23 – 0.31
Calculated (average)	6.3	29.27	0.26

Table 6.19: The range and average of the results ($A = 2.0$) – Kobe

From Table 6.18 and Table 6.19, it appears that an amplification factor of 2.0 produces results which are closer to the actual values of the earthquake. The same conclusion can be made that the distance R is more sensitive to the change in the amplification factor than the earthquake magnitude and the peak ground acceleration.

6.5 THE 1999 CHI-CHI EARTHQUAKE

6.5.1 Background information

In the early morning (01:47 Taiwan time) of September 21, 1999, a great earthquake struck central Taiwan near the small town of Chi-Chi, Nantou County. The epicenter was located by the Central Weather Bureau (CWB) to be at 23.87° N, 120.75° E with a depth of 7 km (Lee et al., 2000). This earthquake became known as the 921 earthquake or the “Chi-Chi” earthquake. The magnitude of the Chi-Chi earthquake was $M_L = 7.3$ or $M_w = 7.6$. It is the strongest ever recorded in Taiwan for the past one hundred years. The measured maximum seismic accelerations located at lat. $23^{\circ}52.70'$ E and $120^{\circ}41.06'$ N (monitoring station TCU 129, 13 km away from the fault rupture) were 1.00 g in east-west component, 0.62 g in north-south component, and 0.34 g in vertical component, respectively (Ni and Lai, 2000). It was responsible for over 2,400 fatalities; more lives would have been lost if the earthquake had occurred during daylight hours. In the first few days following the earthquake, there were many major aftershocks, several over magnitude 6.5. Damage estimates, including calculations for lost productivity, range between US\$20 and \$30 billion. In recent centuries, large earthquake have been most

frequent on the eastern side of Taiwan, but this epicenter was near the center of the island, closer to the major population areas along Taiwan's west and north coasts.

The earthquake was caused by the rupture of the Chelungpu fault. The main part of the fault is a north-south-striking thrust fault that dips about 30 degrees to the east. It is part of the western thrust zone that accommodates some of the crustal shortening caused by the collision of the Philippine Sea Plate and the Eurasia Plate. At the north end of the rupture, there is a sharp bend to the east and the fault rupture strikes nearly east-west. The total length of the rupture is about 80 km, with a down-dip width of about 40 km. The epicenter is located about 15 km from the south end of the fault.

Evidences of soil liquefaction such as sand boils were widely observed in this earthquake. Damage caused by liquefaction include failure of riverbanks, damages to roadways, pipelines, and port facilities, and building settlement and tilting. More than 900 dwellings were severely damaged by the effect of soil liquefaction.

6.5.2 Description of liquefied sites

The liquefaction cases of Chi-Chi earthquake are more widespread and serious than other previous records in Taiwan. Hence, these liquefaction phenomena attracted much attention from society and geotechnical engineering circles. The distribution of liquefaction sites in central Taiwan, including Miaoli, Taichung, Nantou, Zhanghua,

Yunlin, and Jiayi County, is shown as Figure 6.38. Discussions of liquefaction cases in Nantou City and Zhanghua County (Ni and Lai, 2000) are as follows.

Nantou City

The distribution of liquefaction failures induced by Chi-Chi Earthquake in Nantou City is mainly along the sides of Maoluo River and its branches (Figure 6.39). Jungongliao (Jungong-Li), which is about 15 km ($R = 16.55$ km) from the fault rupture, is the most serious area of liquefaction failures in Nantou City. According to the data from the monitoring station at Nantou Elementary School (TCU 076, 15.25 km away from the epicenter), the PGA is 0.347 g in E-W component, and 0.428 g in N-S component. It is thus clear that the seismic wave of Chi-Chi Earthquake in Nantou City was very violent.

Many buildings tilted and settled with a great deal of sand boils in Jungongliao. A new three-story building at the north end of Jungong Bridge tilted 3° backward (southeast). A two-story building at no. 308 on Chungshin Road settled non-uniformly. The front tilted 6° southwest, and the back tilted 7° northeast, i.e. the middle part had more settlement. Besides building damages, there were settlement of embankments, sand boils on the flood plain, lateral spreading of soil, and failure of revetments along the bank of Maoluo River.

The horizontal PGA was very large (0.428 g), it is possible that Jungongliao is on the alluvial plain of Maoluo River, therefore liquefaction failure is very serious. Figure 6.40 and Figure 6.41 show the SPT N-value profiles of the flood plain of Maoluo River (BH-

Figure 6.38: Distribution of liquefaction sites in central Taiwan during Chi-Chi earthquake (after Ni and Lai, 2000)

Figure 6.39: Distribution of liquefaction sites in Nantou City (after Ni and Lai, 2000)

Figure 6.40: SPT N-value at various depth (BH-7)
(after Ni and Lai, 2000)

**Figure 6.41: SPT N-value at various depth (BH-14)
(after Ni and Lai, 2000)**

7) and Jungongliao (BH-14) in Nantou City. The depths of watertable of the two sites are at 0.6 m and 1.5 m, respectively.

Zhanghua County

The major soil liquefied area in this county during the earthquake is located at the western foothill of Bagua Mountain. The towns from north to south are Datsun Town, Yuanlin Town, and Shetou Town (Figure 6.42). The soil layer is an alluvial deposit that is formed by Zhuoshui River. Bagua Mountain is located between Zhanghua County and Nantou County. The soil at the western foothill of Bagua Mountain was pushed up during earthquake. The strong seismic wave (force) may have caused the soil liquefaction along the foothill. The three liquefied towns are described as below:

- *Datsun Town* is located at the western foothill of Bagua Mountain. Liquefaction sites were mainly found in Huangtsuo and Meigang Village. The distance from the site to the fault rupture is about 28 km ($R = 28.86$ km). The closest monitoring station is located at Yuanlin Elementary School (TCU110, 27.5 km away from the epicenter). The PGA was 0.192 g in E-W component, and 0.181 g in N-S component. Many houses and buildings are tilted and settled. A two-story building located at Meigang Road settled uniformly due to soil liquefaction. However, no serious damage was found in the structure except settlement. Many obvious sand boils were also found in the free field in this area. Figure 6.43 and Figure 6.44 show the SPT N-value profiles of Huangtsuo Village (BH-5) and

Figure 6.42: Distribution of liquefaction sites in Zhanghua County (after Ni and Lai, 2000)

**Figure 6.43: SPT N-value at various depth (BH-5)
(after Ni and Lai, 2000)**

**Figure 6.44: SPT N-value at various depth (BH-3)
(after Ni and Lai, 2000)**

Meigang Village (BH-3) in Datsun Town. The depths of watertable of the two sites are at 1.0 m and 0.6 m, respectively.

- *Yuanlin Town* is also located at the western foothill of Bagua Mountain. It is the most seriously liquefied area at the western foothill of Bagua Mountain. Lunya-Li is the most seriously liquefied village in this town. The site is about 27.5 km ($R = 28.38$ km) away from the fault rupture. The closest monitoring station is also located at Yuanlin Elementary School (TCU 110). In Lunya-Li, houses or buildings tilted or settled, sand boiled hole, and in-filled drainage with boiled sand could be seen everywhere. An old well settled due to liquefaction with the top of the well went from 1 metre down to 15 cm. The ground surface around the well also subsided and cracked due to the liquefied soil. Figure 6.45 and Figure 6.46 show the SPT N-value profiles of Lunya-Li (BH-28 and BH-29) in Yuanlin Town. The depths of watertable of the two sites are at 2.3 m and 2.0 m, respectively.
- *Shetou Town* is also located at the western foothill of Bagua Mountain. The distance from the town to the fault rupture is about 23 km ($R = 24.04$ km). The closest monitoring station is located at Nanya Elementary School (TCU138, 23.54 km away from the epicenter). The PGA was 0.206 g in E-W component, and 0.211 g in N-S component. The major liquefied place is located in Shanhu Village. Typical damages due to soil liquefaction are the settlement and tilting of buildings. Figure 6.47 shows the SPT N-value profile of Shanhu Village (BH-40) in Shetou Town. The depth of watertable is at 1.2 m.

**Figure 6.45: SPT N-value at various depth (BH-28)
(after Ni and Lai, 2000)**

**Figure 6.46: SPT N-value at various depth (BH-29)
(after Ni and Lai, 2000)**

**Figure 6.47: SPT N-value at various depth (BH-40)
(after Ni and Lai, 2000)**

6.5.3 Case analysis

The Chi-Chi earthquake, same as the Loma Prieta and the Kobe earthquakes, belongs to the category of shallow crustal earthquakes and therefore the ground motion model developed by Atkinson and Silva (2000) for shallow crustal California earthquakes was used in the analysis. The in-situ soil parameter is also given by the SPT N-value as CPT data for these sites are not yet available as of this date (2001). As a result, representative N-values within the liquefied layers are determined as shown in Table 6.20. Also shown in the table are the fines content (FC), the total vertical in-situ stress, σ_v , and the effective vertical in-situ stress, σ'_v , of the liquefied layer at each site.

Site	Depth of sand (m)	σ_v (kPa)	σ'_v (kPa)	N-value	FC (%)
BH-7	5.3	103.19	56.98	12	13
BH-14	5.8	110.04	68.45	11	34
BH-5	4.8	91.50	54.61	6	49
BH-3	2.8	52.92	31.73	6	49
BH-28	5.9	112.14	75.65	5	43
BH-29	5.7	108.15	71.85	4	43
BH-40	4.3	82.08	51.47	6	28

Table 6.20: Results of test at liquefied layers – Chi-Chi

For obtaining the amplification value A of the peak acceleration at the ground surface, again, a plot showing the peak horizontal acceleration against the closest distance to the rupture surface is drawn as shown in Figure 6.48. This figure presents a large number of data points all together and thus is not as clear as similar plots for the previous earthquakes. From that, the amplification factor for the peak horizontal ground

Figure 6.48: Peak horizontal accelerations of motions recorded at various sites during the Chi-Chi earthquake (after Idriss and Abrahamson, 2000)

acceleration on top of soil (Category D) and soft soil (Category E) to that of rock (Category B) and stiff soil (Category C) is estimated to be about 1.0 to 1.5 at a distance of around 20 km from the fault line. This range of the amplification factor can actually be implemented from the theoretical values (0.56 to 1.53) as obtained by the Atkinson and Silva (2000) ground motion model. As a result, values of A equal to 1.0 and 1.5 have been used in the analysis in order to compare the difference in between the calculated values. The results obtained are tabulated in Table 6.21 and Table 6.22 and the summaries are shown in Table 6.23 and Table 6.24 respectively.

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
BH-7	5.3	7.1	17.24	0.41
BH-14	5.8	7.3	14.03	0.58
BH-5	4.8	6.4	12.64	0.36
BH-3	2.8	6.5	9.34	0.53
BH-28	5.9	5.7	7.32	0.38
BH-29	5.7	5.1	4.03	0.42
BH-40	4.3	6.2	10.85	0.36

Table 6.21: Results of analysis using $A = 1.0$ – Chi-Chi

Site	Depth of sand (m)	Calculated M_w	Calculated R (km)	Calculated a_{s-max} (g)
BH-7	5.3	7.5	34.40	0.35
BH-14	5.8	7.7	28.00	0.50
BH-5	4.8	6.9	29.97	0.28
BH-3	2.8	7.0	22.14	0.43
BH-28	5.9	6.3	20.62	0.29
BH-29	5.7	5.9	16.06	0.28
BH-40	4.3	6.7	25.73	0.30

Table 6.22: Results of analysis using $A = 1.5$ – Chi-Chi

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	7.6	16.55 – 28.86	0.192 – 0.428
Calculated (range)	5.1 – 7.3	4.03 – 17.24	0.36 – 0.58
Calculated (average)	6.3	10.78	0.43

Table 6.23: The range and average of the results ($A = 1.0$) – Chi-Chi

Parameter	M_w	R (km)	a_{s-max} (g)
Actual	7.6	16.55 – 28.86	0.192 – 0.428
Calculated (range)	5.9 – 7.7	16.06 – 34.40	0.28 – 0.50
Calculated (average)	6.9	25.27	0.35

Table 6.24: The range and average of the results ($A = 1.5$) – Chi-Chi

Comparing Table 6.23 and Table 6.24 with emphasis on the range and average values of the distance R and the a_{s-max} , it can be seen that an amplification factor of 1.5 is a better representation of the actual situation.

6.6 ANALYSIS ON OTHER EARTHQUAKES

To illustrate its general applicability, the proposed novel method is applied to other earthquakes selected from the database created in Chapter 3 for the development of the new energy liquefaction potential assessment method. Liquefaction cases having the seismic energy function, S , close to the line of demarcation are chosen. This will ensure a choice of cases in which the SPT values are sufficiently close to the threshold value for liquefaction occurrence. Such a choice would lead to a meaningful case study. Two amplification factors, namely 1.5 and 2.0 are assumed for all the selected cases and the

analysis has been conducted using the ground motion model of Atkinson and Silva (2000) developed for shallow crustal earthquakes. Based on these assumptions, the earthquake magnitude, M_w , and the hypocentral distance, R , have been calculated. Table 6.25 lists the selected earthquake cases and the related information while Table 6.26 lists the calculated values of M_w and R for both cases of the A value. Figure 6.49 compares the calculated and the measured values of M_w . The comparison shows that the calculated values agree well with the actual measured values as in the above four cases.

Case No.	Earthquake	Date	Measured M_w	Measured R (km)	Depth of water table (m)	Depth of sand (m)	N-value	FC (%)	$(N_{1.60})_{cs}$
1	Mino-Owari	10/27/91	7.8	30.0	0.8	13.7	17	5	19.40
4	Mino-Owari	10/27/91	7.8	51.7	0.9	13.7	17	0	17.83
21	Kwanto	9/1/23	7.9	72.0	1.0	5.0	12	5	17.30
27	Nishi-Saitama	9/21/31	6.6	19.0	0.0	4.5	8	5	14.40
76	Niigata	6/16/64	7.5	62.0	1.0	10.0	15	2	14.30
111	Haicheng	2/4/75	7.0	56.6	1.5	8.2	11	9	11.74
135	Tangshan	7/27/76	7.6	18.6	1.5	6.5	17	7	19.30
139	Tangshan	7/27/76	7.6	17.2	0.9	3.5	14	9	22.02
144	Tangshan	7/27/76	7.6	17.0	1.1	6.0	18	5	21.40
145	Tangshan	7/27/76	7.6	45.2	0.7	8.5	15	5	15.60
153	Tangshan	7/27/76	7.6	47.1	4.5	10.0	18	5	14.90
173	Tangshan	7/27/76	7.6	66.3	2.3	7.0	15	2	15.60
216	Miyagiken-Oki 2	6/12/78	7.5	114.0	0.0	4.0	8	0	14.20
255	Imperial Valley	10/16/79	6.7	28.0	1.5	2.3	7	24	14.81

Table 6.25: Selected earthquake cases and the related information

Case No.	Earthquake	Date	Measured M_w	Measured R (km)	Calculated		Calculated	
					M_w	R	M_w	R
					A = 1.5		A = 2.0	
1	Mino-Owari	10/27/91	7.8	30.0	7.9	37.0	8.1	52.2
4	Mino-Owari	10/27/91	7.8	51.7	7.7	37.2	7.9	52.6
21	Kwanto	9/1/23	7.9	72.0	7.3	31.0	7.5	43.8
27	Nishi-Saitama	9/21/31	6.6	19.0	6.8	33.2	7.1	55.7
76	Niigata	6/16/64	7.5	62.0	7.2	39.5	7.4	55.8
111	Haicheng	2/4/75	7.0	56.6	6.5	27.5	6.8	46.2
135	Tangshan	7/27/76	7.6	18.6	7.6	28.6	7.8	40.4
139	Tangshan	7/27/76	7.6	17.2	7.5	19.4	7.8	32.6
144	Tangshan	7/27/76	7.6	17.0	7.8	28.7	8.0	40.5
145	Tangshan	7/27/76	7.6	45.2	7.3	36.5	7.5	51.6
153	Tangshan	7/27/76	7.6	47.1	7.2	29.3	7.4	41.4
173	Tangshan	7/27/76	7.6	66.3	7.2	30.6	7.4	43.2
216	Miyagiken-Oki 2	6/12/78	7.5	114.0	6.8	33.7	7.0	47.6
255	Imperial Valley	10/16/79	6.7	28.0	6.2	11.2	6.6	22.3

Table 6.26: Results of analysis on selected earthquake cases

6.7 SUMMARY REMARKS

The above four case studies and the general analysis on other earthquakes have demonstrated the novel method proposed in this thesis can produce promising results on the characteristics of past earthquakes. This novel method utilizes both geotechnical and seismological principles in order to create conditions for a unique set of the required parameters to satisfy. However, one has to note that this method is applicable only to

Figure 6.49: Plot calculated M_w vs measured M_w

sites on level to gently sloping ground with liquefied sand deposits at depths ranging from 1 to 15 m. These are the conditions that form the basis for the development of all the geotechnical liquefaction potential assessments involved in the novel method.

Through the analysis of the four recent earthquakes that have recorded ground motion data and other earthquake cases from the database, it can be concluded that an amplification factor of 1.5 to 2 can be used for general soils such as alluvium and fill deposits, while a higher amplification factor of 3 has to be adopted when dealing with highly compressible soils such as the Bay Mud in the case of the Loma Prieta earthquake. The fact is, as far as a reasonable A is chosen, the calculated value for the earthquake characteristics lie within an acceptable margin.

Chapter 7

CONCLUSIONS AND SUGGESTIONS FOR FUTURE RESEARCH

7.1 CONCLUSIONS

A new energy approach for assessing seismic liquefaction potential has been developed in this thesis. It is based on a large database of 160 liquefaction cases and 203 non-liquefaction cases. The energy attenuation parameter used in the new approach was evaluated from recently developed strong motion models and thus provides a much better estimation of the seismic energy arriving at a site. This eliminates the use of different attenuation parameter for the Eastern and the Western North America. With the use of equivalent clean sand SPT N-values, the new method can assess the liquefaction potential of sandy soils having different fines content using the same equation. This improves the use of different equations for different soil types in current methods.

The existing correlations between SPT and CPT values are also investigated in the thesis. It has been found that they are far from precise to be adopted in the novel method to

assess past earthquake characteristics. Besides, all of them are developed using all kinds of soil rather than liquefiable soils only. A new correlation is thus developed based on case records of liquefiable soils only. The new correlation is a linear function between the equivalent clean sand SPT value and the equivalent clean sand CPT value. Hence, it is easier to apply than the existing methods. Furthermore, this new correlation allows the conversion from CPT to SPT data without the requirement of additional index testing in order to determine the grain characteristics of the soil. Thus the requirement of grain size information through soil sampling is eliminated. Validations of the new correlation have been performed via other soil parameters and show that good agreement existing between the empirical correlations which are obtained indirectly by using correlations that are established through independent sources of in-situ data and back-analyses by different investigators.

The main contribution of this thesis is the novel method for the assessment of past earthquake characteristics based on a combination of geotechnical and seismological principles. The phenomenon of earthquake-triggered liquefaction is examined and a set of equations has been established that integrates geotechnical and seismological equations together. A total of four independent equations are obtained and the solution of the equations is possible with information commonly obtained in a geotechnical ground investigation program. The resulting earthquake parameters are the moment magnitude, maximum peak horizontal acceleration and the hypocentral distance of an earthquake that had caused soil liquefaction.

Four case studies have been conducted in order to examine the applicability of the proposed method which includes the 1988 Saguenay earthquake, the 1989 Loma Prieta earthquake, the 1995 Hyogoken-Nambu (Kobe) earthquake and the 1999 Chi-Chi earthquake. Comparison of the calculated seismic parameters with actual measurements from the four cases shows that this method has a good potential for assessing the parameters of past earthquakes. Further analysis has also been done on different earthquakes selected from the database created for the development of the new energy liquefaction potential assessment method. From that it can be observed that the calculated values agree well with the actual measured values as in the four case studies. It can therefore be concluded that the novel method provides a useful assessment of the characteristics of past earthquakes that occurred all over the world.

7.2 SUGGESTIONS FOR FUTURE RESEARCH

The work presented in this thesis can be extended with the following research activities:

- Perform field investigations at liquefied sites that were induced by past earthquakes especially for those earthquakes that were recorded qualitatively. This will provide a potentially improved method for reassessing the characteristics of these earthquakes.
- Perform field investigations to measure both SPT and CPT together at liquefied sites induced by earthquakes. This can help to strengthen the correlation between

the SPT and the CPT values proposed in this thesis. Since this method eliminate the need of sampling and defining the grain size characteristics, it would be very useful in converting the vast SPT based experience into useful experience for CPT application.

- Increase the number of simultaneous independent equations in the novel method. Not only does this allow more earthquake parameters to be determined but this can also increase the number of constraint to the set of equations and thus increase the accuracy of the calculated values.

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