

**EMPIRICAL DESIGN CHARTS AGAINST EARTHQUAKE-INDUCED
LIQUEFACTION IN COHESIONLESS SOILS BASED ON IN-SITU**

TESTS

A Thesis

by

JOSE RAFAEL MENENDEZ

Submitted to the Office of Graduate Studies of
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Major Subject: Civil Engineering

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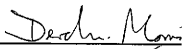
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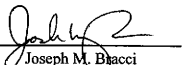
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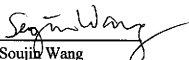
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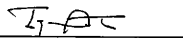

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ABSTRACT

Empirical Design Charts Against Earthquake-Induced Liquefaction in Cohesionless Soils

Based on In-Situ Tests. (May 1997)

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Available methods to predict the liquefaction susceptibility of cohesionless soils are based either in empirical charts (in-situ test) or laboratory tests. In-situ tests are a valuable source of information; especially in cohesionless soils, due to the expensive and complicate procedures to obtain an undisturbed sample.

During the last years, different work has been done in the development of relationships between in-situ tests and probability of liquefaction. This work deals with the development of empirical design charts based on the database published in the references for two of the most common in-situ tests, the standard penetration test (SPT) and the cone penetration test (CPT). The statistical methods used in order to develop the charts were the Discriminant Analysis and the Logistic Regression.

To my parents, Juan J. and Libertad, especially to my father who encouraged me to follow his steps in geotechnical engineering.

To my wife, Rocio, for her love and permanent support.

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I wish to express my deepest appreciation for the guidance, and encouragement of my Committee Chair, Professor Derek Morris. This work would not have reached its goals without his ideas and strong support. I must also thank Dr. Morris for his encouragement and advice in non-academic areas which are essential to a good engineer. I should give special thanks to my Committee Co-Chair, Professor Albert T. Yeung for his help and guidance. My sincere appreciation and thanks to Professor Norman Tilford for his patience and time. I also wish to express my gratitude for their help and understanding to Dr. Joseph Bracci and to Dr. Soujin Wang.

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TABLE OF CONTENTS

	Page
ABSTRACT	iii
DEDICATION	iv
ACKNOWLEDGMENTS	v
TABLE OF CONTENTS	vi
LIST OF FIGURES	viii
LIST OF TABLES	x
 CHAPTER	
I INTRODUCTION	1
1.1 Objectives	1
1.2 Basic Concepts	2
1.3 Mechanisms of Soil Liquefaction	2
1.4 Factors Affecting Soil Liquefaction	4
II METHODS FOR EVALUATION OF LIQUEFACTION	12
2.1 Simplified Procedures	12
2.2 Methods Based on In-Situ Tests	20
2.3 Probabilistic and Statistical Analysis	30
III DATA ANALYSIS	35
3.1 Problems Associated with Data Collection	35
3.2 Analysis of Data Characteristics	36
3.3 Data Catalogs	42
3.4 Statistical Procedures	42

CHAPTER	Page
IV EMPIRICAL CHARTS	48
4.1 Model Formulation	48
4.2 Cone Penetration Test (CPT).....	49
4.3 Standard Penetration Test (SPT)	58
4.4 Discussion of Results.....	66
V SUMMARY AND CONCLUSIONS.....	79
REFERENCES	82
APPENDIX	93
VITA	124

LIST OF FIGURES

FIGURE	Page
1 Effect of Confining Pressure	5
2 Drainage Effect on Cyclic Shear Strength	8
3 Effect of Seismic History on Cyclic Load Characteristics of Sand	10
4 Correlation Between Stress Ratios and N_1 Values for Clean Sands and $M=7\ 1/2$	13
5 Relationship Between Maximum Shear Stress Ratio and Residual Pore Pressure	18
6 Comparison of Curves Proposed by Different Researchers	21
7 Summary Chart for Evaluation of the Cyclic Strength of Sands Based in Normalized CPT q_{c1} value.....	23
8 Resistance to Liquefaction of Sand as Function of Relative Density, Dilation Angle or Penetration Resistance	25
9 Proposed Correlation Between Cyclic Stress Ratio and Cumulative Strain at 10 Cycles.....	26
10 Proposed Correlation Between Liquefaction Resistance Under Level Ground Conditions and Dilatometer Horizontal Stress Index for Sands	28
11 Correlation Between Field Liquefaction Behavior of Sands for Level Ground Conditions and Electrical Parameter Defining Void Ratio, Anisotropy and Shape.....	29
12 Cone Data Used in Statistical Analysis	51
13 Lower Boundary for Mean Grain Size (D_{50}) and Cone Resistance (q_{c1}).....	53
14 Discriminant Curves for Model 5	55
15 Logistic Curves for Model 5	57
16 Standard Penetration Data Used in Statistical Analysis.....	60

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FIGURE	Page
17 Lower Boundary for mean Grain Size and Standard Penetration Resistance	61
18 Discriminant Curves for Model 5	64
19 Logistic Curves for Model 5	65
20 Statistical Curves and Data Cases for $D_{50}=0.10-0.30$ mm.....	67
21 Statistical Curves and Data Cases for $D_{50}=0.20-0.40$ mm	68
22 Comparison of empirical curves for CPT Data and $D_{50}=0.25$ mm.....	69
23 Statistical Curves and Data Cases for $D_{50}=0.10-0.30$ mm (SPT).....	71
24 Statistical Curves and Data Cases for $D_{50}=0.20-0.40$ mm (SPT).....	72
25 Comparison of empirical curves for SPT Data and $D_{50}=0.25$ mm.....	75
26 Correlation Between q_{c1} , $(N_1)_{60}$ and CSRN	77
27 Design Chart Against liquefaction for CPT and SPT Values.....	78

LIST OF TABLES

TABLE		Page
1	Soil Liquefaction Based on the Index I_L	16
2	In Situ Test to Asses Liquefaction Potential	20
3	Summary of Liquefaction Source Catalogs	42
4	Classification of Observations	52
5	Discriminant Model for CPT Data	54
6	Logistic Models for CPT Data	56
7	Classification of SPT Observations	62
8	Discriminant Model for SPT Data	62
9	Logistic Model for SPT Data	63
10	Seed and Idriss (1971) Data Set	94
11	Tokimatsu and Yoshimi (1983) Data Set	96
12	Shibata and Teparaksa (1988) Data Set	101
13	Reyna (1991) Data Set	107
14	Liao (1986) Data Set	110
15	Tokimatsu et al. (1994) Data Set	122
16	Arulanandan et al.(1994) Data Set	123

CHAPTER I

INTRODUCTION

1.1 OBJECTIVES

The main purpose of this work is to develop design charts against earthquake-induced liquefaction in cohesionless soils based on in-situ tests. The data necessary to create the charts were collected from data catalogs available in the references. Two statistical models were applied in order to analyze the data, discriminant analysis and logistic regression. Statistical methods were chosen over probabilistic methods since the objective was to establish a boundary between soils susceptible to fail by liquefaction and soils which have an adequate behavior against earthquake action.

Discriminant analysis and logistic regression had been previously used to elaborate charts given adequate results (Liao 1986, Reyna 1991). In this work the variables selected as representatives of soil behavior to analyze the liquefaction phenomenon were the normalized cyclic stress ratio (CSR_N), the mean particle size (D_{50}), the normalized penetration resistance for the standard penetration test (N_{160}) and the normalized cone resistance for the cone penetration test (q_{c1}).

Although the charts are empirically based, there are practically oriented and do not have the intention to substitute the in-situ and laboratory testing, but represent an aid in the design process.

The citations of the following pages follow the style of the *Journal of Geotechnical Engineering*.

1.2 BASIC CONCEPTS

During earthquakes the shaking of ground may cause loss of strength or stiffness that results in the settlement of buildings, landslides, failure of earth dams and other hazards. The process leading to such loss of strength or stiffness is called soil liquefaction. It is a phenomenon associated primarily, but not exclusively, with saturated cohesionless soils. The word liquefaction, as generally used, includes all phenomena involving excessive deformations or movements as a result of transient or repeated disturbance of saturated cohesionless soils.

Seed (1976) defines liquefaction as follows: "Liquefaction: denotes a condition where a soil will undergo continued deformation at a constant low residual stress or with no residual resistance due to the build-up and maintenance of high pore water pressure which reduce the effective confining pressure to a very low value; pore pressure build-up leading to true liquefaction of this type may be due either to static or cyclic stress applications".

Castro and Poulos (1977) define liquefaction in the following way: "Liquefaction is a phenomenon wherein a saturated sand loses a large percentage of its shear resistance (due to monotonic or to cyclic loading) and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as its reduced shear resistance". Casagrande (1975) defines liquefaction as follows: "It is the response of loose, saturated sand when subjected to strains or shocks that results in substantial loss of strength and in extreme case leads to flow slides".

1.3 MECHANISMS OF SOIL LIQUEFACTION

Whitman (1985) defines two types of failures that involve soil liquefaction: "Disintegrate failure" is the condition where a soil mass can deform continuously under a shear stress less than or equal to the static shear stress applied to it. "Non-disintegrative failures" involve unacceptably large permanent displacements or settlements during (and/or immediately after) shaking, but the earth mass remains stable following shaking without great changes in geometry.

The ground failure caused by liquefaction may be manifested in several forms. *Sand Boils* are evidence of elevated pore pressure in the soil and an indication that liquefaction has occurred. *Flow failures* may be composed of completely liquefied soil or blocks of intact material riding on a layer of liquefied soil. *Lateral Spreads* involves lateral displacement of large, superficial blocks of soils as a result of liquefaction in a subsurface layer.

Ground Oscillation appears where slopes are too gentle to allow lateral displacement, liquefaction at depth commonly decouples overlying soil blocks, allowing them to jostle back and forth on the liquefied layer during an earthquake. *Loss of bearing capacity* is produced when the soil supporting a building or structures liquefies and loses strength, large soil deformations can occur, allowing the structure to settle and tip.

Buoyant rise of Buried Structures is present in tanks, pipelines, cut-off timber piles, and other buried structures that are lighter in weight than the surrounding soil rise buoyantly when the surrounding soil liquefies. *Ground Settlement* can be present while subsidence from tectonic movement occurred over a wide area.

The National Research Council (1985) described three different possible failure mechanisms developing during earthquakes: globally drained, globally undrained, and locally undrained. In the first mechanism, loose sands densify under dynamic loading and internally develop high seepage gradients that could reach critical levels at exposed exit points.

In the second mechanism, with the sand layer not allowed to drain freely, the sand could virtually settle away from the overlying containment layer, thus spontaneously developing a fluidized zone at the contact of the layers. In the third mechanism, the sand layer develops high excess pore pressures internally during the shaking; this can reduce the effective stress and cause a corresponding loss of shear strength in the sand.

1.4 FACTORS AFFECTING SOIL LIQUEFACTION

The major factors associated with the liquefaction of saturated cohesionless soils appear to be: cyclic shear stress level, initial effective confining pressure, initial relative density, drainage conditions, and previous strain-stress history. Less importance have soil grain characteristics such as particle size, shape, and gradation. The foregoing factors reflect the physical properties of the soil, the initial stress condition, stratigraphy in the ground, and the characteristics of the applied earthquake motions.

1.4.1 Cyclic Shear Stress Level

The fundamental concept of liquefaction is based upon the shear strain/volumetric-strain coupling exhibited by soils. The process of pore pressure buildup, leading to liquefaction under cyclic loading, is dependent upon the volumetric strain response under applied shear stresses. The residual increment of pore water pressure generated by an applied dynamic shear stress cycle is, under undrained conditions, related to the shear strain that is, in turn, related to the magnitude of that stress cycle. In the field the magnitude of dynamic shear stress may be ascertained from the acceleration levels, either by rough approximation or by more sophisticated computer analysis.

In the laboratory, the applied shear stress levels are defined according to the type of test. In triaxial testing the applied shear stress is taken as one-half the maximum deviator stress excursion (when symmetric stress reversals are used). Laboratory testing procedures generally simulated shaking in only one direction, whereas actual earthquake motions may have components all three principal directions. The conclusion that the most critical stresses from a liquefaction viewpoint arise from vertically propagating horizontal shear waves appears to be relatively satisfactory. Vertical stress components are not considered significant since these are of a dilatational nature and completely absorbed by the pore water.

1.4.2 Initial Effective Confining Stress

The resistance of a soil to liquefaction under cyclic loading has been noted to be a function of the effective confining pressure, prior to application of shear (Fig. 1). Although larger confining stresses would seem to enhance volume decrease and, hence, liquefaction (at least under monotonic loading conditions), under cyclic loading this is apparently more than offset by other factors such as the increasing level to which the pore pressure must be generated to achieve instability.

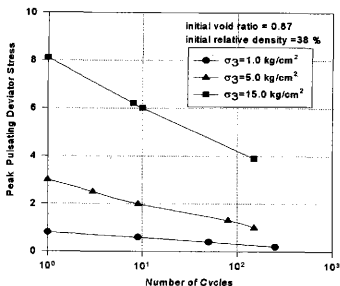


FIG.1. Effect of Confining Pressure (Adapted from Lee, K.L., and Seed H.B. (1967). "Cyclic stress conditions causing liquefaction of sand." *J. Soil Mech. and Found. Div.*, ASCE, 93(SM1), 47-70.)

1.4.3 Initial Relative Density

The relative density of a soil appears to be one of the major factors regarding liquefaction potential of cohesionless sands. Relative density is stressed here rather than absolute density since it is actually the pore volume of the soil compare to its

minimum and maximum possible pore volumes that is of significance. The denser a soil, the lower is its tendency toward volume contraction during shearing; the lower is the pore pressure that will be generated; hence, the more unlikely to liquefy (Fig. 2).

Relative density can be controlled in laboratory using reconstructed samples; however, in typical field situations with complex stratification, relative density may lose its meaning. It is also conceivable that there is an upper limit of relative density D_r , above which a soil under field behavior will either no longer tend to compress and generate pore pressure or will, immediately upon commencing yielding, undergo volume increases which prohibit liquefaction. It is impossible to define an upper limit to D_r beyond which liquefaction will not occur.

1.4.4 Characteristics of the Shear Stress Record

Earthquake ground motions generally consist of a number of randomly distributed peak stress cycles of varying shapes and magnitudes. Difficulties involved in analyzing the various random earthquake ground motions have led to attempt to express earthquakes in terms of an equivalent number of uniform stress cycles. The number of significant cycles in a particular earthquake record depends directly upon the frequency content and the duration of loading. These, in turn, are related to the magnitude of the earthquake, the distance of its epicenter, and the nature of the material through which the stress waves must propagate.

There are some weakness in simulating random earthquake motions in terms of uniform cycles. Martin et al. (1975) noted that the tendency for dry sands to undergo volume change is a direct function of dynamic shear strain level. Dynamic shear strain level is a function of soil modulus of rigidity G , which in turn depends upon the effective confining stress level and, hence, the pore water pressure generated. Since the pore pressure level existing at the time of application of a specific peak is very important, the relative position of any peak in a sequence of loading cycles is significant. The previous discussion of the effects of the stress

reversals also suggests that the peculiar characteristics of the loading history may be significant.

Perhaps, for this reason, field observations of liquefaction of level ground have generally been limited to relatively shallow depths in few cases below 15 to 20 meters. In the isotropically consolidated triaxial test the effective confining stress prior to application of shear stress is the difference between the chamber pressure σ_3 and any back pressure applied to the pore fluid. The shear stress level required to cause liquefaction in remolded sand specimens at relative density less than 80% has been found to vary linearly with confining stress levels (Seed and Lee 1966, and Peacock and Seed 1968). Therefore it has been found convenient to normalize the effects of dynamic cyclic shear stress level with the value of initial effective confining stress. It is important to recognize that the use of this normalized ratio may not always be applicable to field conditions, particularly where strongly developed structure or cementation is present.

1.4.5 Drainage Conditions

The rate at which pore water pressure is permitted to dissipate from within the soil body has a major influence upon whether or not liquefaction can occur, particularly under cyclic loading (Wong and Seed 1975). Since the rate of pore pressure dissipation is known to be a function of the square of the longest drainage path, the detail geometry of the soil profile is also important (Fig. 2).

The conventional type of liquefaction test on saturated sands is performed under conditions where there is no drainage. This, it cannot be made to represent field conditions where there is some dissipation of pore pressure during the loading period. When the drainage effect of a sample is evaluated from the laboratory test, the following differences in the drainage conditions will be found:

(1) The influx of pore water into the element is zero, and (2) the pore pressure buildup is only within the limits of the specimen (Fig. 2). It is concluded, that the partially drained cyclic strength obtained from the laboratory test indicates the

maximum potential cyclic strength of the element in the deposit except in the case of extremely low values of the loading frequency or for non-uniform deposits (Zen et al. 1985).

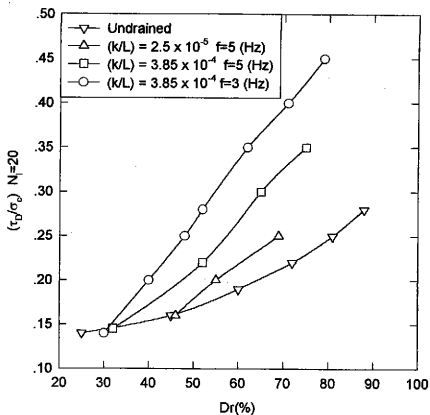


FIG. 2 Drainage Effect on Cyclic Shear Strength (Adapted from: Zen, K., Umehara, Y., and Ohneda, H. (1985). "Evaluation of drainage effect in sand liquefaction." *Proc. 11 Int. Conf. Soil Mech. and Found. Engrg.*, San Francisco, 4, 1931-1934.)

1.4.6 Grain Characteristics

At low relative densities, poorly graded water-deposit sand was found to have a lower cyclic strength than the well-graded sands at the same relative density. The

opposite trend was observed at higher relative density. Contractive deformation occurred during cyclic loading in the poorly graded sand over a range of relative densities from its loosest deposition state up to a relative density of about 43%. The more well graded sands showed strain development due only to cyclic mobility over the same range of relative densities. This implies that gradation may control the occurrence of contractive deformation and, hence, possible flow failure at low relative densities (Vaid et al. 1990).

Under normal triaxial test conditions, fine silty sands appear to be most susceptible to liquefaction. Fine-grained soils, with cohesive strength, are less vulnerable to liquefaction specially for fines contents over 15%. This observation is apparently influenced by the system compliance for coarser soils. Alternatively, fine-grained materials such as cohesive soils get their strength primarily from intermolecular bonds rather than gravity forces; thus, liquefaction in the classical sense does not apply. Sensitive or highly structured clays can nevertheless undergo dramatic reductions in strength under cyclic loading.

Vaid et al. (1985) reported that substantial decrease in resistance to liquefaction has been shown to occur with increase in confining pressure for two sands with essentially identical gradation but differing in particle angularity. The decrease in resistance with confining pressure increases with increase in relative density and is larger for angular than for rounded sand. Angular sand could be susceptible to liquefaction even at relative densities approaching 100% under moderate earthquakes if the confining pressure is high. At low confining pressures, angular sand is considerably more resistant to liquefaction than rounded sand over the entire range of relative density.

Koester (1994) concluded that sand mixture containing fines up to about 24% of their dry weight may be inherently collapsible (due, possibly, to the relative compressibility of the finer soil between sand grains). When fines content exceeds that associated with lower-bound cyclic strength, the fines fraction dominates the

cyclic loading response if the soil. Plasticity index exerts much less effect on cyclic strength of soils containing fines at a given void ratio than does the fines content.

1.3.7 Previous Stress-Strain History

The importance of factors other than density on liquefaction characteristics of sand was first demonstrated by Finn et al. (1970) who showed by means of simple shear tests on small-scale samples of saturated sand that the liquefaction characteristics were influenced by the strain history to which they had previously been subjected. A typical example showing the stress ratios required to cause 100% pore pressure response for a freshly deposited sand and a similar deposit that had previously been subjected to a strain history representative of several very small earthquake shocks is shown in Fig. 3.

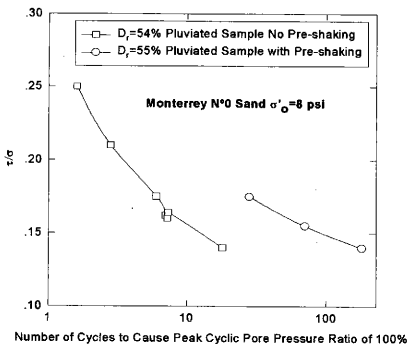


FIG. 3 Effect of Seismic History on Cyclic Load Characteristics of Sand (Seed, H.B. (1979). "Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes." *J. of the Geotech. Engrg. Div., ASCE*, 105(GT2), 201-255.)

An explanation of the possible causes of increased resistance to liquefaction resulting from seismic history effects is that during any period of cyclic straining there is a progressive change in the soil structures with the result that the volume change occurring in any cycle decreases progressively with increasing numbers of cycles (Seed et al. 1977).

CHAPTER II

METHODS FOR EVALUATION OF LIQUEFACTION

2.1 SIMPLIFIED PROCEDURES

2.1.1 Equivalent Uniform Cycle Procedure

A simple procedure by which a series of uniform cyclic stresses, assumed to be equivalent in their effect to the irregular stress sequence produced by an earthquake, could be determined with the method proposed by Seed and Idriss (1971). The method involves the computation of the equivalent uniform cyclic shear stress, τ_{avg} , induced at any point in a soil deposit using the relationship:

$$\tau_{avg} = 0.65 \gamma h a_{max} r_d \dots\dots\dots(1)$$

In which a_{max} = maximum horizontal ground surface acceleration; γ = total unit weight; h = depth below ground surface; and r_d = depth reduction factor. The 0.65 factor assumes that the equivalent uniform shear stress, τ_{avg} , is 65% of the absolute maximum shear stress. The depth reduction factor r_d , recognize that the soil is deformable and does not behave as a rigid body. A range of typical values for this factor has been suggested by Seed and Idriss (1971). Results of one dimensional ground response analyses can also be used to establish appropriate depth reduction factors.

The number of stress cycles over which the equivalent uniform shear stress is repeated may be evaluated either by using an appropriate weighting procedure or by adopting a representative number of cycles from studies of different magnitudes

earthquakes (Seed and Idriss 1971). In all cases, the number of cycles corresponding to an equivalent uniform shear stress, usually $0.65 \tau_{\max}$ (τ_{\max} is the maximum shear stress) has been plotted as a function of the earthquake magnitude. Relationships developed by Seed et al. (1985), are presented in Fig.4.

When number of cycles are computed from a series of earthquake motions, leading to results such as those shown in Fig. 4, it is necessary to assume that: (1) the motion is uniform at all sites and for all distances from the motion source; (2) the time history of stress at the depth of interest is directly proportional to the acceleration recorded at or near the ground surface; and (3) for all soils the laboratory liquefaction test data results can be represented by a single normalized curve relating stress ratio to the number of cycles causing liquefaction.

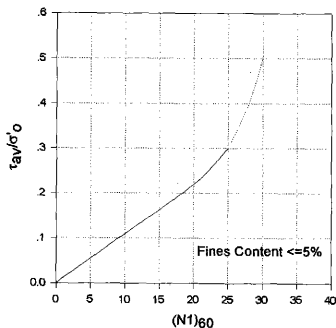


FIG.4. Correlation Between Stress Ratios and N_1 Values for Clean Sands and $M=7 \frac{1}{2}$ (Adapted from : Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M. (1985). "Influence of SPT procedures in soil liquefaction resistance evaluations." *J. Geotech. Engrg.*, ASCE, 111(12), 1245-1445.)

The data presented in Fig. 4 shows the extend of variation in the equivalent number of uniform cycles computed from recorded acceleration time histories using a consistent procedure. The extend of variation is most apparent when it is seen that the standard deviation of the results is approximately equal to a factor of two.

2.1.2 Liquefaction Resistance Factor (F_L)

An ability to resist the liquefaction of a soil element at an arbitrary depth may be expressed by the liquefaction resistance factor (F_L) identified by the following equation (Iwasaki et al. 1982):

$$F_L = \frac{R}{L} \dots\dots\dots(2)$$

When the factor F_L at a certain soil is less than 1.0 the soil liquefies during earthquakes. R in the equation (2) is the in-situ resistance or undrained cyclic strength of a soil element to dynamic loads during earthquakes, and can be simply evaluated according to numerous undrained cyclic shear test results using undisturbed specimens as follows:

For $0.04 \text{ mm} \leq D_{50} \leq 0.6 \text{ mm}$

$$R = 0.0882 \sqrt{\frac{N}{\sigma'_o + 0.7}} + 0.225 \log_{10} \frac{0.35}{D_{50}} \dots\dots\dots(3)$$

For $0.6 \text{ mm} \leq D_{50} \leq 1.5 \text{ mm}$

$$R = 0.0882 \sqrt{\frac{N}{\sigma'_o + 0.7}} - 0.05 \dots\dots\dots(4)$$

Where N is the number of blows of the standard penetration test, σ'_o is the effective overburdened pressure (kg/cm^2), and D_{50} is the mean particle size (mm). L

in the equation 5 is the dynamic load induced in the soil element by a seismic motion, and can be simply estimated by:

$$L = \frac{\tau_{\max}}{\sigma_o} = \frac{a_{\max}}{g} \frac{\sigma_o}{\sigma_o} r_d \dots\dots\dots(5)$$

Where τ_{\max} is the maximum shear stress (kg/cm^2), a_{\max} is the maximum acceleration at the ground surface, g is the acceleration of the gravity, σ_o is the total overburdened pressure (kg/cm^2), and r_d is the reduction factor of dynamic shear stress to account for the deformation of the ground. From a number of seismic response analysis for grounds, Iwasaki (1986) proposed the following relation for the factor r_d .

$$r_d = 1.0 - 0.015 Z \dots\dots\dots(6)$$

Where Z is the depth in meters. The value of a_{\max} can be estimated in view of the input bedrock motion and response characteristics of the ground layer. According to Iwasaki (1986) the value of a_{\max} at a site for an anticipated earthquake with magnitude M on the Richter scale and an epicentral distance (D) can be obtained by the following equation:

$$a_{\max} = 18.4 \cdot 10^{0.302M} \cdot D^{-0.8} \dots\dots\dots(7)$$

2.1.3 Liquefaction Potential Index (I_L)

An ability to resist liquefaction at a given depth of grounds can be evaluated by the factor I_L . However, it must be noticed that the damage to structures due to soil liquefaction is considerably affected by the severity of liquefaction degree. Iwasaki (1986) proposed the liquefaction potential index (I_L), defined by the equation 8 to estimate the severity of liquefaction degree at a given site (Arakawa et al. 1984).

$$I_L = \int_0^{20} F W(z) dz \dots\dots\dots(8)$$

Where $F=1-F_L$ for $F_L \leq 1.0$ and $F=0$ for $F_L > 1$, and $W(Z)=10-0.5Z$ (Z in meters). $W(Z)$ accounts for the degree of soil liquefaction according to the depth, and the triangular shape of $W(Z)$ and the depth of 20 m are decided considering the past earthquakes. For the case of $F_L=0$ for the entire depth, I_L become 100 being the highest, and for the case of $F \geq 1.0$ for the entire depth, I_L become 0 being the lowest.

TABLE 1. Soil Liquefaction Based on the Index I_L

$I_L=0$	liquefaction risk is very low
$0 < I_L \leq 5$	liquefaction risk is low
$5 < I_L \leq 15$	liquefaction risk is high
$I_L > 15$	liquefaction risk is very high

From the analysis of 64 liquefied points and 23 non-liquefied points, considering the action of six great earthquakes in Japan, Iwasaki (1986) proposed a relationship between the liquefaction index (I_L) and the potentials of soil liquefaction summarize in Table 1.

2.1.4 Steady State Approach

Poulos et al. (1985) presented a liquefaction evaluation procedure in which for the liquefaction analysis the undrained steady-state shear strength is required, and the procedure involves the following steps: (1) Determine in-situ void ratio obtained from a suitable undisturbed sample of loose sand at depth in situ by fixed-piston sampling, freezing of the ground and coring or sampling in test pits; (2) Determine the steady-state void ratio as a function of effective stress using compacted specimens (S_{su}) computed from the results of each consolidated-undrained triaxial test:

$$\begin{aligned}
 S_{su} &= q_s \cos \phi_s \\
 \sin \phi_s &= \frac{q_s}{\sigma_{3s} + q_s} = \frac{q_s}{(\sigma_{3s} - \Delta\mu_s) + q_s} \dots\dots\dots(9) \\
 q_s &= \frac{\sigma_{1s} - \sigma_{3s}}{2}
 \end{aligned}$$

In which $\sigma_{1s} - \sigma_{3s}$ is the principal stress difference at the steady state from the triaxial test, $\bar{\sigma}_{3s}$ is the effective minor principal stress at the steady state; $\bar{\sigma}_{3s}$ is the effective minor principal stress at start of shear (after consolidation); $\Delta\mu_s$ is the pore pressure induced in the specimen at the steady state of deformation, and ϕ_s is the steady-state friction angle (in terms of effective stress).

(3) Determine the undrained steady-state strengths for undisturbed specimens, a series of consolidated-undrained triaxial tests is performed on “undisturbed” specimens from the zone being evaluate. Sufficient tests are needed to determine the average steady-state strength reliably. (4) Correct measured undrained steady-state strength to in-situ void ratios from the measurements made during undisturbing sampling, the in-situ void ratio for each of the tested “undisturbed” specimens can be computed. (5) Calculate the in-situ driving shear stress and the factor of safety. The in-situ driving shear stress (τ_d) in the zone being evaluated is calculated by conventional methods of stability analysis. It is the shear stress required to maintain static equilibrium. The factor of safety against liquefaction, F_L is:

$$F_L = \frac{\text{undrained steady - state shear strength}}{\text{shear stress required to maintain static equilibrium}} = \frac{S_{su}}{\tau_d} \quad (10)$$

2.1.5 Residual Pore Water Pressure

To apply this procedure three classes of information must be made known or assumed. *In-situ soil properties*, it is most preferable to conduct cyclic triaxial shear tests on undisturbed samples of soils. If it is impossible to conduct tests on

undisturbed samples, the cyclic strength may be evaluated based on the results of some indirect field tests. *Field conditions*, the resistance of in-situ deposits to pore water pressure buildup depends on the depth of the ground water table and the in-situ coefficient of earth pressure at rest (Fig. 5).

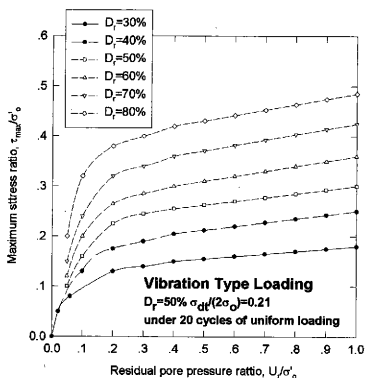


FIG. 5 .Relationship Between Maximum Shear Stress Ratio and Residual Pore Pressure (Adapted from: Ishihara, K. (1977). "Simple method of analysis for liquefaction of sand deposits during earthquakes." *Soils and Foundations*, 17(3), 1-18.)

Time history of acceleration, must be given at the ground surface for computing the shear stresses induced in the ground during a given earthquake, for this method is only necessary to specify the maximum acceleration and the type of waves

(shock or vibration type). The steps to calculate the factor of safety against liquefaction are the following (Ishihara, 1977).

(1) The cyclic stress ratio $\sigma_{diff}/(2\sigma'_o)$ causing liquefaction under 20 cycles of uniform loading must be converted to the stress ratio in terms of the maximum shear stress, $\tau_{max,i}/\sigma'_o$. The conversion can be done simply by dividing the cyclic stress ratio by the reduction factor, R_f , which is either 0.55 or 0.70 depending upon whether the given wave is of shock type or vibration type, respectively.

From Fig. 5 the value of μ_r/σ'_o is determined from the following relationship τ_{max}/σ'_o is calculated with the equation :

$$\frac{\tau_{max}}{\sigma'_o} = \frac{3}{1+2K_o} \frac{\tau_{max,i}}{\sigma'_o} \dots\dots\dots(11)$$

(2) Estimate the magnitude of maximum stress ratio that may be applied to the soil element in the deposit when it is subjected to a shaking due to the earthquake. Based on the information concerning the depth of the ground water table and the unit weight of the soils, the maximum stress ratio τ_{max}/σ'_o can be computed with the following equation :

$$\frac{\tau_{max}}{\sigma'_v} = \frac{a_{max}}{g} r_d h \left(\frac{Z}{H} \right) \dots\dots\dots(12)$$

(3) By locating the computed maximum stress ratio on Fig. 5, it is possible to determine the residual pore pressure ratios for each depth of the deposit, then the factor of safety against liquefaction, F_1 , may be defined as:

$$F_1 = \frac{\tau_{max,i}/\sigma'_v}{\tau_{max}/\sigma'_v} \dots\dots\dots(13)$$

2.2 METHODS BASED ON IN-SITU TESTS

Table 2 shows the advantages and disadvantages of the five in-situ techniques (SPT, CPT, PMT, DMT and shear wave velocity) that have been used to assess liquefaction potential.

TABLE 2. In Situ Test to Assess Liquefaction Potential

Field Testing Technique	Advantages	Disadvantages
Standard Penetration Test (SPT)	popular testing tool, large data base	equipment variable, blow count average/12"
Cone Penetration Test (CPT)	continuous reading, economical, fast, standardized test	no sample, limited data base
Pressuremeter (PMT)	test soils in "undisturbed state"	present approach required lab testing
Dilatometer (DMT)		use in partially drained (silts) conditions is not recommended
Shear Wave Velocity	no drilling necessary, test difficult sites (gravels, etc)	no sample, limited use, limited reliability

2.2.1 Standard Penetration Test (SPT)

Two methods of evaluation of liquefaction resistance using the SPT find widespread usage. In the United States, Seed et al. (1985) developed a method that separates liquefaction and nonliquefaction from observed field performance data on a plot of cyclic stress ratio and SPT N_1 -value normalized to a stress level of 1 ton/ft². The method also distinguishes between various fines contents in sands (see Fig.6).

The second method to establish a correlation between the cyclic strength and N value is to collect a large number of laboratory test data in the cyclic strength of undisturbed soil samples recovered from deposits of known penetration resistance.

An empirical correlation between these two quantities can easily be established: one of the relations incorporated in the Japanese code of bridge design (Tatsuoka et al., 1980) who developed a correlation between cyclic strength for 20 cycles of equivalent loading (R_{20}), SPT N-value normalized for overburden pressure effects, and particle size expressed in terms of mean particle diameter (D_{50}).

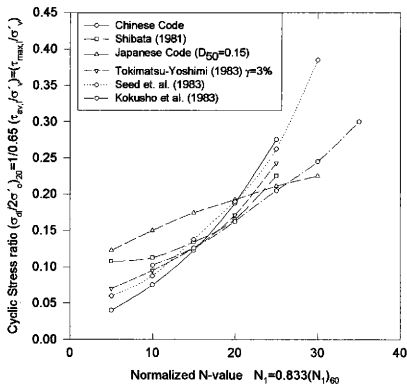


FIG.6. Comparison of Curves Proposed by Different Researchers (Adapted from: Ishihara, K. (1993). "Liquefaction and flow failure during earthquakes." *Géotechnique*, 43(3), 351-415)

Similar attempts were made by Kokusho and Yoshida (1985) on the basis of a vast body of laboratory test data on clean sands. Relations based on a large body of field performance data obtained mainly in Japan were proposed by Tokimatsu and

Yoshimi (1983). On the basis of recent earthquakes in China, the criterion for identifying sandy deposits as being susceptible or immune to liquefaction was presented in the form of a code requirement (Ishihara 1993):

$$\left[\frac{\sigma_{dl}}{2\sigma'_o} \right]_{20} = \frac{\tau_{\max,1}}{\sigma'_v} = \frac{1}{1000} (9.5N_1 + 0.466N_1^2) \dots\dots\dots(14)$$

All the explained relations are shown in Fig. 6. From the cluster of curves proposed by various researchers, it is apparent that the relations fall in approximately the same range for $N_1=10-25$, where actual data were available in abundance. It has been apparent that the standard penetration test has not been standardized. There are important differences between the procedures used in different countries and there can be significant differences in the practice followed within a country. There are several aspects of the problem to consider: the manner in which energy is delivered to the drill rod, the length of the drill rod, the effect of the type of sampling tube, the effective stress present at the depth where the blow count is being evaluated, the diameter of the drill hole, the type of bit used in the drilling operation, the frequency of delivery of the hammer blows, and the nature of the drilling fluid (Schmertmann 1979, Kovacs and Salomone 1979, Seed and De Alba 1986).

2.2.2 Cone Penetration Test (CPT)

Based on a compilation of a large body of field performance data Robertson and Campanella (1985) proposed correlations for clean sands and silty sands as shown in Fig. 7, where the cone tip is expressed in the form of q_{c1} , a value normalized to an effective overburden pressure of $\sigma'_o = 1 \text{ kg/cm}^2$. Similar correlations were established

by Seed and De Alba (1986), Shibata and Teparaska (1988), and Jamiolkowski et al. (1985), in which the effects of fines content are allowed for in terms of the median grain size. The correlations for the case of apparently clean sands with $D_{50} \geq 0.25$ mm and for silty sands with $D_{50} \leq 0.15$ mm proposed in these works are shown in Fig. 7.

In most of the correlations mentioned above, effects of the presence of fines are allowed for in such a way that the penetration resistance becomes smaller with increasing fines content if soils possess equal cyclic strength. At constant penetration resistance, soils are observed to have increasing cyclic strength with increasing fines content.

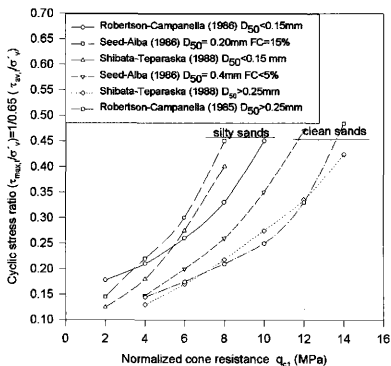


FIG.7. Summary Chart for Evaluation of the Cyclic Strength of Sands Based in Normalized CPT q_{c1} Value (Adapted from: Ishihara, K.(1993). "Liquefaction and flow failure during earthquakes." *Geotechnique*, 43(3), 351-415)

Liquefaction studies in China have led to a correlation between earthquake shaking conditions causing liquefaction or cyclic mobility and the cone penetration of sands. In this correlation the critical value of cone penetration resistance, q_{crit} , separating liquefiable from non-liquefiable conditions to a depth of 15m is determined by:

$$q_{cr} = q_{co} [1 - 0.065(H_w - 2)][1 - 0.05(H_o - 2)] \dots \dots \dots (15)$$

Where H_w is the depth to groundwater table (m); H_o is the thickness of cohesive overburden (m); q_{co} is the reference critical CPT value in MPa for liquefiable conditions when $H_o=2$ m and $H_w=2$ m is a function of the earthquake intensity of the site. Farrar (1990) present a compilation of the different available procedures for the assessment of liquefaction potential one of them is suggested by Olsen (1984) consist in a correlation between SPT and CPT through the use of static stress level normalized tip and friction sleeve resistances compared with a normalized blow count. The boundary curves proposed by Shibata and Teparaksa (1988) are of the hyperbolic type and expressed by equation 25, in which $C_2=D_{50}/0.25$.

$$(q_{cl})_{cr} = C_2 \left[50 + \left[\frac{\left(\frac{\tau}{\sigma'_o} \right) - 0.1}{\left(\frac{\tau}{\sigma'_o} \right) + 0.1} \right] \right] \dots \dots \dots (16)$$

2.2.3 Pressuremeter Test (PMT)

Vaid et al. (1981) proposed a method for the assessment of liquefaction potential based in a correlation between cyclic stress ratio and dilation angle, ν . The relationship is shown in Fig. 8. The dilation angle is derived form self-boring pressuremeter data using the theory by Hughes et al. (1977).

The correlation from a relationship between relative density and dilation angle for Ottawa sand. The dilation angle was considered to be a useful parameter to represent the in-situ state of a sand and was computed using a tangent at a shear strain of 10%. The dilation angle was obtained from a simple shear test corrected to a normal pressure of 1 Tn/ft².

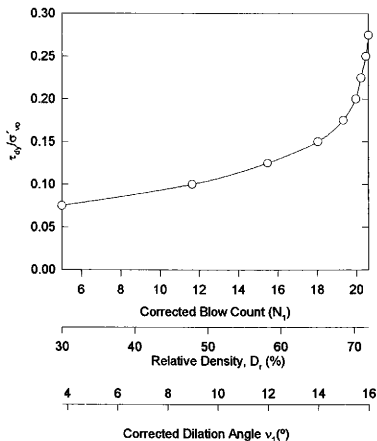


FIG.8. Resistance to Liquefaction of Sand as Function of Relative Density, Dilation Angle or Penetration Resistance (Adapted from: Vaid, Y.P., Byrne, P.M., and Hughes, J.M.O. (1981). "Dilation angle and liquefaction potential." *J. of the Geotech. Engrg. Div.*, ASCE, 107(GT7), 1003-1008.)

The dilation rate of a soil is a direct measure of the volume change characteristics, which have been considered a primary factor for liquefaction potential.

The main advantage of the pressuremeter method is that it uses a parameter (ν) that can be measured in the field and in the laboratory. This enables direct comparison of field and laboratory data.

An alternative pressuremeter liquefaction resistance correlation was obtained by Robertson (1982). The correlation is shown in Fig.9 could be used as an independent check as to the liquefaction resistance of a sand deposit using a self-boring pressuremeter. The liquefaction resistance could be determined from both the corrected dilation angle and the cumulative strain.

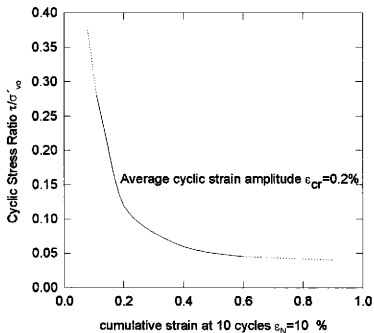


FIG.9. Proposed Correlation Between Cyclic Stress Ratio and Cumulative Strain at 10 Cycles (Adapted from: Robertson, P.K. (1982). "In-situ testing of soils with emphasis on its application to liquefaction assessment." Ph.D.Dissertation,University of British Columbia, Vancouver, British Columbia, Canada.)

2.2.4 Dilatometer Test (DMT)

Marchetti (1980) suggested that the horizontal stress index K_d could be used as a parameter to assess the liquefaction resistance under level ground conditions of sands under cyclic loading. K_d appears to reflect the following soil variables (Robertson, 1982; Robertson and Campanella, 1986): Relative density, D_r ; In-situ stresses, K_o ; stress history and pre-stressing; aging; and cementation. However, it is not possible to identify the individual responsibility of each variable. Marchetti (1980) suggested the following tentative correlation between the cyclic stress ratio to cause liquefaction (τ_f/σ'_o) and the horizontal stress index K_d .

$$\frac{\tau_f}{\sigma'_v} = \frac{K_d}{10} \dots\dots\dots(17)$$

Marchetti has shown that K_d appears to increase with increases in K_o , aging, cementation, and stress history. Robertson and Campanella (1986) developed a relationship shown in based on a K_d - D_r relationship for normally consolidated, uncemented sands, any increase in the mentioned factors will produce an increase in apparent density and thus be reflected by an increase in liquefaction resistance.

The correlation shown in Fig. 10 is only applicable for testing in sands where penetration and expansion occurs under drained conditions. Testing in silty sands or silts may generate significant pore pressures, which would influence the measured K_d values.

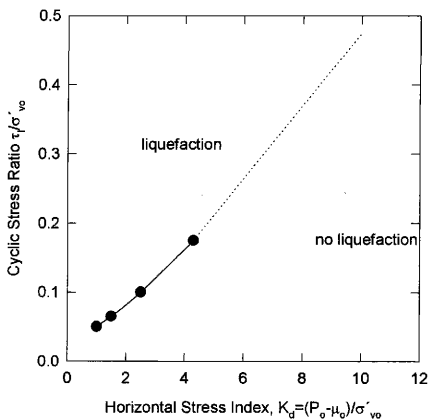


FIG.10. Proposed Correlation Between Liquefaction Resistance Under Level Ground Conditions and Dilatometer Horizontal Stress Index for Sands (Adapted from: Robertson, P.K., and Campanella, R.G. (1986). "Estimating liquefaction potential of sands using the flat plate dilatometer." *Geotech. Testing Journal*, ASTM, 1(9), 38-40.)

2.2.5 Electrical Resistivity

Arulmoli et al. (1985), Arulanandan et al. (1986) and Arulanandan et al. (1988) have developed techniques for measuring the resistivity and capacitance of soil in-situ, showing that these characteristics can be correlated to liquefaction resistance as measured by cyclic load tests in the laboratory.

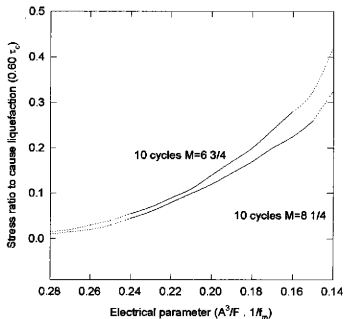


FIG.11. Correlation Between Field Liquefaction Behavior of Sands for Level Ground Conditions and Electrical Parameter Defining Void ratio, Anisotropy and Shape (Adapted from: Arulmoni, K., Arulanandan, K., and Seed, H.B. (1985). "A new method for evaluating liquefaction potential." *J. Geotech. Engrg.*, ASCE, 111(1), 95-114.)

The cyclic stress ratio required to cause liquefaction was correlated to an electrical parameter ($A^3/\bar{F}\bar{f}_{max}$) using cyclic laboratory tests. The electrical parameter combines three electrical parameters, defined as follows: A is the anisotropy index $=(F_v/F_H)^{1/2}$ where F_v is the vertical formation factor and F_H is the horizontal formation factor; \bar{F} is the average formation factor $=(F_v+2F_H)/3$; and \bar{f}_{max} is the average shape factor.

The validity of the correlation was checked using in-situ measurements from a limited number of sites where liquefaction had or had not occurred. Arulmoli et al.

(1985) used an electrical probe to predict relative density, cyclic stress ratio and K_{2max} from in-situ electrical measurements. These values were compared with values measured independently from controlled laboratory tests. Reasonable agreement was found between predicted and measured values.

The correlation shown in Fig. 11 appears to provide reasonable predictions of whether liquefaction would occur or not at three major earthquake sites, although the data points were a significant distance from the boundary separating liquefiable from non-liquefiable sites.

2.3 PROBABILISTIC AND STATISTICAL ANALYSIS

Deterministic models of soil liquefaction give a yes or no answer as to whether liquefaction will occur or not, or an answer in the form of a factor of safety. Probabilistic and statistical methods can be introduced at various stages of a liquefaction risk assessment, the following items had been identified as sources of uncertainty: (1) uncertainty in the magnitude and location of earthquakes that can potentially affect the site, (2) Uncertainty of the acceleration and duration of ground motion at a site, resulting from an earthquake but attenuated by distance and filtered by the site response, (3) Uncertainty in the basic physical models of soil liquefaction behavior (model uncertainty), and, (4) Uncertainty in the soil resistance parameters input to physical model (the site characterization problem).

2.3.1 Probabilistic Models

The probability that liquefaction occurred at a specific site within a time period T can be generally expressed as:

$$P(Y=1) = \int_{\Omega} \{1 - \exp(-\lambda T) \int_{\Psi} P(Y=1/\Omega, \Psi) g(\Psi) d\Psi\} g(\Omega) d(\Omega) \dots (18)$$

For the usual case of small values resulting from the integral over Ψ (which is the probability that any earthquake will result in liquefaction), P is well approximated by:

$$P(Y=1) \cong \lambda T \int_{\Omega} \int_{\Psi} P(Y=1/\Omega, \Psi) \cdot f(\Omega) \cdot g(\Psi) d\Psi d(\Omega) \dots \dots \dots (19)$$

Where Ψ is a vector of earthquake load parameters, Ω is a vector of liquefaction resistance parameters, Y is an indicator of liquefaction ($Y=1$ if liquefaction occurs, and $Y=0$ if it does not occur), $P(Y=1/\Omega, \Psi)$ is the conditional probability of liquefaction, $g(\Psi)$ is the probability distribution of the earthquake load parameters, and represents the uncertainty of specific magnitudes and locations of earthquake occurrence, $f(\Omega)$ is the probability distribution of the soil resistance parameter and represent the fact that site conditions are often spatially variable and inhomogeneous, and λ is the overall rate of earthquake occurrence from all potential seismic sources within the vicinity of the project site.

Some liquefaction risk procedures, such as this presented by Youd and Perkins (1978), does not account for any uncertainty of this factor. That is, they assume that the conditional probability function $P(Y=1/\Omega, \Psi)$ can only take on values of 1 or 0 (liquefaction or no liquefaction), which represents a purely deterministic formulation. All the uncertainty in their liquefaction risk assessments arises from the uncertain nature of seismicity. Actually, there is some uncertainty in any method used to determine the likelihood of liquefaction given an earthquake. Approaches which have been used to obtain the conditional probability of liquefaction in other than yes or no terms can be categorized as probabilistic or statistical

Haldar and Tang (1979) use a first order second-moment (FOSM) method applied to the Seed and Idriss (1971) simplified method to obtain the conditional probability of liquefaction. Basically, this involved estimating the uncertainties

(FOSM parameters) of the components of the Seed-Idriss model and propagating these uncertainties through the model.

A more sophisticated FOSM model has been presented by Fardis and Veneziano (1982) incorporating the effects of pore pressure diffusion, soil stiffness reduction, and variations of soil properties within a stratum. In both models the assumption of normality log-normality if load and resistance parameters is used in estimating the conditional probability of liquefaction.

Probabilistic analysis based on pore pressure generation models have been presented by Chameau and Clough (1983). The accumulation of pore pressure is calculated using a nonlinear formulation, based either in laboratory data or on a basis constitutive physical model. Their result for a conditional probability $P(Y=1/\Omega, \Psi)$ is equated to the probability that the pore pressure ratio r_u is equal to 1, and calculated assuming random arrivals of shear stress (or equivalent acceleration) peaks among positive zero crossing of the earthquake record. This distribution of the shear stress peaks has been modeled as beta, gamma, Rayleigh, or exponential distributed. The number of positive crossing is a measure of duration and may be also treated as a random variable, as is done in the application of Chameau and Clough's method by Kavanzanjian et al. (1985).

2.3.2 Statistical Analysis

One aspect of uncertainty in liquefaction analysis is the problem of the determination of the best boundary separating liquefaction and nonliquefaction behavior, this is known in statistics as a problem of classification or discrimination.

This problem has been treated by Christian and Swieger (1975) using empirical data on site liquefaction behavior and a statistical method known as linear discriminant

analysis. Their results have been interpreted to mean that: $P(Y=1/Dr,A)=1$ if $Dr < KA^{0.31069}$ or 0 if $Dr > KA^{0.31069}$, where Dr is the relative density of the soil and A is a modified site acceleration. The results presented by Christian and Swieger (1975) are just an example of a variety of results that can be obtained using discriminant analysis. Generally associated with the linear discriminant analysis methodologies are underlying assumptions of normality and randomness of data that are not satisfied by the available collection of liquefaction data.

Yegian and Whitman (1978) presented a different classification method, termed the "least squares of the misclassified points". In essence, this method finds the boundary that best separates liquefaction from nonliquefaction based on minimizing the sum of the squared distances between the misclassified points and the boundary line. For a given earthquake, the probability of liquefaction can be expressed very simply as:

$$P_E[F_L] = P[F_L|E]P[E] \dots\dots\dots(20)$$

In which $P_E[F_L]$ is the overall probability of liquefaction of some site during earthquake E ; $P[F_L|E]$ is the probability of liquefaction at the site given that the earthquake E occurs, and $P[E]$ is the probability that the earthquake occurs. The total overall probability of liquefaction at the site is obtained *summing* over all possible earthquakes:

$$P[F_L] = \sum_E P[F_L|E]P[E] \dots\dots\dots(21)$$

The problem with classification methods is that the discrimination criteria still give a deterministic yes-or-no type answer to whether liquefaction will or will not

occur at a site, rather than a continuous conditional probability $P(Y=1/\Omega, \Psi)$ that varies smoothly among 0 and 1.

Veneziano and Liao (1984) have presented a class of statistical methods that avoid many problems in estimating $P(Y=1/\Omega, \Psi)$. Instead of treating the problem as a classification problem using Y as a binary (0 or 1) response variable. An advantage of this approach is that the probability distributions of Ω and Ψ do not need to be estimated and that $P(Y=1/\Omega, \Psi)$ is directly evaluated with no need to appeal to Baye's Theorem.

Liao (1986) and Liao et al. (1988) presented statistical models developed to calculate the conditional probability of liquefaction as a function of earthquake load and soil resistance parameters and the binary logistic regression is the principal method used to derive the statistical models. They obtained two types of models, one that uses the cyclic stress ratio as the earthquake load parameter and the other one uses σ'_v as the load parameter, and explicit function of magnitude and distance, both types of models use the corrected/normalized SPT $(N_1)_{60}$ values as the indicator of liquefaction resistance.

Reyna (1991) developed relationships to evaluate liquefaction potential using in-situ tests, in particular the Dilatometer and the cone penetrometer using the discriminant analysis to confirm the validity of the proposed boundary curves proposed for the CPT.

CHAPTER III

DATA ANALYSIS

3.1 PROBLEMS ASSOCIATED WITH DATA COLLECTION

Non-Uniform Quality of the Data, some of the data reported in catalogs are from earthquakes that occurred more than hundred years ago. This data clearly is less reliable than information from recent events. Another cause of potentially poor quality data is the lack of earthquake recording networks in some geographic regions.

Lack of Statistical Independence Between Case Studies, Tokimatsu and Yoshimi and Tokimatsu (1983) and Shibata and Teparaksa (1988) catalogs show two or more cases studies using data at different depths from the same boring. Also, many cases are obtained from several borings at the same site, the physical proximity of the data raises the question of independence between the cases. It is also common practice to use the same boring data as a series of successive case studies in different earthquakes (Seed et al. 1983).

Non-Proportional Sampling of Liquefaction versus Non-Liquefaction Sites, in general, liquefaction sites tend to be studied in more detail and are more extensively reported than non-liquefaction sites. Hence, the proportion of liquefied to non-liquefied sites tends to be higher in the catalogs than in reality. This source of bias, from reported relative frequency, affects the estimation of liquefaction probability.

Measurement Errors, errors of this type are present, for example, in the estimation of earthquake magnitude, epicentral distance, the SPT N-value or CPT q_c value.

Difficulties of Site Characterization, in most of the liquefaction catalogs, sites are characterized by only a single resistance value (CPT or SPT). It is not clear and consistent how the selected value is representative of a boring profile or even an entire site.

Lack of Differentiation Between SPT Data Obtained Before and After Earthquake Occurrence, although changes in density and resistance are reported to occur as a result of earthquake shaking, these changes are often ignored. The implicit assumption usually made is that post-earthquake values of resistance adequately represent the site before the earthquake occurred.

Lack of Differentiation Between N-values Obtained Using Various SPT Methods, with the exception of the Tokimatsu and Yoshimi (1983) and Seed et al. (1985), the various source catalogs do not differentiate between the “standard” rope and pulley method and the “free fall” methods of performing the SPT. Tokimatsu and Yoshimi (1983) also indicate that a difference exist between the results of the SPT using Japanese and non-Japanese drilling methods. Seed et al. (1985) had provided corrections to account for these factors.

3.2 ANALYSIS OF DATA CHARACTERISTICS

3.2.1 Earthquake Magnitude and Distance Measures

Earthquake magnitude measurements can vary by several tenths of a unit, depending on the particular location of the earthquake recording station and the type of recording instrument used. As originally defined, the Richter magnitude M , which seismologist currently refer as a local magnitude M_L , was measured using the maximum amplitude wave (P, S, or surface wave) recorded by a standard instrument (Wood-Anderson torsion seismograph) which has a specified natural period (0.8 sec), magnification (2800) and damping factor (0.8), corrected to represent a measurement at a standard distance (100 km) from the earthquake epicenter.

Two additional magnitude scales have been used in the documentation of liquefaction case studies in Japan; the Kawasumi and the Japan Meteorological Agency (JMA) magnitude scales. Estimates of earthquake sizes based on the JMA scale are comparable to those given on the Richter scale, but the Kawasumi magnitude (which is based on an intensity scale) appears to assign slightly larger ratings of earthquake size. The Kawasumi magnitude is usually reported only for historical (pre-instrumental) earthquake in Japan.

The location of earthquake hypocenter or epicenters and focal depths are determined from arrival time of seismic waves at various recording stations, and are also subjected to large variations. Another distance measure incorporated in the synthesized catalog (Appendix A) is the “distance to energy release” or DER. Where possible DER is defined as the closest distance to the surface fault rupture. In cases where the surface manifestation of the fault rupture is evident, it is defined as the closest distance to the surface projection of the “zone of energy release”, which is sometimes determined from the spatial distribution of earthquake aftershocks. However, if neither of the above measures are available, DER was assumed to be equal to the epicentral distance.

3.2.2 Acceleration and Cyclic Stress Ratio

An important quantity considered in liquefaction analysis is the cyclic stress ratio (CSR) defined originally by Seed and Idriss (1971) as:

$$CSR \approx 0.65 \frac{a}{g} \frac{\sigma_v}{\sigma'_v} r_d \dots\dots\dots (22)$$

In a more recent paper, Seed et al. (1985) have implicitly defined a magnitude normalized CSR_N where r_M is the magnitude normalization factor.

$$CSR_N \approx 0.65 \frac{a}{g} \frac{\sigma_v}{\sigma'_v} \frac{r_d}{r_M} \dots\dots\dots (23)$$

The intent of this normalization is to account for the effects of duration of shaking which is correlated to the earthquake magnitude M . In accordance with Seed

et al. (1985), r_M is defined so that CSRN correspond for a $M=7.5$ earthquake. Use of average values of r_d (as a function of depth) and r_M (as a function of magnitude) is recommended in practice, and they are given in chart or tabular form. The following formulas were fitted to the recommended average functions:

$$\begin{aligned} r_d &= 1.0 - 0.00765z & z \leq 9.15\text{m} \\ r_d &= 1.174 - 0.0276z & z > 9.15\text{m} \end{aligned} \quad (24)$$

$$r_M = 0.032M^2 - 0.631M + 3.934 \quad (25)$$

Where z is the depth in meters, and M is the earthquake magnitude. The primary variable affecting the value of CSR or CSRN is the peak ground acceleration “ a ”, which can be obtained in several ways. In 127 of 278 catalogs entries in Liao 1986 (see Appendix), the peak acceleration is obtained from measurement at a “nearby” station. In a few cases, a strong motion recorder is actually close enough to be considered “on site”. Other methods of estimating acceleration include performing a site response analysis with the input from a ground motion record some distance away, scaled to reflect inferred bedrock motions at the site of interest. In many cases, accelerations are calculated from earthquake attenuation relationships and/or correlations to an intensity damage scale.

In the catalog presented by Liao 1986 (Appendix), in the case of historical cases of liquefaction/non-liquefaction from California and Japan where the acceleration was not reported, or where the reported acceleration was suspect, accelerations were estimated from one of two attenuation relationships. For cases in California, The Joyner and Boore equation was used:

$$\frac{a}{g} = \frac{(0.0955)10^{0.249M}}{r^{0.00225r}} \quad (26)$$

Where M is the moment magnitude and $r=(d^2+7.3^2)^{1/2}$, in which d is defined as the closest distance (km) to surface projection of the fault rupture. For Japanese earthquakes, the relationship used is du to Kawashima for soft alluvium or reclaimed ground:

$$\frac{a}{g} = \frac{0.4109 \times 10^{0.262M}}{(R_{EP} + 30)^{1.208}} \dots\dots\dots(27)$$

Where M is the Japanese Meteorological Association (JMA) magnitude and REP is the epicentral distance in kilometers.

3.2.3 Correction/Normalization Factors of SPT

There are two corrections or normalization that need to be made to the N-value obtained directly from the standard penetration test (SPT). The first is to take into account the effect of the overburden pressure, and the second is to account for the effects of using different sampling equipment and/or practices in performing SPT. The SPT resistance corrected for overburden is denoted as N_1 and is calculated as (Seed et al. 1985):

$$N_1 = C_N N \dots\dots\dots(28)$$

Where C_N is the overburden correction/normalization factor. The additional normalization factor to account for sampling equipment and practices is denoted as C_E and the additional correction made is calculated as:

$$(N_1)_{\omega} = C_E N_1 = C_E C_N N \dots\dots\dots(29)$$

Several correction factors for overburden have been published in the literature and the more generalized expression is (Decourt 1989):

$$C_N = \left[\frac{(\sigma'_{v0})_1}{(\sigma'_{v0})} \right]^n \dots\dots\dots(30)$$

Jamiolkowski (1985) based on experimental results in normally consolidated sands proposed $n=0.56$, Decourt (1989) suggest a factor of 0.5. The expression coincide with the mathematical form of C_N proposed by Liao and Whitman (1986):

$$C_N = \sqrt{\frac{1}{\sigma_v}} \dots\dots\dots(31)$$

The values of the correction factor C_E are based on the recommendations of Seed et al. (1985), and are calculated as:

$$C_E = \frac{ER}{60} C_{JAP} C_{ROD} C_{LIN} \dots \dots \dots (32)$$

In the above equation, ER is the energy ratio defined as the percent of the theoretical free-fall energy transmitted to the rods from the SPT hammer. ER has been found to vary depending on the method of releasing the hammer and the type of anvil (Skempton 1986). The denominator of 60 reflects the recommendation by Seed et al. (1985) that the SPT data be normalized to an equivalent ER=60%. C_{JAP} is a correction for the different standards of drilling practice in Japan.

C_{ROD} corrects for the effect of short lengths when performing the SPT at shallow depth (10m), the weight or stiffness of the rod stem, of a given length, appears to have little effect (Skempton 1986).

C_{LIN} is a correction to account for the practicing of leaving out the inside liners from the barrels of the SPT sampler, giving an internal diameter of 1 1/2 in instead of the standard 1 3/8.

The importance of these corrections is to attempt to make the different types of SPT measurements comparable. Only for effects to compare the values in the Appendix the values of N and $(N_1)_{60}$ are shown.

3.2.4 Correction/Normalization Factors of CPT

With the exception of Robertson (1982) direct approach, all the remaining empirical liquefaction assessment methods use a type of stress level normalization equation. Practice within the United States has been to normalize the measurements to 1 TSF (96 kPa) stress level with an equation in the form:

$$q_{c1} = c_n q_c \dots \dots \dots (33)$$

Where q_{c1} is the normalized cone end bearing stress, q_c is the measured cone end bearing stress, $c_n = 1/(\sigma'_o)^n$, σ'_o is the initial vertical effective stress, and n is a normalization factor experimentally determined. Robertson and Campanella (1985)

presented a graph for obtaining c_n . The results represented on this figure were based on large calibration chamber tests performed on Ticino sand. Although no value for n was given, the response curve may be closely approximated by letting $n=0.60$. For sands Olsen (1984) recommended a value of $n=0.70$, based upon a review of several large calibration chamber tests. Jamiolkowski et al. (1985) summarized several large calibration chamber tests. The expression developed is:

$$q_c = C_0^{(DrC_1)}(\sigma'_0)^{C_2} \dots\dots\dots(34)$$

Where Dr is the relative density and C_0 , C_1 , and C_2 are experimentally derived constant. From equation 45, the constant C_2 is equivalent to the value of n shown in equation 44. The average value for all the specimens considered is $C_2=0.72$ varying approximately from 0.60 to 0.85 (Carter 1988).

3.2.3 Site Characterization and Data Independence

In the context of liquefaction analysis, site characterization refers to the problems of determining a representative SPT or CPT resistance and depth at which liquefaction is likely to occur. In case studies where liquefaction has occurred, it may be possible to identify the depth of liquefaction from comparison of the soil ejected from sand boils with samples obtained at depth in borings. However, such data are not always available, and in the case of non-liquefaction, an estimation of the critical depth where liquefaction would most likely occur (given a stronger earthquake) can require a considerable degree of judgment.

In the present work was considered the minimum N_1 value as the characteristic of a boring profile, criteria that is consistent with the concept of liquefaction occurring at a critical depth, as first developed by Seed and Idriss (1971). This is because the critical depth of liquefaction, is virtually controlled by the variation of N_1 with depth. For CPT values the lowest in the soil profile was considered such as representative.

3.3 DATA CATALOGS

The data was compiled from different catalogs shown in Appendix. It is important notice that the data provided in each catalog vary even for the same case due to the author interpretation. Table 3 summarizes the type of information obtained from each catalog and the number of liquefaction and non-liquefaction cases.

TABLE 3. Summary of Liquefaction Source Catalogs

Source Catalog	Number of Cases		Earthquake Load Parameters	Site Soil Parameters	Comments
	liq.	non-liq.			
Tokimatsu and Yoshimi 1983	51	46	a, M, CSR	$N, N_1, D_r, z_w, z_{Lr}, \sigma_o, FC, CC, GC, D_{50}, UC$	Liquefaction occurrence classified in 4 states: extensive, moderate, marginal and no liquefaction
Shibata and Teparaksa 1988	84	41	a, M, CSR	$q_s, q_1, D_r, z_w, z_{Lr}, \sigma_o, D_{50}$	Analysis the liquefaction in the same boring under different depths
Liao 1986	114	164	a, D, M, R, CSR, CSRN	$N, N_1, (N_1)_{60}, D_r, \sigma_o, z_w, z_{Lr}, FC, CC, GC, D_{50}, UC$	Extensive compilation, show minimum N instead of average penetration resistance.
Reyna 1991	37	21	a, M, CSR	$q_s, q_1, D_r, z_w, z_{Lr}, \sigma_o, D_{50}$	Compilation of recent events, only CPT test.
Tokimatsu et al. 1994	11	4	a, M, CSR,	$N, N_1, D_r, z_w, z_{Lr}, \sigma_o$	Few details of the earthquake characteristics.

3.4 STATISTICAL PROCEDURES

3.4.1 Discriminant Analysis

Discriminant analysis is a procedure from the field of multivariate statistics that is used to separate items into categories (or groups) based on a number of observations. With this procedure, a discriminant function is obtained to classify each observation into one of the categories. It is assumed that the distribution within groups should be approximately multivariate normal. The discriminant function, also

known as a classification criterion, is determined by a measure of generalized squared distance. The classification criterion can be based either the individual within-categories covariance matrices or the pooled covariance matrix (SAS/STAT 1993).

A test of homogeneity of the within-covariance matrix is necessary in order to decide to use the individual within-categories covariance matrices or the pooled covariance matrix (equation 43).

The following notation is used to describe the generalized square distance ("Mahalanobis' D^2 "): t is a subscript to distinguish groups; S_t the covariance matrix within group; $|S_t|$ the determinant of S_t ; S the pooled covariance matrix; x a vector containing the variable of an observation; $DF^{(t)}$ the degree of freedom of the i group; q_t the prior probability for group t ; and m_t the vector containing the means of the variables in the group t .

The generalized squared distance from x to group t is:

$$D_t^2(x) = g_1(x,t) + g_2(t) \dots\dots\dots(35)$$

Where

$$g_1(x,t) = (x - m_t)' S_t^{-1} (x - m_t) + \log_e |S_t| \dots\dots\dots(36)$$

If the within-group covariance matrices are used, or

$$g_1(x,t) = (x - m_t)' S_t^{-1} (x - m_t) \dots\dots\dots(37)$$

If the pooled covariance matrix is used; and

$$g_2(t) = -2 \log_e(q_t) \dots\dots\dots(38)$$

If the prior probabilities are not all equal, or

$$g_2(t) = 0 \dots\dots\dots(39)$$

if the prior probabilities are all equal.

The goal of the discriminant analysis is to assign observations to the groups which they have the greatest resemblance while at the same time, minimizing the effects of misclassification. Prediction, therefore, has two aspects which require the specification of a classification criterion and the formation of measures of

resemblance. The assumption of multivariate normality permits the construction of measures of resemblance based upon the m characteristics which describe each observation. By substituting the values of an observation's characteristics into each group's probability density function, it is possible to estimate the proportion of each population's members lying outside the range defined by the values of the observation's m characteristics. This proportion serves as an index how closely a given observation "resembles" the group as compared with the rest of the population. This probability can be used as a measure of resemblance (Eisenbeis and Avery 1972). Hence the posterior probability of an observation x belonging to group t is

$$p_t(x) = \frac{\exp(-0.5D_t^2(x))}{\sum_{\mu} (\exp(-0.5D_{\mu}^2(x)))} \dots\dots\dots(40)$$

An observation is classified into group μ if setting $t=\mu$ produces the smallest value of $D_t^2(x)$ or the largest value of $p_t(x)$.

The test of homogeneity of within covariance matrices is expressed as:

$$V = \frac{\prod | \text{Within } S_i |^{N/2}}{| \text{Pooled } S |^{N/2}} \dots\dots\dots(41)$$

$$\rho = 1 - \left[\sum \frac{1}{N(i)} - \frac{1}{N} \right] \frac{2P^2 + 3P - 1}{6(P+1)(K+1)} \dots\dots\dots(42)$$

$$DF = 0.5(K-1)P(P+1) \dots\dots\dots(43)$$

Where DF=degree of freedom; k =number of groups; P =number of variables; N =total number of observations-number of groups; and N_i =number of observations in each group. Under null hypothesis:

$$-2\rho \ln \left[\frac{N^{PN/2} V}{\prod N_i^{PN_i/2}} \right] \dots\dots\dots(44)$$

Is distributed as chi-square (DF). If the chi-square values is not significant at 0.1 level, a pooled covariance matrix is used in the discriminant function. the

parameter V (sample discriminant function for parameters unknown) evaluated in the equation (45) become:

$$V = [x - 1/2(m_1 + m_2)]^T S^{-1}(m_1 - m_2) \dots \dots \dots (45)$$

The probability of identifying a site as non-liquefiable (P_1) when it will in fact liquefy is equal to the following expression when the prior probabilities are equal (Kleinbaum and Kupper 1978):

$$P_1 = \Phi(-\sqrt{D_1^2}/2) \dots \dots \dots (46)$$

Where Φ is the standard normal cumulative function; and D_1^2 is the Manalanobis' distance defined by equation (35). Lachenbruch (1975) pointed out that these estimates themselves may be misleading for small sample sizes and, on the basis of sampling experiments, has suggested using the following expression in place of D_1^2 in equation 46:

$$\frac{N_1 + N_2 - \rho - 3}{N_1 + N_2 - 2} D^2 \dots \dots \dots (47)$$

3.4.2 Logit Analysis

Logit analysis or logistic regression is a very powerful and flexible tool for analyzing liquefaction data. Essentially, the objective of the logit analysis is to obtain an expression for the conditional probability of liquefaction P as:

$$P = \frac{1}{1 + e^{-(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \dots + \beta_m X_m)}} \dots \dots \dots (48)$$

$$\logit[P] = \ln\left[\frac{P}{1-P}\right] = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \dots + \beta_m X_m \dots \dots \dots (49)$$

In the above equations the x_k 's are various explanatory variables such as cyclic stress ratio, corrected SPT resistance, corrected CPT resistance or mean diameter and some of the variables can themselves be binary, explanatory variables of this type

are often called “indicator” variables. The β_k 's are regression coefficients to be obtained by fitting equation (57) with data. Methods to obtain β_k values include a least-squares formulation and the maximum likelihood approach. The second one will be used due to its statistical properties: consistency, asymptotic normality distributed and asymptotic efficiency (Cook and Gross 1968), the last two properties are especially relevant in developing goodness-of-fit statistics for the logit models.

The likelihood or probability l_i of observing either $Y=1$ or $Y=0$ for the i th case is:

$$l_i = P_i^{Y_i}(1 - P_i)^{1 - Y_i} \dots\dots\dots(50)$$

Since $Y_i=0$ or $Y_i=1$, $l_i=P_i$ in the case of liquefaction, and $l_i=(1-P_i)$ for non-liquefaction. If there are n independent observations, then the joint probability of occurrence of all observations is:

$$l = l_1 l_2 l_3 \dots l_m = \prod_{i=1}^m l_i = \prod_{i=1}^m P_i^{Y_i}(1 - P_i)^{1 - Y_i} \dots\dots\dots(51)$$

This is the likelihood function that needs to be sought to maximize with respect to the parameters β_k . In practice, what is commonly done is to maximize the log of the likelihood function rather than the likelihood function itself. This does not affect the values of the β estimates, since the logarithmic function is a monotonic one-to-one transformation. Hence the L denote the log-likelihood function and write as:

$$L = \ln(l) = \sum_{i=1}^m [Y_i \ln P_i + (1 - Y_i) \ln(1 - P_i)] \dots\dots\dots(52)$$

In which P_i is the probability of liquefaction and Y_i is the response and can be either 0 or 1. To obtain the maximum likelihood estimates of β_k (where $\partial L / \partial \beta_k = 0$, for all k), an iterative solution based on the Newton-Raphson formula is used. At the r^{th} step, one calculates:

$$\{\beta_k\}_{r+1} = \{\beta_k\}_r - \left[\frac{\partial^2 L}{\partial \beta_j \partial \beta_k} \right]_r^{-1} \left\{ \frac{\partial L}{\partial \beta_k} \right\}_r \dots\dots\dots(53)$$

Where

$$\frac{\partial L}{\partial \beta_k} = \sum_{i=1}^n x_{ik}(Y_i - P_i)$$

$$\frac{\partial^2 L}{\partial \beta_j \partial \beta_k} = \sum_{i=1}^n -x_{ij}x_{ik}(1 - P_i)P_i \dots \dots \dots (54)$$

At zero-th step, $\{\beta_k\}$ is usually initialized by setting $\beta_k = 0$ for all k . Convergence is achieved usually within 5 to 10 iterations. Grigg (1973) and Liao (1986) has developed programs to solve the algorithms also is possible solve through the logit procedure of SAS/STAT software (1993).

The next step is to perform a goodness-of-fit statistics to decide whether a proposed model is statistically significant and to compare various competing models in trying to decide which is "best". Since a large majority of the observations are unique in the sense that in general no two observations yield identical values on all variables, the fitted model cannot be evaluated using chi-square statistics (χ^2) goodness of fit tests.

To obtain a goodness of fit test, Hosmer and Lemeshow suggest that the range of probability, ρ [0,1], be divided into s mutually exclusive categories and then a comparison of the observed and predicted frequencies be carried out using χ^2 statistics. The categories can be determined by ranking the n ρ -values (n is the number of observations) and then dividing them into s equal groups or by dividing the range of ρ into s equal intervals. The statistics will be expressed as:

$$\sum_{j=1}^s \frac{(o_j - n_j \bar{\rho}_j)^2}{n_j \bar{\rho}_j (1 - \bar{\rho}_j)} \approx \chi^2 \dots \dots \dots (55)$$

Where o_j is the actual frequency of the group j ; n_j is the predicted frequency, and $\bar{\rho}^2$ is the average value of ρ in group j . Expression 55 is approximately χ^2 with $(s-2)$ degrees of freedom if the fitted logistic regression model is correct (Jobson 1992).

CHAPTER IV

EMPIRICAL CHARTS

4.1 MODEL FORMULATION

In statistical analysis models represent the boundary between liquefaction and non liquefaction for a probability of 0.5. In order to select an adequate model is necessary to combine statistical test and some level of judgment. The following issues were considered in the formulation of the models: statistical significance, accuracy of estimation, physical interpretation, and biases due to particular features of the data. The two first issues can be test and estimate from the statistical calculations, but the two last issues are more complicate and require some level of judgment.

Under these considerations, a local model was proposed based in the expression proposed by Seed and Idriss (see section 2.2.1) with a normalized magnitude of $M=7.5$. The problem to select an adequate set of variables representing the liquefaction phenomenon is still a subject of discussion. Although previous works (Seed et al. 1984, Liao 1986, Shibata and Teparaksa 1988, and Reyna 1991) show the importance of considering a variable that can effectively represents the grain size distribution, some authors chosen the fines content (FC) such this factor and others the mean grain size (D_{50}). Since the major limitation of an empirical correlation is the characteristics of the cases considered in the analysis, and most of the cases collected are clean sands, hence in the present wok the mean grain size (D_{50}) is considered such a representative variable for grain size distribution. The influence of the grain size is discussed in section 1.4.6.

Considering all these conditions six models were proposed to perform logistic and discriminant analysis. The following expressions represent the general model formulation for logistic and discriminant analysis.

$$R \leq \text{CSRN} \cdot D_{50}^b \cdot c \quad (\text{model 1})$$

$$R \leq \text{CSRN} \cdot c \quad (\text{model 2})$$

$$R \leq a \cdot \text{CSRN} + b \cdot D_{50} + c \quad (\text{model 3})$$

$$R \leq a \cdot \text{CSRN} + c \quad (\text{model 4})$$

$$R \leq a \cdot \ln(\text{CSRN}) + b \cdot D_{50} + c \quad (\text{model 5})$$

$$R \leq \exp(a \cdot \text{CSRN} + b \cdot D_{50} + c) \quad (\text{model 6})$$

Where R is the modified resistance equal to q_{e1} for the CPT data and $(N_1)_{60}$ for SPT data, CSRN is the normalized cyclic stress ratio, and a, b, c, and d are regression constants to be determined.

4.2 CONE PENETRATION TESTING (CPT)

4.2.1 Discriminant Analysis

The discriminant analysis was performed with the data collected from data catalogs show in Appendix. The discriminant analysis was applied in order to obtain the discriminant expression (model) for the modified cone penetration resistance (q_{e1}) as a function of normalized cyclic stress ratio (CSRN), and mean diameter (D_{50}). Following the procedure described in section 3.4.1. for the first model the within-group covariance matrix S_t , was calculated as:

$$S_t = \begin{bmatrix} \text{Var}(\ln \text{CSRN})^t & \text{Cov}(\ln \text{CSRN}, \ln q_{c1})^t & \text{Cov}(\ln \text{CSRN}, \ln D_{50})^t \\ \text{Cov}(\ln \text{CSRN}, \ln q_{c1})^t & \text{Var}(\ln q_{c1})^t & \text{Cov}(\ln q_{c1}, \ln D_{50})^t \\ \text{Cov}(\ln \text{CSRN}, \ln D_{50})^t & \text{Cov}(\ln q_{c1}, \ln D_{50})^t & \text{Var}(\ln D_{50})^t \end{bmatrix} \quad (56)$$

and the first element of the pooled covariance matrix is given by:

$$S_{(1,1)} = \frac{\text{Var}(\ln \text{CSRN})^{(1)} \text{DF}^{(1)} + \text{Var}(\ln \text{CSRN})^{(2)} \text{DF}^{(2)}}{\text{DF}^{(1)} + \text{DF}^{(2)}} \quad (57)$$

For the other five models the pooled covariance matrix as the same characteristics. The data analysed is summarize in Fig. 12. The values of cyclic stress ratio are normalized to a magnitude of 7.5 (CSR_N) following the Seed and Idriss formulation (equation 25) and the values of q_{c1} correspond to the critical depth (depth with the least value of q_{c1}). This approach may produce conservative results but taking more than one point for the same site and earthquake (Teparaksa 1988) could introduce biases (see discussion in section 3.2.3). The total number of cases analyzed was 56, in which 37 represent liquefaction cases and 19 non-liquefaction cases, the characteristics of each case (soil parameters, seismic action and source) are provided in Appendix.

The DISCRIM procedure of SAS/STAT software was used in order to performed the discriminant analysis. This software provides the within or pooled covariance matrix and the statistical properties of the data. Table 3 shows the cases that were misclassified based on the generalized square distance. The prior probabilities for each observation are known after perform the procedure, and thus are assumed to be equal. Therefore the function $g_2(t)$ is equal to 0, and $D^2(t)$ becomes equal to g_1 (equation 37). Each case has been placed in the class from which it has the smallest generalized squared distance.

Fig 13 shows the lower boundary for values of D_{50} , only few exceptions fall below the lower line like liquefaction case 377 (Shibata and Teparaksa 1988).

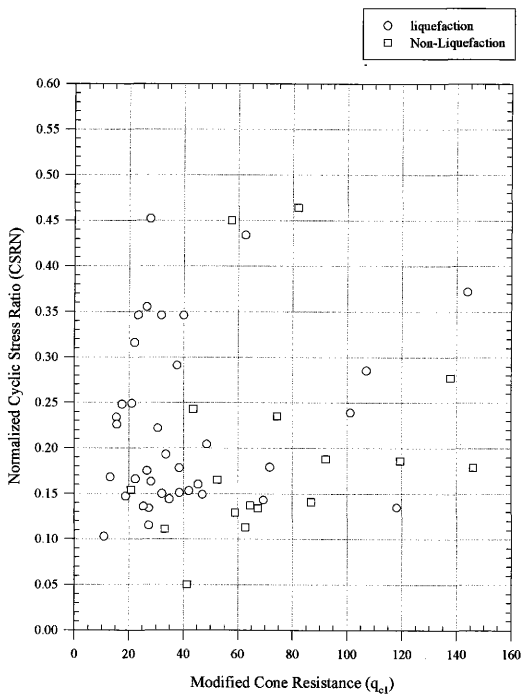
**FIG.12. Cone Data used in Statistical Analysis**

TABLE 4. Classification of Observations

Model		All Data	Liquefaction	No Liquefaction
Model 1	N.Misclassified	9	6	3
	N. Cases	56	34	22
Model 2	N.Misclassified	13	8	5
	N. Cases	56	34	22
Model 3	N.Misclassified	11	4	7
	N. Cases	56	40	16
Model 4	N.Misclassified	12	6	6
	N. Cases	56	37	19
Model 5	N.Misclassified	10	4	6
	N. Cases	56	37	19
Model 6	N.Misclassified	12	9	3
	N. Cases	56	31	25

Table 4 shows the number of cases misclassified as a result of performed different models, these numbers give also a reference if the model is statistically significant or how effective is the model in classify the cases.

The procedure used to develop the discriminant function was to evaluate a parameter V (sample discriminant function for parameters unknown) defined by the following equation:

$$V = [x - 1/2(m_1 + m_2)]^T S^{-1}(m_1 - m_2) \dots\dots\dots(58)$$

Where x is the set of variables to be determined, m_1 and m_2 are the vector of means of the variables, and S^{-1} is the inverse of the within covariance matrix.

A new site that has not yet experienced an earthquake should be assigned to the nonliquefaction category if V is greater than or equal to a predetermined constant C . For an equal cost of misclassification $C=V=0$. The equations for the required cone resistance, q_{c1} is obtained replacing the values obtained from the DISCRIM procedure (SAS/STAT 1993) in equation 47 considering a value of V equal to 0. The equations are presented in Table 5.

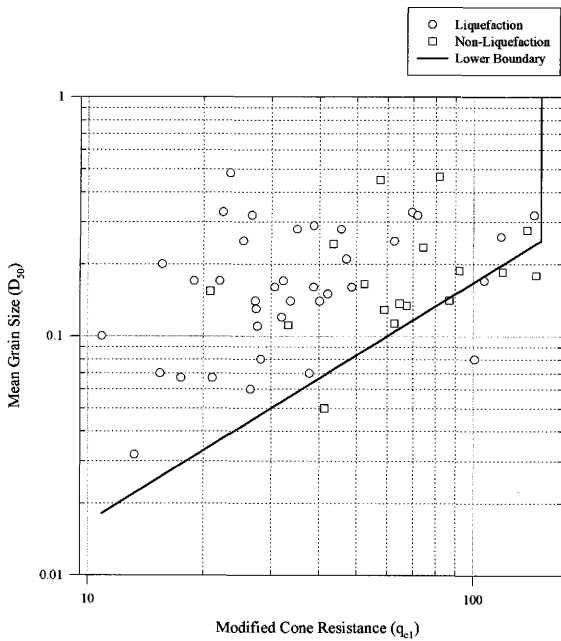


FIG.13. Lower Boundary for Mean Grain Size (D_{50}) and Cone Resistance (q_{c1})

TABLE 5. Discriminant Model for CPT Data

Model	Expression	χ^2 value	Prob> χ^2
1	$q_{cl} \leq 273.99 \text{ CSRN}^{0.6857} D_{50}^{0.4555}$	2.3526	0.8846
2	$q_{cl} \leq 157.62 \text{ CSRN}^{0.6821}$	2.2563	0.5209
3	$q_{cl} \leq 140.7547 \text{ CSRN} + 142.8568 D_{50} + 6.6805$	4.9972	0.5442
4	$q_{cl} \leq 133.9881 \text{ CSRN} + 34.3380$	3.5703	0.3117
5	$q_{cl} \leq 40.5967 \ln(\text{CSRN}) + 142.3080 D_{50} + 104.3617$	5.3798	0.4961
6	$q_{cl} \leq \exp(2.436797 \text{ CSRN} + 2.516253 D_{50} + 2.940123)$	2.2643	0.8939

The probability of identifying a site as non-liquefiable, P_1 , is calculated using the expression 48 considering the case of equal cost of misclassification ($V=0$). The results vary from 0.317 for the fourth model to 0.8846 for the first model. According with the procedure presented in section 3.4.1 the probability P_1 is one of the factors to be consider in order to choose the best fit model.

Under this consideration the model that has the lowest probability of equal cost of misclassification is model fourth. Model five has the lowest number of misclassified cases which are the second factor to be consider in the selection of a best discriminant expression and the second lowest probability. Based in the considerations presented in section 4.1, the model that has the best physical representation (concave upward curvature). The following graph shows the discriminant curves for model 5 in a range of values of the mean grain size (D_{50}).

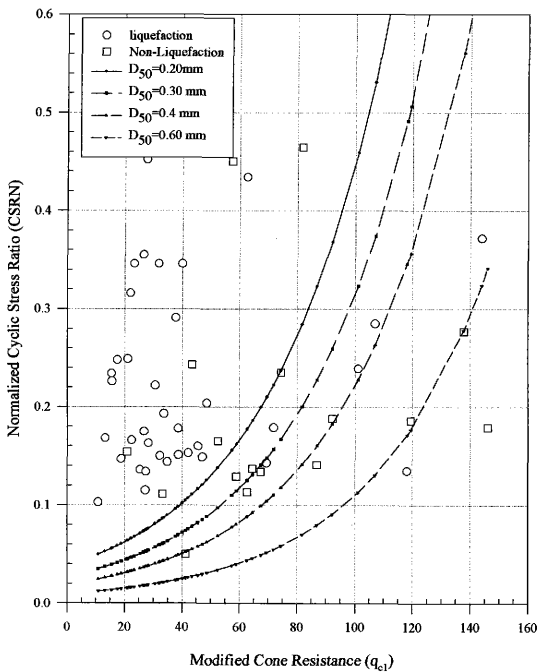


FIG.14. Discriminant Curves for Model 5

4.2.2 Logit Analysis

The logistic regression analysis was performed with the same set of data than the discriminant analysis. The same models were also consider, in order to compare the results between procedures. The details of the logistic analysis are shown in section 3.4.

The LOGIT procedure of the SAS/STAT software (1993) was used to analyze the data also for purposes of comparison the PROBIT procedure was also performed. Since the posterior probability will be consider equal for liquefaction and non-liquefaction ($P=0.5$), $\text{logit}(P)$ become equal to 0. The expressions obtained are shown in table 6.

TABLE 6. Logistic Models for CPT Data

Model	Equation	Goodness-of-fit-statistics	ρ
1	$q_{cl} \leq 605.44\text{CSRN}^{0.75146}\text{D}_{50}^{0.72118}$	3.6781	0.8160
2	$q_{cl} \leq 226.73\text{CSRN}^{0.71759}$	4.1914	0.7575
3	$q_{cl} \leq 176.0507\text{CSRN} + 189.36\text{D}_{50} + 5.8427$	11.736	0.1096
4	$q_{cl} \leq 164.85\text{CSRN} + 46.3894$	7.063	0.4223
5	$q_{cl} \leq 147.44\ln(\text{CSRN}) + 185.327\text{D}_{50} + 121.94$	13.695	0.0596
6	$q_{cl} \leq \exp(2.6968\text{CSRN} + 3.30\text{D}_{50} + 3.00)$	6.7132	0.4593

For logistic analysis the model that has the best performance in terms of the goodness-of-fit statistics is model 5. The following graph shows the curves generated with model 5 for different values of mean diameter (D_{50}).

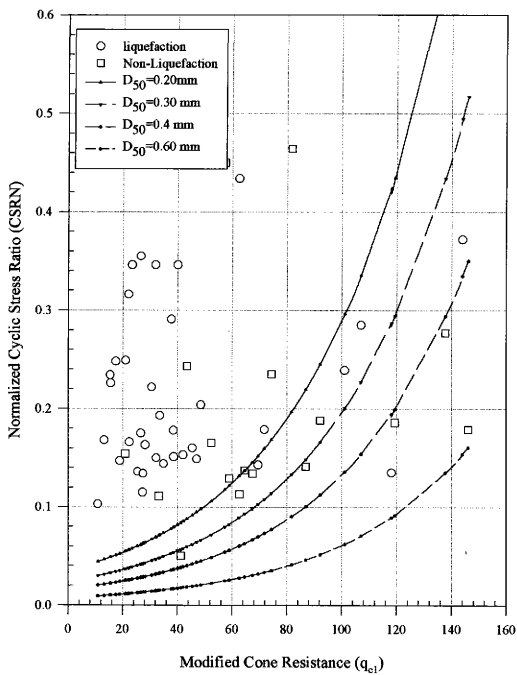


FIG. 15. Logistic Curves for Model 5

4.3 STANDARD PENETRATION TEST (SPT)

4.3.1 Discriminant Analysis

The discriminant analysis has been used previously by Christian and Swieger (1975) with summarized SPT data. The relationship for SPT and cyclic stress ratio using the discriminant analysis was done considering the median grain size such a representative factor of grain size.

The discriminant analysis was applied in order to obtain the discriminant expression for the normalized penetration resistance $(N_1)_{60}$ as a function of normalized cyclic stress ratio (CSRN), and mean diameter (D_{50}). Following the procedure described in section 3.4.1. for the first model the within-group covariance matrix S_1 , was calculated as:

$$S_1 = \begin{bmatrix} \text{Var}(\ln \text{CSRN})^t & \text{Cov}(\ln \text{CSRN}, \ln(N_1)_{60})^t & \text{Cov}(\ln \text{CSRN}, \ln D_{50})^t \\ \text{Cov}(\ln \text{CSRN}, \ln(N_1)_{60})^t & \text{Var}(\ln(N_1)_{60})^t & \text{Cov}(\ln(N_1)_{60}, \ln D_{50})^t \\ \text{Cov}(\ln \text{CSRN}, \ln D_{50})^t & \text{Cov}(\ln(N_1)_{60}, \ln D_{50})^t & \text{Var}(\ln D_{50})^t \end{bmatrix} \quad (59)$$

and the first element of the pooled covariance matrix is given by:

$$S_{(1,1)} = \frac{\text{Var}(\ln \text{CSRN})^{(1)}DF^{(1)} + \text{Var}(\ln \text{CSRN})^{(2)}DF^{(2)}}{DF^{(1)} + DF^{(2)}} \quad (60)$$

For the other five models the pooled covariance matrix as the same characteristics. The data analysed is summarized in Fig. 14. The values of cyclic stress ratio are normalized to a magnitude of 7.5 event (CSRN) following the Seed and Idriss formulation (equation 25) and the values of q_{c1} correspond to the critical depth (depth with the least value of q_{c1}). This approach may produce conservative results but taking more than one point from the same site and earthquake (Teparaksa 1988) could introduce biases (see discussion in section 3.2.3). The total number of cases

analyzed was 56, in which 37 represent liquefaction cases and 19 non-liquefaction cases, the characteristics of each case (soil parameters, seismic action and source) are provided in Appendix.

Since the mean grain size (D_{50}) is the other variable included in the analysis is important know the relation with the cone penetration resistance (q_{c1}) and defined a lower bound state line for the liquefaction cases considering in the different models.

The DISCRIM procedure of SAS/STAT software was used in order to performed the discriminant analysis. This software provides the within or pooled covariance matrix and the statistical properties of the data. Table 3 shows the cases that were misclassified based on the generalized square distance. The prior probabilities for each observation are known after perform the procedure, and thus are assumed to be equal. Therefore the function $g_2(t)$ is equal to 0, and $D^2(t)$ becomes equal to g_1 (equation 37). Each case has been placed in the class from which it has the smallest generalized squared distance.

The lower bound line for SPT data is shown in the next figure. The limit N-value is 25 for a mean grain size (D_{50}) of 0.15, values that have a good correlation with the results of Seed et al. (1983 Fig. 11), in which approximately the discriminant curve is asymptotic at N-value of 28 for a $D_{50} < 0.15$ mm, hence the lower bound line obtained is a good representation of the boundary line.

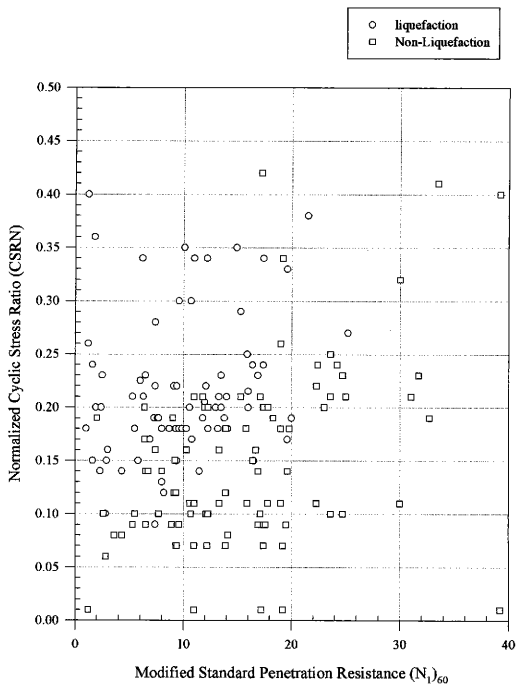


FIG.16. Standard Penetration Data used in Statistical Analysis

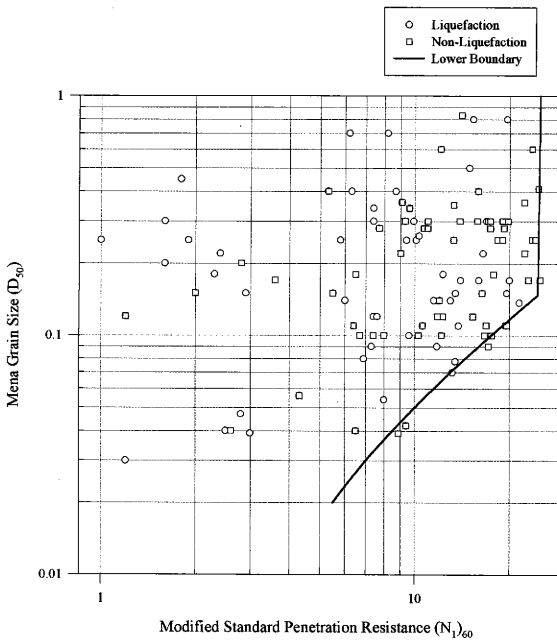


FIG.17. Lower Boundary for mean Grain Size and Standard Penetration Resistance

After perform the DISCRIM procedure (SAS/STAT 1993) in each model, some of the cases are misclassified, providing a reference of how well is the model to represent the discriminant curve for the data. Table 7 presents the number of misclassified cases for each model.

TABLE 7. Classification of SPT Observations

Model	Means	All Data	Liquefaction	No Liquefaction
Model 1	N. misclassified	38	10	28
	N. Cases	165	93	72
Model 2	N. misclassified	42	7	35
	N. cases	165	103	62
Model 3	N. misclassified	27	9	18
	N. cases	165	84	81
Model 4	N. misclassified	29	10	19
	N. cases	165	84	81
Model 5	N. misclassified	23	9	23
	N. cases	165	89	79
Model 6	N. misclassified	38	28	10
	N. cases	165	57	108

Substituting the values in equation 58 with the program outputs is possible obtained the discriminant expression for each model, under the consideration of equal cost of misclassification ($V=0$). Table 8 shows the discriminant models for SPT Data.

TABLE 8 . Discriminant Models for SPT Data

Model	Expression	χ^2 value	Prob> χ^2
1	$(N_1)_{00} \leq 124.337 \text{ CSRN}^{1.2896} D_{50}^{0.0099}$	64.1130	0.0001
2	$(N_1)_{00} \leq 116.87 \text{ CSRN}^{1.3193}$	56.2886	0.0001
3	$(N_1)_{00} \leq 92.8013 \text{ CSRN} + 5.5944 D_{50} - 5.4305$	25.3466	0.0003
4	$(N_1)_{00} \leq 95.1368 \text{ CSRN} - 4.5377$	12.8467	0.0050
5	$(N_1)_{00} \leq 12.07312 \ln(\text{CSRN}) + 6.189683 D_{50} + 33.61134$	75.0750	0.0001
6	$(N_1)_{00} \leq \exp(9.763077 \text{ CSRN} + 0.27394 D_{50}) + 0.3884$	22.5624	0.0010

From the point of view of the probability of equal cost of misclassification model five has the best fit and also the least number of misclassified. Hence the model 5 is considered the best fit model. The following graph shows the curves generated with model 5 for different values of mean diameter (D_{50}).

4.3.2 Logit Analysis

The procedure follows in order to analyze the SPT data is presented in detail in section 3.4.2. Since the posterior probability will be consider equal for liquefaction and non-liquefaction ($P=0.5$), $\text{logit}(P)$ become equal to 0 and the logit regression results of models are showed in table 9.

TABLE 9. Logistic Model for SPT Data

Model	Equation	Goodness-of-fit-statistics	ρ
1	$(N_1)_{60} \leq 214.1657 \text{CSRN}^{1.61435} D_{50}^{0.1095}$	10.963	0.2038
2	$(N_1)_{60} \leq 203.568 \text{CSRN}^{1.69380}$	9.1003	0.3339
3	$(N_1)_{60} \leq 93.6194 \text{CSRN} + 6.60644 D_{50} - 6.5385$	11.736	0.1096
4	$(N_1)_{60} \leq 96.3572 \text{CSRN} + 5.5716$	7.3225	0.5023
5	$(N_1)_{60} \leq 17.58659 \ln(\text{CSRN}) + 6.9985 D_{50} + 41.5207$	4.8703	0.7713
6	$(N_1)_{60} \leq \exp(9.167 \text{CSRN} + 0.81757 D_{50} + 0.4368)$	6.7132	0.4593

From the statistic goodness of fit values model 4 is has the best fit. Model 4 represents a straight line and from the point of view of the physical representation of the liquefaction phenomenon the shape of the empirical relation should be upward concave curve. From this consideration model 5 was chosen as the best fit model, and its graphical representation is show in the next figure.

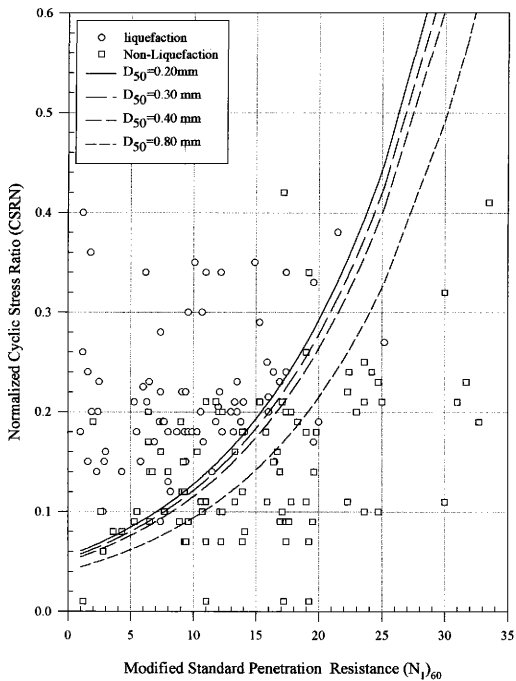


FIG.18. Discriminant Curves for Model 5

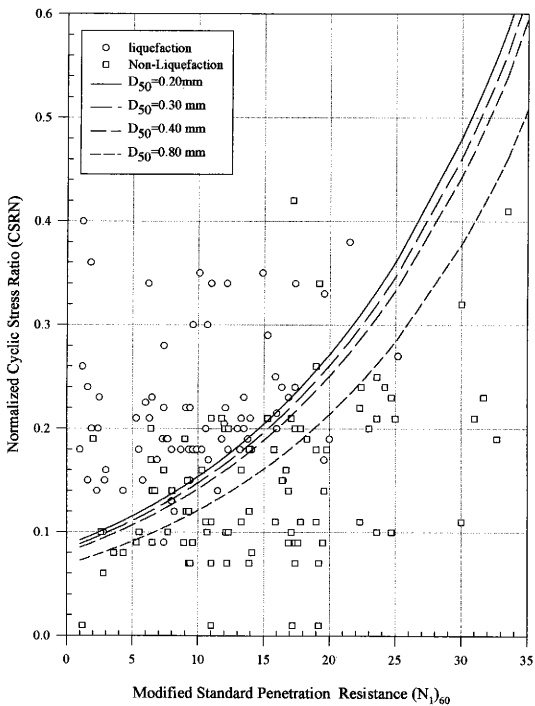


FIG.19. Logistic Curves for Model 5

4.4 DISCUSSION OF RESULTS

The variables considered in the statistical analysis are the resistance factor, the cyclic stress ratio and the mean diameter. The fine's content was not take into consideration based in previous works done by Liao (1988) and Goh (1994) show that the fines content does not improve the correlation factor.

Koester (1992) establish that plasticity index has more influence in the behavior of silty sands, since the present work is oriented to clean and silty sand with a fine content less than 15%, the influence of plasticity was not take into account.

Since the mean grain size was considered representative of the grain size characteristics it is important compares the behavior of the two statistical expressions with the actual data for a specific range of D_{50} .

In the case of the CPT data the two models show a good discrimination behavior for 0.2 mm and 0.3 mm of D_{50} (Fig. 20 and Fig. 21). It is important notice that most of the cases are in the range of 0.1 to 0.4 of normalized cyclic stress ratio (CSR_N). Range where the results coincide with most of the methods available, the discrepancy is in the range of 0.4 to 0.6 of CSR_N (Fig. 22). This region has not enough number of cases to determine with accuracy a discriminative curve.

The logistic regression expression obtained has a good performance from the point of view of the statistical results and also by comparing these with the discriminant curve and previous empirical correlation's. Showing the major differences for values of CSR_N higher than 0.4 corresponding to values of q_{c1} approximately of 120 kg/cm². Since these are high values the occurrence of failure is unlikely. In the zone of more frequent events the curve shows a better discriminant performance with respect to the previous curves (Fig.22).

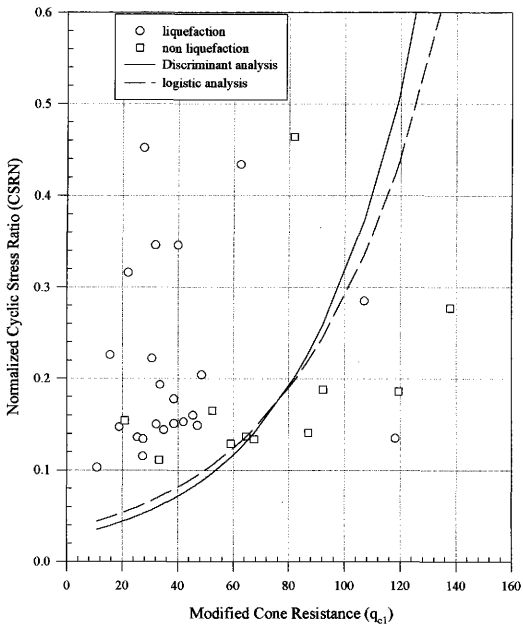


FIG.20. Statistical Curves and Data Cases for $D_{50}=0.10-0.30$ mm

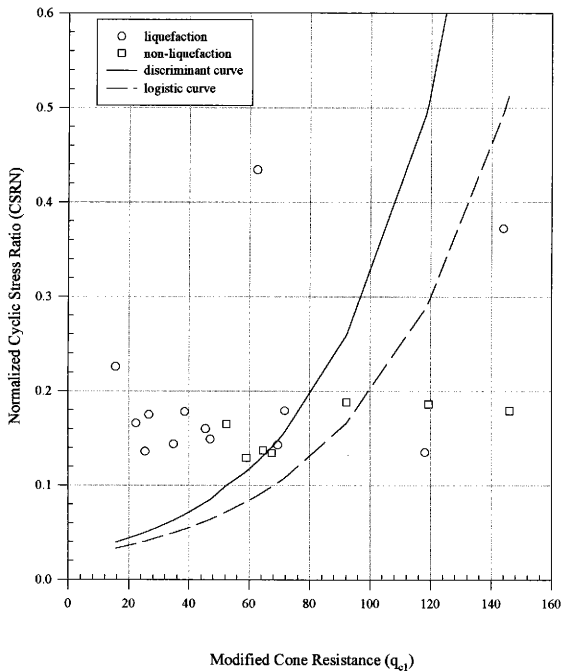


FIG.21. Statistical Curves and Data Cases for $D_{50}=0.20-0.40$ mm

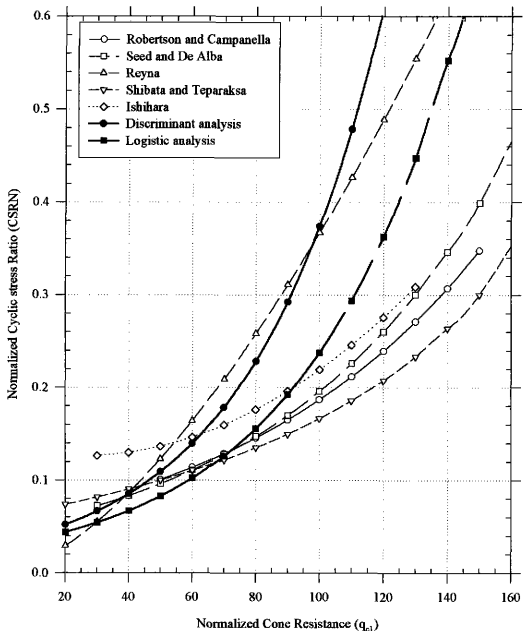


FIG.22. Comparison of empirical curves for CPT Data and $D_{80}=0.25$ mm

One of the probable reasons is the number of cases collected for this work that is higher than the cases used for Robertson and Campanella (1985) and Reyna (1991), the case of Shibata and Teparaksa (1984) is different since these authors develop their relationship using more than one depth for each case.

For the Seed et al. (1985) correlation the main problem is that the author developed the curves based in SPT data correlated with the CPT data, incorporating more uncertainties in their undirected analysis.

To analyze the SPT data base some normalization and correction factors were applied to all selected cases (Fig. 16). Five statistical models were run for discriminant and logistic analysis. The results obtained from discriminant analysis shows (Table 7) that model 5 give the minimum number of misclassified cases (23) compared with the other four models. Logistic analysis for model 5 also gives the highest χ^2 value and the minimum probability. From the statistical point of view model 5 is the best discriminant curve.

Figures 23 and 24 show the curves obtained from logistic and discriminant analysis compared with the available cases for different mean diameters. For the $(N_1)_{60}$ values in the range of 0 to 18 the discriminant curve tends to be conservative and then switch to the unconservative side but in this range the discriminant curve misclassified a large number of cases, hence, the logistic expression is a better boundary for this range.

The tendency of the logistic curve agrees with the tendency show in previous works done by Seed et al. (1985) and Liao (1988). In the selection of the best discriminant curve is important take into consideration the statistical results and also the tendency of the curves, hence, model 5 obtained with logistic analysis is the best model (Fig.25).

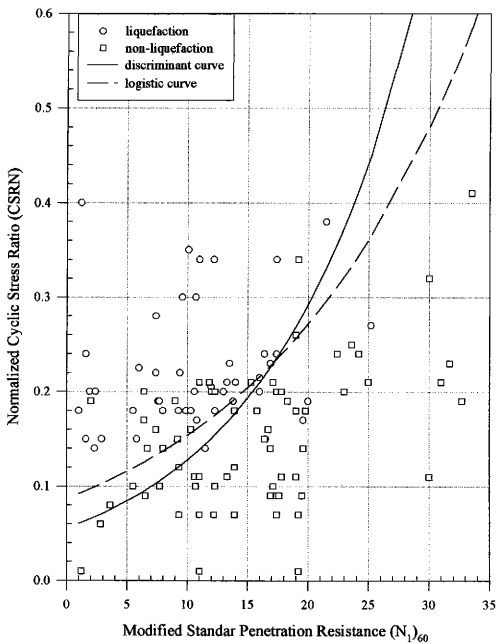


FIG.23. Statistical Curves and Data Cases for $D_{50}=0.10-0.30$ mm (SPT)

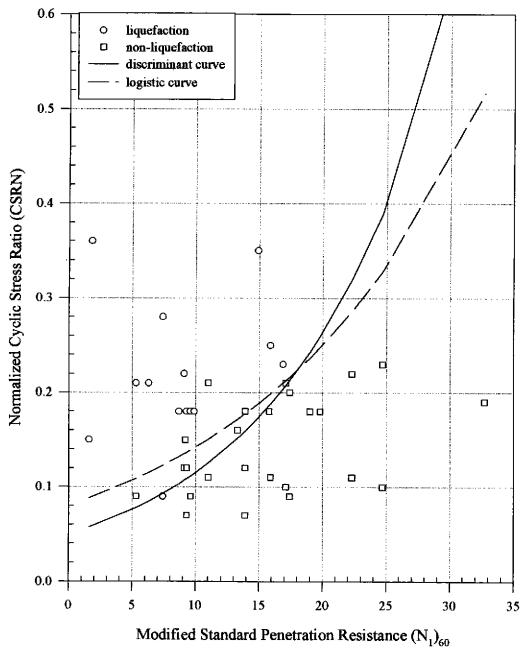


FIG.24. Statistical Curves and Data Cases for $D_{50}=0.20-0.40$ mm (SPT)

There is a good correlation between the two statistical methods in the range of 0 to 0.4 for the normalized cyclic stress ratio (CSR_N) when is analyzed the SPT data base. Most of the non-liquefaction cases have a CSR_N over 0.4 and the two methods show a considerable difference for this range.

The charts developed show a good correlation with other authors in the range of 0-20 for the SPT values, for higher values of N the difference is significant. The main reason is the conservative approach of some author due to the study cases available for $(N_1)_{60}$ higher than 30, Seed et al. (1983) consider an asymptotic curve at point 28 of $(N_1)_{60}$ values.

The comparison of the empirical chart is valid only for clean and silty sands with a fine content less than 15%. For fine content over this value some corrections should be made in N-values to take into consideration the influence of plasticity and fines.

Figure 25 shows the comparison between the available empirical charts and the tendency of the logistic curve is similar to Seed et al (1985) and Liao (1988). But this tends to be less conservative. This difference is based in the number of cases analyzed by previous authors. Liao (1988) used a data base of 278 cases and Seed and Idriss (1971) 34 cases. Another factor is the inclusion of the mean diameter as a variable that takes into account the grain distribution, Seed et al (1985) considering the fines contents the variable that represents the grain distribution.

One limitation of the analysis done is the number of cases that is less than 300, number recommended to run logistic and discriminant analysis. Another limitation is the bias introduced for the different number of liquefaction and non-liquefaction cases. Usually liquefaction cases tend to be more than non-liquefaction cases and it's has an influence in the boundary expression that tends to be conservative.

In order to improve the design charts developed is necessary increase the number of liquefaction and non-liquefaction cases. This process can be done each time that new cases are available. Using model 5 and logistic analysis is possible

update the design charts for SPT and CPT. More variables can be incorporated in the model if the information is available.

The definition of variables depends in some way of the available information, is possible has an empirical model with more variables but with less number of cases or a model with less variable and more cases, for this work the second option was selected.

Most of the available cases have mean diameter but only few of them have fine's content or plasticity. The original sources of most of the cases are difficult to obtain and sometimes the author made some corrections to the original data that has not been well documented.

Others in-situ tests do not have a data base that can let created an empirical correlation most of the correlation's is based in laboratory results and needs to be compared with in-situ results. For the pressuremeter test is very difficult obtained good in-situ result specially if it is take into consideration that the test is done under the water level in cohesionless soils. Unless a freezing technique is applied it will be very difficult run this type of test. The standard penetration test and the cone penetration test are common and have a large data base and a large number of available correlations some of them based on laboratory results and some other just empirical correlations.

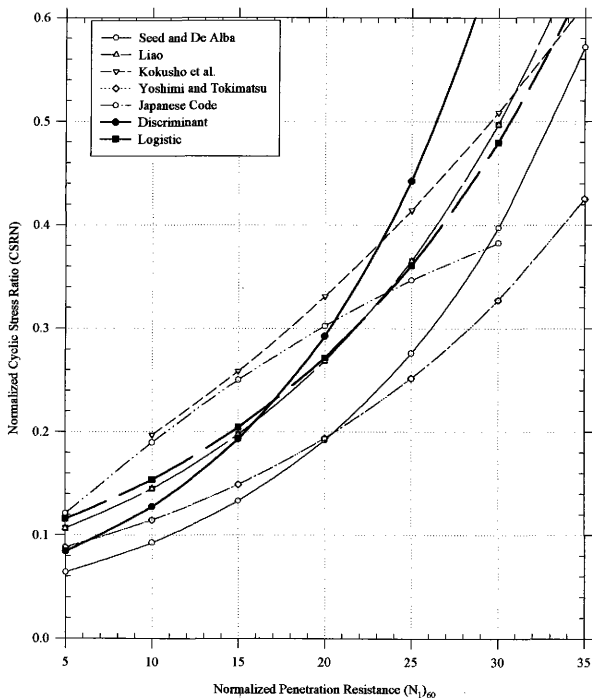


FIG.25. Comparison of empirical curves for SPT Data and $D_{50}=0.25$ mm

Figure 26 shows the correlation between the standard and cone penetration values against the normalized cyclic stress ratio (CSR_N), in this graph the logistic curve for SPT and CPT results were plotted as one curve in order to compare the two statistical tools. The tendency is the same but the discriminant is less conservative for values of $(N_1)_{60}$ more than 20 and q_{c1} values more than 130.

Figure 27 is a final design graph for logistic model under different mean diameter for SPT and CPT test. This chart can be used after correct the penetration values obtained in-situ and only for clean and silty sands with a fine content less than 15%. From figure 27 is possible known that the CPT results are more conservative but follow the same tendency than the SPT results.

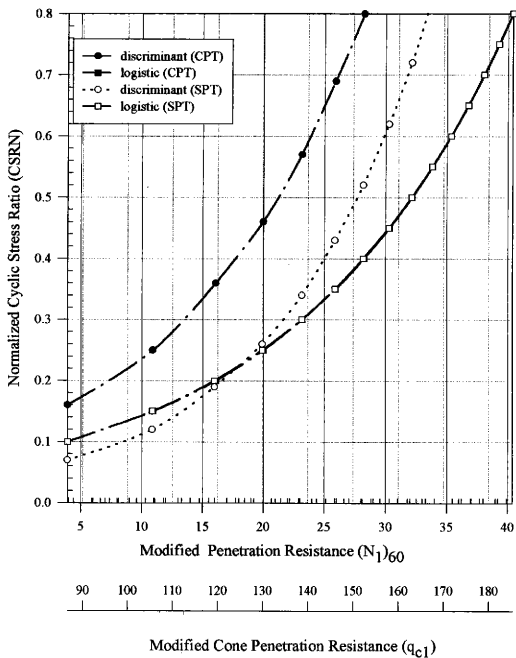


FIG.26. Correlation Between q_{c1} , $(N_1)_{60}$ and CSRN

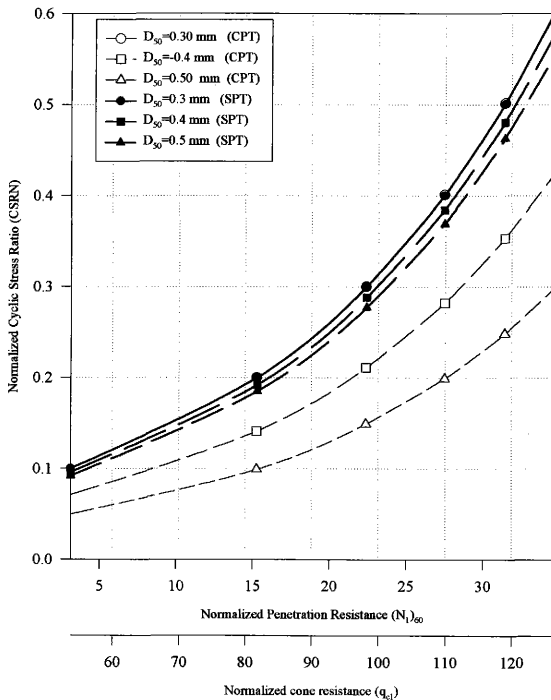


FIG 27. Design Chart against liquefaction for CPT and SPT values

CHAPTER V

SUMMARY AND CONCLUSIONS

In-situ tests provide an adequate source of information to deal with the prediction of liquefaction occurrence, although the interpretation of the results can vary widely depending of the methods used. The standard penetration test (SPT) is one of the most common test to predict the behavior of soils under earthquake actions. The test by itself has many implicit limitations and problems, attempts were made to uniform the data produces in different countries under different test conditions, but still remain some uncertainties. Some years ago were proposed some corrections to “standardize” the test, two of the most common accepted is the overburden correction and the sixty percent of energy. For purposes of this work, all the previous N-values published in different catalogs were “standardize” to a 60% of energy and normalized to a $1 \text{ kg/cm}^2 (N_1)_{60}$.

The second test includes in this work is the cone penetration (CPT) which present some advantages over the SPT, specially because provides a continuo soil profile and continuous reading of point and lateral resistance. Two corrections were apply to the data base. One to take into account the effect of overburden pressure and the other to correct the values for different relative densities. The most important problem in CPT data is that the amount of cases available are very small, this conditions force some authors to develop their empirical charts based on SPT modified to CPT values through correlations that are function of the grain size characteristics. This procedure incorporates more uncertainties and it was not considering in the thesis.

Statistical models were used to calculate the discriminant expressions for liquefaction occurrence as a function of earthquake load and soil resistance

parameters. The models for CPT are based on 56 cases, 37 represent liquefaction cases and 19 non-liquefaction cases. The data is a synthesis of two previous catalogs published by Shibata and Teparaksa (1988) and Reyna (1991). Six different models were performed using the discriminant analysis and the same models using logistic regression.

Model number 5 was selected as the best model because gives the minimum number of misclassified cases and also the maximum value of the goodness of fit test (see Table 6).

For the SPT the number of cases was 165 cases, 94 cases of liquefaction and 71 of non-liquefaction. The data is a synthesis of five previous catalogs. Also the same models were performed with SPT data. All the models are based on the Seed and Idriss (1971) parameterization that employs the cyclic stress ratio as a measure of the earthquake load. The best statistical model for SPT cases was the number 5 (see Table 9). Logistic analysis gives better results than discriminant analysis (Fig. 25).

The most important differences of this work with respect to previous in the same area is the development of empirical relation for CPT based exclusively in cases with CPT results avoiding the cases correlate with SPT results, the inclusion of an representative factor of grain size (the mean grain size D_{50}) in the models to count for the effect of the soil gradation, a comparative study of liquefaction based in two statistical methods that are practical tools to predicted the behavior of a soil against liquefaction, and development of expression which are a better discriminant boundary between liquefaction and non-liquefaction.

From the analysis performed it can be seen that both techniques gives similar expressions for certain range (0.0-0.4 cyclic stress ratio) where most of the data cases are located, for the range of 0.4 to 0.6 are less number of cases and results diverge, showing that logistic regression has a better agreement with previous charts. Hence it possible conclude that with the number of cases available the logistic regression gives better results with respect to discriminant analysis. Since the objective of the thesis was not include a probabilistic analysis of occurrence,

discriminant analysis was performed showing a good correlation with the cases specially for CPT data, but if the probability of liquefaction is the major interest the logistic expression can be used with small modifications in the relationship proposed.

Although empirical correlations are a relatively crude approximation of the actual response of cohesionless soils under seismic action, still provide a good source of information, specially for the initial steps in the assessment of liquefaction susceptibility of certain place such the mapping of non-liquefied areas. Some additional effort should be made to include some other important factors in the development of future empirical relations such as the drainage conditions, soil profile and condition of liquefaction. Hence it is necessary more detail field studies of pre-earthquake and post-earthquake behavior of soil.

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TABLE 10. Seed and Idriss (1971) Data Set

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH	CRITICAL	AVERAGE	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER	DEPTH	PENETRAT.	GROUND	STRESS	BEHAVIOR	
					TABLE		RESISTANCE	SURFACE	RATIO	(liquefaction)	
	1	2	3	4	(m)	(m)	CRITICAL	ACCELERAT.	CSR		12
					6	7	DEPTH		τ_{2s}/σ'_o		
101	Niigata	1802	6.6	Niigata	0.9	6.1	6	0.12	0.135	no	Kawasumi
102	Niigata	1802	6.6	Niigata	0.9	6.1	12	0.12	0.135	no	Seed and Idriss
103	Niigata	1887	6.1	Niigata	0.9	6.1	6	0.08	0.090	no	Seed and Idriss
104	Niigata	1887	6.1	Niigata	0.9	6.1	12	0.08	0.090	no	Seed and Idriss
105	Mino Owari	1891	8.4	Ogaki	0.9	13.7	17	0.35	0.390	yes	Kishida
106	Mino Owari	1891	8.4	Ginan West	1.8	9.2	10	0.35	0.370	yes	Kishida
107	Mino Owari	1891	8.4	Unuma	1.8	7.6	19	0.35	0.350	no	Kishida
108	Mino Owari	1891	8.4	Ogase Pond	2.4	6.1	16	0.35	0.350	yes	Kishida
109	Santa Barbara	1925	6.3	Sheffield Dam	4.6	7.6	-	0.20	0.160	yes	Seed et al.
110	El Centro	1940	7.0	Brawley	4.6	4.6	-	0.25	0.155	yes	Ross
111	El Centro	1940	7.0	All-Am. Canal	6.1	7.6	-	0.25	0.155	yes	Ross
112	El Centro	1940	7.0	Solfatara Cana	1.5	6.1	-	0.25	0.260	yes	Ross
113	Tohnankai	1944	8.3	Komei	1.5	4.0	4	0.08	0.080	yes	Kishida
114	Tohnankai	1944	8.3	Meiko St.	0.6	2.4	1	0.08	0.090	yes	Kishida
115	Fukui	1948	7.2	Takaya	3.4	7.0	18	0.30	0.300	yes	Kishida
116	Fukui	1948	7.2	Takaya	0.9	7.0	28	0.30	0.320	no	Kishida
117	Fukui	1948	7.2	Shonenji Temp	1.2	6.1	3	0.30	0.290	yes	Kishida
118	Fukui	1948	7.2	Agr. Union	0.9	6.1	5	0.30	0.330	yes	Kishida
119	San Francisco	1957	5.5	Lake Merced	2.4	3.1	7	0.18	0.130	yes	Ross
120	Chile	1960	8.4	Puerto Montt	3.7	4.6	6	0.15	0.150	yes	Lee
121	Chile	1960	8.4	Puerto Montt	3.7	4.6	8	0.15	0.150	yes	Lee
122	Chile	1960	8.4	Puerto Montt	3.7	6.1	18	0.15	0.150	no	Lee
123	Niigata	1964	7.5	Niigata	0.9	6.1	6	0.16	0.175	yes	Seed and Idriss

TABLE 10. (Continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	AVERAGE PENETRAT. RESISTANCE CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO CSRN τ_{av}/σ'_v	FIELD BEHAVIOR (liquefaction)	REFERENCE
1	2	3	4	5	6 (m)	7 (m)	8 (N)	9 (gals)	10	11	12
124	Niigata	1964	7.5	Niigata	0.9	7.6	15	0.16	0.175	yes	Kishida
125	Niigata	1964	7.5	Niigata	0.9	6.1	12	0.16	0.175	no	Seed and Idriss
126	Niigata	1964	7.5	Niigata	3.7	7.6	6	0.16	0.160	no	Seed and Idriss
127	Alaska	1964	8.3	Snow River	0.0	6.1	5	0.15	0.180	yes	Ross et al.
128	Alaska	1964	8.3	Snow River	2.4	6.1	5	0.15	0.150	yes	Ross et al.
129	Alaska	1964	8.3	Quartz Creek	0.0	7.6	40	0.12	0.145	no	Ross et al.
130	Alaska	1964	8.3	Scott Glacier	6.1	6.1	10	0.16	0.185	yes	Ross et al.
131	Alaska	1964	8.3	Valdez	1.5	6.1	13	0.25	0.250	yes	Coulter-Migliaccio
132	Tokachioki	1968	7.8	Hachinohe	0.9	3.7	14	0.21	0.230	no	Ohsaki
133	Tokachioki	1968	7.8	Hachinohe	0.9	3.7	6	0.21	0.730	yes	Ohsaki
134	Tokachioki	1968	7.8	Hachinohe	1.5	3.1	15	0.21	0.185	no	Ohsaki
135	Tokachioki	1968	7.8	Hakodate	0.9	4.6	6	0.18	0.205	yes	Kishida

TABLE 11. Tokimatsu and Yoshimi (1983) Data Set

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	AVERAGE PENETRAT. RESISTANCE CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(m)	(m)	(N)	(gals)	CSR τ_{av}/σ'_o	(liquefaction)	12
201	Mino-Owari	1891	7.9	Ogaki	0.8	10.0	20.0	0.320	0.364	yes	Kishida, 1969
202	Mino-Owari	1891	7.9	GINAN	2.0	7.0	10.0	0.320	0.317	yes	Kishida, 1969
203	Mino-Owari	1891	7.9	Unuma	1.9	6.0	17.0	0.280	0.275	yes	Kishida, 1969
204	Mino-Owari	1891	7.9	Ogase	2.4	5.0	13.0	0.280	0.246	yes	Kishida, 1969
205	Kanto	1923	7.9	Arakawa 7	4.0	7.0	10.0	0.200	0.163	yes	Kodera, 1964
206	Kanto	1923	7.9	Arakawa 7	4.0	8.0	1.0	0.200	0.168	yes	Kodera, 1964
207	Kanto	1923	7.9	Arakawa 12	4.0	4.3	2.2	0.200	0.130	yes	Kodera, 1964
208	Kanto	1923	7.9	Arakawa 21	1.0	8.0	16.5	0.200	0.225	yes	Kodera, 1964
209	Kanto	1923	7.9	Arakawa 30	1.0	5.0	11.9	0.200	0.187	yes	Kodera, 1964
210	Kanto	1923	7.9	Arakawa 49	3.0	5.0	5.7	0.200	0.162	no	Kodera, 1964
211	Kanto	1923	7.9	Arakawa 49	3.0	8.0	2.0	0.200	0.181	no	Kodera, 1964
212	Tohnankai	1944	8.0	Komei	2.0	5.0	8.0	0.200	0.193	yes	Kishida, 1969
213	Tohnankai	1944	8.0	Meiko	0.5	3.5	1.0	0.200	0.243	yes	Kishida, 1969
214	Tohnankai	1944	8.0	Tenaga	2.5	3.0	2.0	0.200	0.148	yes	Kishida, 1969
215	Tohnankai	1944	8.0	GINAN	2.0	7.0	10.0	0.160	0.161	yes	Kishida, 1969
216	Fukui	1948	7.3	Takaya 2	0.8	4.0	7.0	0.350	0.392	no	Kishida, 1969
217	Fukui	1948	7.3	Takaya 2	0.8	8.0	29.0	0.350	0.388	no	Kishida, 1969
218	Fukui	1948	7.3	Takaya 45	3.7	7.0	19.0	0.350	0.267	yes	Kishida, 1969
219	Fukui	1948	7.3	Shonenji	1.2	4.0	8.0	0.400	0.375	yes	Kishida, 1969
220	Fukui	1948	7.3	Agri Union	0.9	6.0	8.0	0.400	0.450	no	Kishida, 1969
221	Fukui	1948	7.3	Agri Union	0.9	7.5	20.0	0.400	0.451	no	Kishida, 1969
222	Niigata	1964	7.5	Niigata	1.0	7.0	8.0	0.160	0.170	yes	Kishida, 1966
223	Niigata	1964	7.5	Niigata	1.0	7.0	12.0	0.160	0.170	no	Ohsaki, 1966

TABLE 11. (continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	AVERAGE PENETRAT. RESISTANCE CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
1	2	3	4	5	6	7	9	10	11	12	
				(m)	(m)	(N)	(gals)	CSR _N	τ_{av}/σ'_o	(liquefaction)	
224	Niigata	1964	7.5	Niigata	2.0	7.0	18.0	0.160	0.151	no	Koizumi, 1966
225	Niigata	1964	7.5	Niigata	1.0	10.0	10.0	0.160	0.168	yes	Koizumi, 1966
226	Niigata	1964	7.5	Niigata	1.0	10.0	16.0	0.160	0.168	no	Koizumi, 1966
227	Niigata	1964	7.5	Niigata	2.0	10.0	20.0	0.160	0.154	no	Koizumi, 1966
228	Niigata	1964	7.5	Showa Br.2	0.0	4.3	4.0	0.160	0.205	yes	Takada et al. 1965
229	Niigata	1964	7.5	Showa Br.4	1.3	6.0	27.0	0.160	0.161	no	Takada et al. 1965
230	Niigata	1964	7.5	Road Site	2.5	6.0	12.0	0.160	0.128	no	Ishihara et al. 1979
231	Niigata	1964	7.5	River site	0.6	4.5	6.0	0.160	0.177	yes	Ishihara et al. 1979
232	Tokachi-oki	1968	7.9	Nanaehama	1.0	4.0	5.0	0.200	0.124	yes	Kishida, 1970
233	Tokachi-oki	1968	7.9	Hachinohe 2	2.0	6.0	28.0	0.230	0.222	no	Ohsaki, 1970
234	Tokachi-oki	1968	7.9	Hachinohe 3	0.6	4.0	6.0	0.230	0.270	yes	Ohsaki, 1970
235	Tokachi-oki	1968	7.9	Hachinohe 4	1.0	4.0	16.0	0.230	0.246	no	Ohsaki, 1970
236	Miyagiken-oki	1978	6.7	Arahama	1.0	6.3	10.0	0.100	0.092	no	Iwasaki et al. 1981
237	Miyagiken-oki	1978	6.7	Nakamura 4	0.5	3.3	5.0	0.120	0.117	yes	Tsuchida et al. 1979
238	Miyagiken-oki	1978	6.7	Nakamura 5	1.3	3.3	7.0	0.120	0.098	no	Tsuchida et al. 1979
239	Miyagiken-oki	1978	6.7	Yuriageka 1	1.8	5.3	2.0	0.120	0.100	no	Tsuchida et al. 1979
240	Miyagiken-oki	1978	6.7	Yuriageka 2	0.9	4.3	11.0	0.120	0.109	no	Tsuchida et al. 1979
241	Miyagiken-oki	1978	6.7	Yuriagi Br.1	1.7	4.3	4.0	0.120	0.094	no	Tsuchida et al. 1979
242	Miyagiken-oki	1978	6.7	Yuriagi Br.2	1.3	3.3	13.0	0.120	0.098	no	Tsuchida et al. 1979
243	Miyagiken-oki	1978	6.7	Yuriagi Br.3	0.3	4.3	9.0	0.120	0.125	no	Tsuchida et al. 1979
244	Miyagiken-oki	1978	6.7	Oiiri 1	4.3	6.3	9.0	0.140	0.089	no	Tsuchida et al. 1979
245	Miyagiken-oki	1978	6.7	Oiiri 2	2.4	6.3	8.0	0.140	0.111	no	Tsuchida et al. 1979
246	Miyagiken-oki	1978	6.7	Kitawabu 2	3.0	3.3	11.0	0.140	0.080	no	Tsuchida et al. 1979

TABLE 11. (continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	AVERAGE PENETRAT. RESISTANCE CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(m)	(m)	(N)	(gals)	CSR _{av} / σ'_v	(liquefaction)	12
247	Miyagiken-oki	1978	6.7	Shiomi 6	2.5	4.0	6.0	0.140	0.093	no	Tsuchida et al. 1979
248	Miyagiken-oki	1978	6.7	Hiyori 18	2.5	5.0	9.0	0.140	0.100	no	Tsuchida et al. 1979
249	Miyagiken-oki	1978	6.7	Nakajima 18	2.5	6.0	12.0	0.140	0.105	no	Tsuchida et al. 1979
250	Miyagiken-oki	1978	6.7	Ishinomak 12	1.4	4.0	4.0	0.120	0.098	no	Tsuchida et al. 1979
251	Miyagiken-oki	1978	7.4	Arahama	1.0	6.3	10.0	0.200	0.206	yes	Tsuchida et al. 1979
252	Miyagiken-oki	1978	7.4	Nakamura 1	0.9	3.3	19.0	0.320	0.314	no	Tsuchida et al. 1979
253	Miyagiken-oki	1978	7.4	Nakamura 4	0.5	3.3	5.0	0.320	0.350	yes	Tsuchida et al. 1979
254	Miyagiken-oki	1978	7.4	Nakamura 5	1.3	3.3	7.0	0.320	0.292	yes	Tsuchida et al. 1979
255	Miyagiken-oki	1978	7.4	Yuriageka 1	1.8	5.3	2.0	0.240	0.224	yes	Tsuchida et al. 1979
256	Miyagiken-oki	1978	7.4	Yuriageka 2	0.9	4.3	11.0	0.240	0.245	yes	Tsuchida et al. 1979
257	Miyagiken-oki	1978	7.4	Yuriageka 3	2.2	5.3	20.0	0.240	0.206	no	Tsuchida et al. 1979
258	Miyagiken-oki	1978	7.4	Yuriage Br 1	1.7	4.3	4.0	0.240	0.210	yes	Tsuchida et al. 1979
259	Miyagiken-oki	1978	7.4	Yuriage Br 2	1.3	3.3	13.0	0.240	0.219	yes	Tsuchida et al. 1979
260	Miyagiken-oki	1978	7.4	Yuriage Br 3	0.3	4.3	8.0	0.240	0.281	yes	Tsuchida et al. 1979
261	Miyagiken-oki	1978	7.4	Yuriage Br 5	1.3	7.3	17.0	0.240	0.241	no	Tsuchida et al. 1979
262	Miyagiken-oki	1978	7.4	Oiiri 1	4.3	6.3	9.0	0.240	0.172	yes	Tsuchida et al. 1979
263	Miyagiken-oki	1978	7.4	Oiiri 2	2.4	6.3	8.0	0.240	0.214	yes	Tsuchida et al. 1979
264	Miyagiken-oki	1978	7.4	Kitawabu 2	3.0	3.3	11.0	0.280	0.179	yes	Tsuchida et al. 1979
265	Miyagiken-oki	1978	7.4	Kitawabu 3	3.0	6.0	23.0	0.280	0.223	no	Tsuchida et al. 1979
266	Miyagiken-oki	1978	7.4	Shiomi 2	2.5	6.0	10.0	0.240	0.202	no	Tsuchida et al. 1979
267	Miyagiken-oki	1978	7.4	Shiomi 6	2.5	4.0	6.0	0.240	0.180	yes	Tsuchida et al. 1979
268	Miyagiken-oki	1978	7.4	Hiyori 5	2.5	7.0	21.0	0.240	0.208	no	Tsuchida et al. 1979

TABLE 11. (continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	AVERAGE PENETRAT. RESISTANCE CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
1	2	3	4	5	6	7	9	10	11	12	
				(m)	(m)	(N)	(gals)	CSR τ_{av}/σ'_o	(liquefaction)		
269	Miyagiken-oki	1978	7.4	Hiyori 18	2.5	5.0	9.0	0.240	0.193	yes	Tsuchida et al. 1979
270	Miyagiken-oki	1978	7.4	Nakajima 2	2.5	4.5	10.0	0.240	0.187	no	Tsuchida et al. 1979
271	Miyagiken-oki	1978	7.4	Nakajima 18	2.5	6.0	12.0	0.240	0.202	yes	Tsuchida et al. 1979
272	Miyagiken-oki	1978	7.4	Sendaikou 1	2.4	6.0	15.0	0.240	0.209	no	Tsuchida et al. 1979
273	Miyagiken-oki	1978	7.4	Sendaikou 4	3.6	7.0	17.0	0.240	0.188	no	Tsuchida et al. 1979
274	Miyagiken-oki	1978	7.4	Ishinomaki 2	1.4	4.0	4.0	0.200	0.183	yes	Ishihara et al. 1980
275	Miyagiken-oki	1978	7.4	Ishinomaki 4	1.4	6.0	15.0	0.200	0.195	no	Ishihara et al. 1980
276	Izu	1978	7.0	Mochikoshi	1.0	7.0	5.0	0.250	0.244	yes	Okusa et al., 1980
277	Chibakenchubu	1980	6.1	Oh	1.0	6.0	4.0	0.100	0.086	no	Ishihara et al., 1981
278	Chibakenchubu	1980	6.1	Oh	1.0	14.0	6.0	0.100	0.094	no	Ishihara et al., 1981
279	Long Beach	1933	6.3	L.A.Harbor	2.1			0.200	0.163	no	Pyke et al., 1978
280	San Francisco	1957	5.5	Lake Merced	2.4	3.0	6.0	0.190	0.100	yes	Ross, 1968
281	Alaska	1964	8.3	Snow River	0.0	6.0	5.0	0.150	0.210	yes	Ross, 1968
282	Alaska	1964	8.3	Snow River	2.4	6.0	5.0	0.150	0.156	yes	Ross, 1968
283	Alaska	1964	8.3	Scott Glac.	0.0	6.0	10.0	0.160	0.224	yes	Ross, 1968
284	Alaska	1964	8.3	Valdez	1.5	6.0	13.0	0.250	0.290	yes	Ross, 1968
285	San Fernando	1971	6.6	Van Norman	3.0	6.0	9.0	0.450	0.287	yes	Lee et al., 1975
286	San Fernando	1971	6.6	Juvenile	4.5	6.0	2.0	0.400	0.237	yes	Seed, 1976
287	San Fernando	1971	6.6	Jensen Pl.	16.5	16.5	24.0	0.450	0.190	yes	Dixon et al., 1973
288	Guatemala	1976	7.5	Amaticlan 1	1.5	4.6	8.0	0.135	0.169	yes	Seed et al., 1981
289	Guatemala	1976	7.5	Amaticlan 2	2.0	4.6	8.0	0.135	0.135	no	Seed et al., 1981
290	Guatemala	1976	7.5	Amaticlan 4	3.3	7.0	14.0	0.135	0.126	no	Seed et al., 1981
291	Tanshan	1976	7.5	Lutai 51	0.5	11.0		0.200	0.192	yes	Zhou, 1981

TABLE 11. (continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	AVERAGE PENETRAT. RESISTANCE CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO CSRN	FIELD BEHAVIOR (liquefaction)	REFERENCE
	1	2	3	4	5 (m)	6 (m)	7 (N)	9 (gals)	10 τ_{av}/G'_o	11	12
292	Tanshan	1976	7.5	Lutai 52	0.5	11.0		0.200	0.192	no	Zhou, 1981
293	Imperial Valley	1979	6.6	Heber Rd.1	1.8	4.0	31.0	0.600	0.449	no	Benett et al., 1981
294	Imperial Valley	1979	6.6	Heber Rd.4	1.8	4.0	4.0	0.600	0.449	yes	Benett et al., 1981
295	Imperial Valley	1979	6.6	Heber Rd.7	1.8	4.0	11.0	0.600	0.449	no	Benett et al., 1981
296	Imperial Valley	1979	6.6	River Park 6	0.2	2.0	3.0	0.200	0.206	yes	Benett et al., 1981
297	Imperial Valley	1979	6.6	River Park 6	0.2	5.0	7.0	0.200	0.209	yes	Benett et al., 1981

TABLE 12. Shibata and Teparaksa (1988) Data Set

CASE N	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST.	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(m) 5	(m) 6	7	8	9	10	11
301	Niigata	1964	7.5	Kawagishicho	1.10	2.8	51.2	0.16	0.150	yes	Ito et al., 1984
302	Niigata	1964	7.5	Kawagishicho	1.10	4.6	22.2	0.16	0.160	yes	Ito et al., 1984
303	Niigata	1964	7.5	Kawagishicho	1.10	5.2	95.8	0.16	0.170	yes	Ito et al., 1984
304	Niigata	1964	7.5	Kawagishicho	1.10	8.0	62.2	0.16	0.170	yes	Ito et al., 1984
305	Niigata	1964	7.5	Kawagishicho	2.00	4.8	69.6	0.16	0.140	yes	Ishihara and Koga, 1981
306	Niigata	1964	7.5	Kawagishicho	2.00	6.7	96.1	0.16	0.150	yes	Ishihara and Koga, 1981
307	Niigata	1964	7.5	Kawagishicho	2.00	11.1	86.8	0.16	0.150	yes	Ishihara and Koga, 1981
308	Niigata	1964	7.5	South Bank	0.50	4.5	117.7	0.16	0.190	no	Ishihara and Koga, 1981
309	Niigata	1964	7.5	South Bank	0.50	5.0	206.1	0.16	0.180	no	Ishihara and Koga, 1981
310	Nihonkaichubu	1983	7.7	Noshirocho	2.00	3.1	144.7	0.16	0.180	no	Sasaki et al., 1984
311	Nihonkaichubu	1983	7.7	Noshirocho	2.00	3.8	218.6	0.16	0.200	no	Sasaki et al., 1984
312	Nihonkaichubu	1983	7.7	Noshirocho	2.00	5.0	193.2	0.16	0.210	no	Sasaki et al., 1984
313	Nihonkaichubu	1983	7.7	Noshirocho	2.10	2.8	26.1	0.16	0.170	yes	Sasaki et al., 1984
314	Nihonkaichubu	1983	7.7	Noshirocho	2.10	3.4	57.4	0.16	0.190	yes	Sasaki et al., 1984
315	Nihonkaichubu	1983	7.7	Noshirocho	2.10	5.1	99.0	0.16	0.210	yes	Sasaki et al., 1984
316	Nihonkaichubu	1983	7.7	Noshirocho	2.10	6.0	105.0	0.16	0.220	yes	Sasaki et al., 1984
317	Tangshan	1976	7.8	T-1	3.70	4.0	46.1	-	-	yes	Zhou and Zhang, 1979
318	Tangshan	1976	7.8	T-1	3.70	4.3	69.9	-	-	yes	Zhou and Zhang, 1979
319	Tangshan	1976	7.8	T-1	3.70	5.0	27.6	-	-	yes	Zhou and Zhang, 1979
320	Tangshan	1976	7.8	T-2	1.30	3.0	18.2	-	-	yes	Zhou and Zhang, 1979
321	Tangshan	1976	7.8	T-2	1.30	3.9	42.4	-	-	yes	Zhou and Zhang, 1979
322	Tangshan	1976	7.8	T-3	1.50	2.4	270.1	-	-	no	Zhou and Zhang, 1979
323	Tangshan	1976	7.8	T-3	1.50	3.4	300.4	-	-	no	Zhou and Zhang, 1979

TABLE 12. (continued)

CASE N	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST. Q_{c1}	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	5	6	7	8	9	10	11
324	Tangshan	1976	7.8	T-4	1.10	3.5	361.1	-	-	no	Zhou and Zhang, 1979
325	Tangshan	1976	7.8	T-5	3.00	7.5	253.0	-	-	no	Zhou and Zhang, 1979
326	Tangshan	1976	7.8	T-6	1.50	5.0	238.0	-	-	yes	Zhou and Zhang, 1979
327	Tangshan	1976	7.8	T-6	1.50	6.5	88.5	-	-	yes	Zhou and Zhang, 1979
328	Tangshan	1976	7.8	T-7	3.00	6.8	118.0	-	-	yes	Zhou and Zhang, 1979
329	Tangshan	1976	7.8	T-8	2.00	4.3	93.0	-	-	yes	Zhou and Zhang, 1979
330	Tangshan	1976	7.8	T-8	2.00	5.9	104.9	-	-	yes	Zhou and Zhang, 1979
331	Tangshan	1976	7.8	T-9	1.10	3.1	202.8	-	-	no	Zhou and Zhang, 1979
332	Tangshan	1976	7.8	T-9	1.10	7.3	208.2	-	-	no	Zhou and Zhang, 1979
333	Tangshan	1976	7.8	T-10	1.45	3.0	25.8	0.40	0.350	yes	Zhou and Zhang, 1979
334	Tangshan	1976	7.8	T-10	1.45	6.0	115.0	0.40	0.410	yes	Zhou and Zhang, 1979
335	Tangshan	1976	7.8	T-10	1.45	7.8	62.5	0.40	0.420	yes	Zhou and Zhang, 1979
336	Tangshan	1976	7.8	T-10	1.45	8.5	79.8	0.40	0.420	yes	Zhou and Zhang, 1979
337	Tangshan	1976	7.8	T-11	0.85	0.9	29.3	0.40	0.270	yes	Zhou and Zhang, 1979
338	Tangshan	1976	7.8	T-11	0.85	1.3	18.7	0.40	0.320	yes	Zhou and Zhang, 1979
339	Tangshan	1976	7.8	T-11	0.85	1.8	89.4	0.40	0.360	yes	Zhou and Zhang, 1979
340	Tangshan	1976	7.8	T-12	1.55	2.0	40.9	0.40	0.290	yes	Zhou and Zhang, 1979
341	Tangshan	1976	7.8	T-12	1.55	3.0	29.1	0.40	0.340	yes	Zhou and Zhang, 1979
342	Tangshan	1976	7.8	T-12	1.55	4.0	44.6	0.40	0.370	yes	Zhou and Zhang, 1979
343	Tangshan	1976	7.8	T-12	1.55	4.7	7.7	0.40	0.390	yes	Zhou and Zhang, 1979
344	Tangshan	1976	7.8	T-12	1.55	6.4	41.3	0.40	0.400	yes	Zhou and Zhang, 1979
345	Tangshan	1976	7.8	T-12	1.55	9.5	83.0	0.40	0.410	yes	Zhou and Zhang, 1979

TABLE 12. (continued)

CASE N	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST.	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
1	2	3	4	5	6	7	8	9	10	11	
					(m)	(m)	Q_{cl} (kg/cm ²)	(gals)	CSR τ_{qv}/σ'_o	(liquefaction)	
346	Tangshan	1976	7.8	T-13	1.05	2.0	29.2	0.40	0.350	yes	Zhou and Zhang, 1979
347	Tangshan	1976	7.8	T-13	1.05	2.1	60.1	0.40	0.360	yes	Zhou and Zhang, 1979
348	Tangshan	1976	7.8	T-13	1.05	2.7	66.4	0.40	0.380	yes	Zhou and Zhang, 1979
349	Tangshan	1976	7.8	T-14	1.25	1.5	96.4	0.40	0.290	yes	Zhou and Zhang, 1979
350	Tangshan	1976	7.8	T-14	1.25	3.0	139.1	0.40	0.370	yes	Zhou and Zhang, 1979
351	Tangshan	1976	7.8	T-15	1.00	1.2	130.8	0.40	0.290	yes	Zhou and Zhang, 1979
352	Tangshan	1976	7.8	T-15	1.00	1.8	20.9	0.40	0.350	yes	Zhou and Zhang, 1979
353	Tangshan	1976	7.8	T-15	1.00	2.5	70.0	0.40	0.380	yes	Zhou and Zhang, 1979
354	Tangshan	1976	7.8	T-16	3.50	4.0	138.3	0.40	0.270	no	Zhou and Zhang, 1979
355	Tangshan	1976	7.8	T-16	3.50	8.4	148.0	0.40	0.340	no	Zhou and Zhang, 1979
356	Tangshan	1976	7.8	T-17	2.80	3.1	153.7	0.20	0.140	no	Zhou and Zhang, 1979
357	Tangshan	1976	7.8	T-17	2.80	4.1	152.6	0.20	0.150	no	Zhou and Zhang, 1979
358	Tangshan	1976	7.8	T-17	2.80	5.2	208.2	0.20	0.170	no	Zhou and Zhang, 1979
359	Tangshan	1976	7.8	T-18	3.60	4.7	19.0	0.20	0.140	yes	Zhou and Zhang, 1979
360	Tangshan	1976	7.8	T-18	3.60	5.2	40.6	0.20	0.150	yes	Zhou and Zhang, 1979
361	Tangshan	1976	7.8	T-19	1.10	1.5	18.4	0.20	0.150	yes	Zhou and Zhang, 1979
362	Tangshan	1976	7.8	T-19	1.10	2.9	79.4	0.20	0.190	yes	Zhou and Zhang, 1979
363	Tangshan	1976	7.8	T-19	1.10	4.0	42.3	0.20	0.210	yes	Zhou and Zhang, 1979
364	Tangshan	1976	7.8	T-19	1.10	5.5	78.6	0.20	0.210	yes	Zhou and Zhang, 1979
365	Tangshan	1976	7.8	T-20	1.10	1.2	244.7	0.20	0.140	no	Zhou and Zhang, 1979
366	Tangshan	1976	7.8	T-20	1.10	1.7	231.3	0.20	0.160	no	Zhou and Zhang, 1979
367	Tangshan	1976	7.8	T-20	1.10	2.1	282.0	0.20	0.180	no	Zhou and Zhang, 1979

TABLE 12. (continued)

CASE N	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST. Q_{cl}	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	5	6	7	8	9	10	11
368	Tangshan	1976	7.8	T-21	3.10	3.1	139.6	0.20	0.130	no	Zhou and Zhang, 1979
369	Tangshan	1976	7.8	T-21	3.10	3.3	118.4	0.20	0.130	no	Zhou and Zhang, 1979
370	Tangshan	1976	7.8	T-21	3.10	4.0	140.1	0.20	0.150	no	Zhou and Zhang, 1979
371	Tangshan	1976	7.8	T-22	0.80	3.7	29.7	0.20	0.220	yes	Zhou and Zhang, 1979
372	Tangshan	1976	7.8	T-22	0.80	4.0	74.6	0.20	0.220	yes	Zhou and Zhang, 1979
373	Tangshan	1976	7.8	T-23	1.35	3.7	32.5	0.20	0.190	yes	Zhou and Zhang, 1979
374	Tangshan	1976	7.8	T-23	1.35	3.9	37.9	0.20	0.190	yes	Zhou and Zhang, 1979
375	Tangshan	1976	7.8	T-24	1.00	2.8	71.2	0.20	0.200	yes	Zhou and Zhang, 1979
376	Tangshan	1976	7.8	T-24	1.00	3.2	46.8	0.20	0.200	yes	Zhou and Zhang, 1979
377	Tangshan	1976	7.8	T-25	0.65	8.2	101.3	0.20	0.230	yes	Zhou and Zhang, 1979
378	Tangshan	1976	7.8	T-26	0.75	5.2	27.2	0.10	0.110	yes	Zhou and Zhang, 1979
379	Tangshan	1976	7.8	T-27	0.65	5.0	15.3	0.20	0.230	yes	Zhou and Zhang, 1979
380	Tangshan	1976	7.8	T-28	0.65	11.0	149.7	0.10	0.110	no	Zhou and Zhang, 1979
381	Tangshan	1976	7.8	T-28	0.65	11.4	61.4	0.10	0.110	no	Zhou and Zhang, 1979
382	Tangshan	1976	7.8	T-29	1.00	4.8	124.4	0.10	0.110	no	Zhou and Zhang, 1979
383	Tangshan	1976	7.8	T-29	1.00	5.3	33.2	0.10	0.110	no	Zhou and Zhang, 1979
384	Tangshan	1976	7.8	T-29	1.00	5.9	210.9	0.10	0.110	no	Zhou and Zhang, 1979
385	Tangshan	1976	7.8	T-30	2.50	4.8	168.2	0.10	0.080	no	Zhou and Zhang, 1979
386	Tangshan	1976	7.8	T-30	2.50	6.0	161.1	0.10	0.090	no	Zhou and Zhang, 1979
387	Tangshan	1976	7.8	T-30	2.50	8.5	187.2	0.10	0.090	no	Zhou and Zhang, 1979
388	Tangshan	1976	7.8	T-31	2.25	2.3	52.5	0.20	0.130	yes	Zhou and Zhang, 1979
389	Tangshan	1976	7.8	T-31	2.25	3.1	38.4	0.20	0.150	yes	Zhou and Zhang, 1979

TABLE 12. (continued)

CASE N	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST.	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
1	2	3	4	5	6	7	8	9	10	11	
					(m)	(m)	Q_{c1} (kg/cm ²)	(gals)	CSRN τ_{av}/σ'_o	(liquefaction)	
390	Tangshan	1976	7.8	T-32	2.30	3.0	46.6	0.20	0.150	yes	Zhou and Zhang, 1979
391	Tangshan	1976	7.8	T-32	2.30	3.2	57.4	0.20	0.150	yes	Zhou and Zhang, 1979
392	Tangshan	1976	7.8	T-32	2.30	3.8	39.4	0.20	0.160	yes	Zhou and Zhang, 1979
393	Tangshan	1976	7.8	T-33	2.30	3.2	41.8	0.20	0.150	yes	Zhou and Zhang, 1979
394	Tangshan	1976	7.8	T-33	2.30	5.0	72.1	0.20	0.180	yes	Zhou and Zhang, 1979
395	Tangshan	1976	7.8	T-33	2.30	5.6	107.0	0.20	0.180	yes	Zhou and Zhang, 1979
396	Tangshan	1976	7.8	T-34	2.50	2.6	27.1	0.20	0.130	yes	Zhou and Zhang, 1979
397	Tangshan	1976	7.8	T-35	2.90	3.9	32.4	0.20	0.150	yes	Zhou and Zhang, 1979
398	Tangshan	1976	7.8	T-35	2.90	4.0	56.7	0.20	0.150	yes	Zhou and Zhang, 1979
399	Tangshan	1976	7.8	T-35	2.90	5.6	48.4	0.20	0.170	yes	Zhou and Zhang, 1979
400	Tangshan	1976	7.8	T-36	2.30	6.0	92.5	0.20	0.180	no	Zhou and Zhang, 1979
401	Tangshan	1976	7.8	L-1	0.40	6.9	112.5	0.20	0.240	no	Zhou and Gou, 1979
402	Tangshan	1976	7.8	L-1	0.40	12.0	42.5	0.20	0.230	no	Zhou and Gou, 1979
403	Tangshan	1976	7.8	L-1	0.40	13.1	51.3	0.20	0.220	no	Zhou and Gou, 1979
404	Tangshan	1976	7.8	L-2	0.21	5.9	33.7	0.20	0.250	yes	Zhou and Gou, 1979
405	Tangshan	1976	7.8	L-2	0.21	6.0	21.2	0.20	0.250	yes	Zhou and Gou, 1979
406	Tangshan	1976	7.8	L-2	0.21	11.2	14.2	0.20	0.230	yes	Zhou and Gou, 1979
407	Tangshan	1976	7.8	L-2	0.21	12.6	20.8	0.20	0.230	yes	Zhou and Gou, 1979
408	Tangshan	1976	7.8	L-2	0.21	12.1	24.4	0.20	0.230	yes	Zhou and Gou, 1979
409	Tangshan	1976	7.8	L-3	0.43	11.2	26.8	0.20	0.230	yes	Zhou and Gou, 1979
410	Tangshan	1976	7.8	L-3	0.43	11.5	17.2	0.20	0.230	yes	Zhou and Gou, 1979
411	Tangshan	1976	7.8	L-4	0.77	11.1	73.0	0.20	0.220	no	Zhou and Gou, 1979

TABLE 12. (continued)

CASE N	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST.	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(m)	(m)	q_{cl} (kg/cm ²)	(gals)	CSR τ_{av}/σ'_o	(liquefaction)	11
412	Vrancea	1977	7.2	Dimbovitza	1.00	4.2	75.5	0.22	0.210	yes	Ishihara and Perlea, 1984
413	Vrancea	1977	7.2	Dimbovitza	1.00	5.0	50.7	0.22	0.220	yes	Ishihara and Perlea, 1984
414	Vrancea	1977	7.2	Dimbovitza	1.00	6.0	39.5	0.22	0.220	yes	Ishihara and Perlea, 1984
415	Vrancea	1977	7.2	Dimbovitza	1.00	7.0	15.7	0.22	0.220	yes	Ishihara and Perlea, 1984
416	Vrancea	1977	7.2	Dimbovitza	1.00	8.0	58.4	0.22	0.220	yes	Ishihara and Perlea, 1984
417	Imperial Valley	1979	6.6	Heber Road (A)	1.80	1.8	439.6	0.80	0.440	no	Youd and Bennett, 1983
418	Imperial Valley	1979	6.6	Heber Road (A)	1.80	3.0	396.1	0.80	0.570	no	Youd and Bennett, 1983
419	Imperial Valley	1979	6.6	Heber Road (A)	1.80	4.0	476.6	0.80	0.640	no	Youd and Bennett, 1983
420	Imperial Valley	1979	6.6	Heber Road(A2)	1.80	1.8	24.6	0.80	0.440	yes	Youd and Bennett, 1983
421	Imperial Valley	1979	6.6	Heber Road(A2)	1.80	4.0	37.6	0.80	0.640	yes	Youd and Bennett, 1983
422	Imperial Valley	1979	6.6	Heber Road(A2)	1.80	6.0	55.3	0.80	0.720	yes	Youd and Bennett, 1983
423	Imperial Valley	1979	6.6	Heber Road (A)	1.80	1.8	56.7	0.80	0.440	no	Youd and Bennett, 1983
424	Imperial Valley	1979	6.6	Heber Road (A)	1.80	3.0	141.1	0.80	0.570	no	Youd and Bennett, 1983
425	Imperial Valley	1979	6.6	Heber Road (A)	1.80	4.0	101.7	0.80	0.640	no	Youd and Bennett, 1983

TABLE 13. Reyna (1991) Data Set

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CYCLIC STRESS RATIO	CONE RESIST.	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(m) 5	(m) 6	CSR τ_{av}/σ'_o 7	q_{c1} (kg/cm ²) 8	(liquefaction) 9	10
401	Niigata	1964	7.5	Kawagishicho	1.10	4.6	0.166	22.45	yes	Shibata and Teparaksa 1988
402	Niigata	1964	7.5	Kawagishicho	2.00	4.8	0.143	69.49	yes	Shibata and Teparaksa 1988
403	Niigata	1964	7.5	South Bank	0.50	4.5	0.186	119.40	no	Shibata and Teparaksa 1988
404	Nihonkaichubu	1983	7.7	Noshirocho	2.00	3.1	0.179	146.10	no	Shibata and Teparaksa 1988
405	Nihonkaichubu	1983	7.7	Noshirocho	2.10	2.8	0.175	26.71	yes	Shibata and Teparaksa 1988
406	Tangshan	1976	7.8	T-10	1.45	3.0	0.355	26.55	yes	Shibata and Teparaksa 1988
407	Tangshan	1976	7.8	T-10	1.45	7.8	0.434	62.57	yes	Shibata and Teparaksa 1988
408	Tangshan	1976	7.8	T-11	0.85	1.3	0.316	22.08	yes	Shibata and Teparaksa 1988
409	Tangshan	1976	7.8	T-12	1.55	3.0	0.346	40.12	yes	Shibata and Teparaksa 1988
410	Tangshan	1976	7.8	T-13	1.05	2.0	0.346	31.95	yes	Shibata and Teparaksa 1988
411	Tangshan	1976	7.8	T-14	1.25	1.5	0.285	107.10	yes	Shibata and Teparaksa 1988
412	Tangshan	1976	7.8	T-14	1.25	3.0	0.372	144.00	yes	Shibata and Teparaksa 1988
413	Tangshan	1976	7.8	T-15	1.00	1.8	0.346	23.42	yes	Shibata and Teparaksa 1988
414	Tangshan	1976	7.8	T-16	3.50	4.0	0.277	137.80	no	Shibata and Teparaksa 1988
415	Tangshan	1976	7.8	T-17	2.80	3.1	0.138	154.00	no	Shibata and Teparaksa 1988
416	Tangshan	1976	7.8	T-17	2.80	5.2	0.169	207.50	no	Shibata and Teparaksa 1988
417	Tangshan	1976	7.8	T-18	3.60	4.7	0.147	18.91	yes	Shibata and Teparaksa 1988
418	Tangshan	1976	7.8	T-19	1.10	1.5	0.154	20.85	no	Shibata and Teparaksa 1988
419	Tangshan	1976	7.8	T-20	1.10	1.7	0.163	259.28	no	Shibata and Teparaksa 1988
420	Tangshan	1976	7.8	T-21	3.10	3.3	0.135	118.20	yes	Shibata and Teparaksa 1988
421	Tangshan	1976	7.8	T-22	0.80	3.7	0.222	30.66	yes	Shibata and Teparaksa 1988
422	Tangshan	1976	7.8	T-23	1.35	3.7	0.193	33.66	yes	Shibata and Teparaksa 1988
423	Tangshan	1976	7.8	T-24	1.00	3.2	0.204	48.61	yes	Shibata and Teparaksa 1988

TABLE 13. (continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CYCLIC STRESS RATIO	CONE RESIST.	FIELD BEHAVIOR	REFERENCE
1	2	3	4	5	6	7	8	9	10	
					(m)	(m)	CSRN τ_{av}/σ'_o	q_{c1} (kg/cm^2)	(liquefaction)	
424	Tangshan	1976	7.8	T-25	0.65	8.2	0.239	101.20	yes	Shibata and Teparaksa 1988
425	Tangshan	1976	7.8	T-26	0.75	5.2	0.115	27.29	yes	Shibata and Teparaksa 1988
426	Tangshan	1976	7.8	T-27	0.65	5.0	0.234	15.44	yes	Shibata and Teparaksa 1988
427	Tangshan	1976	7.8	T-28	0.65	11.4	0.113	62.72	no	Shibata and Teparaksa 1988
428	Tangshan	1976	7.8	T-29	1.00	5.3	0.111	33.22	no	Shibata and Teparaksa 1988
429	Tangshan	1976	7.8	T-30	2.50	6.0	0.092	160.80	no	Shibata and Teparaksa 1988
430	Tangshan	1976	7.8	T-31	2.25	3.1	0.151	38.69	yes	Shibata and Teparaksa 1988
431	Tangshan	1976	7.8	T-32	2.30	3.0	0.149	47.08	yes	Shibata and Teparaksa 1988
432	Tangshan	1976	7.8	T-33	2.30	2.2	0.153	42.10	yes	Shibata and Teparaksa 1988
433	Tangshan	1976	7.8	T-33	2.30	5.0	0.179	71.80	yes	Shibata and Teparaksa 1988
434	Tangshan	1976	7.8	T-34	2.50	2.6	0.134	27.46	yes	Shibata and Teparaksa 1988
435	Tangshan	1976	7.8	T-35	2.00	3.9	0.150	32.26	yes	Shibata and Teparaksa 1988
436	Tangshan	1976	7.8	T-36	2.30	6.0	0.188	92.26	no	Shibata and Teparaksa 1988
437	Tangshan	1976	7.8	L-1	0.40	12.0	0.243	43.51	no	Shibata and Teparaksa 1988
438	Tangshan	1976	7.8	L-2	0.21	11.6	0.249	21.13	yes	Shibata and Teparaksa 1988
439	Tangshan	1976	7.8	L-3	0.43	11.5	0.248	17.50	yes	Shibata and Teparaksa 1988
440	Tangshan	1976	7.8	L-4	0.77	11.1	0.235	74.70	no	Shibata and Teparaksa 1988
441	Vrancea	1977	7.2	Dimbourtza	1.00	7.0	0.226	15.43	yes	Shibata and Teparaksa 1988
442	Imper. Valley	1979	6.6	HR-A1	1.80	4.8	0.464	81.78	no	Shibata and Teparaksa 1988
443	Imper. Valley	1979	6.6	HR-A2	1.80	4.4	0.452	27.77	yes	Shibata and Teparaksa 1988
444	Imper. Valley	1979	6.6	HR-A3	1.80	4.3	0.450	57.37	no	Shibata and Teparaksa 1988
445	Superstition	1987	6.6	Wildlife	1.48	3.5	0.143	28.23	yes	Reyna 1991

TABLE 13. (continued)

CASE N.	EARTHQUAKE	DATE	MAGNITUDE	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	CYCLIC STRESS RATIO	CONE RESIST.	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(m)	(m)	τ_{av}/σ'_{vo}	(kg/cm^2)	(liquefaction)	10
446	Westmorland	1981	5.6	Koorblom	2.40	4.2	0.168	13.23	yes	Reyna 1991
447	Westmorland	1981	5.6	Vail Canal	2.80	4.0	0.141	86.97	no	Reyna 1991
448	Westmorland	1981	5.6	Radio Tower	2.20	5.0	0.103	10.89	yes	Reyna 1991
449	Westmorland	1981	5.6	McKim	1.50	4.8	0.050	41.26	no	Reyna 1991
450	Westmorland	1981	5.6	River	0.00	2.0	0.291	37.76	yes	Reyna 1991
451	Loma Prieta	1989	7.1	TH	2.45	7.5	0.129	59.00	no	Reyna 1991
452	Loma Prieta	1989	7.1	YBC	2.45	6.5	0.144	34.96	yes	Reyna 1991
453	Loma Prieta	1989	7.1	BW	2.45	7.5	0.160	45.49	yes	Reyna 1991
454	Loma Prieta	1989	7.1	P45	1.52	4.5	1.360	25.43	yes	Reyna 1991
455	Loma Prieta	1989	7.1	HP	2.45	7.5	1.650	52.44	no	Reyna 1991
456	Loma Prieta	1989	7.1	HPL	2.45	5.5	1.780	38.63	yes	Reyna 1991
457	Loma Prieta	1989	7.1	P80	2.45	4.5	1.370	64.59	no	Reyna 1991
458	Loma Prieta	1989	7.1	P94	2.45	4.5	1.340	67.47	no	Reyna 1991

TABLE 14. Liao (1986) Data Set

CASE N	EARTHQUAKE	DATE	MAGNT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER	DEPTH	RESISTANCE	GROUND	STRESS	BEHAVIOR	
					TABLE		AT CRITICAL	SURFACE	RATIO		
	1	2	3	4	(m)	(m)	(N)	(gals)	τ_{av}/σ'_v	(liquefaction)	12
					6	7	8	9	10	11	
501	Niigata Sado Island	1802	6.6	Niigata	1.0	7.0	8	0.12	0.12	no	Seed et al. 1975
502	Niigata Sado Island	1802	6.6	Niigata	1.0	7.0	12	0.12	0.12	no	Seed et al. 1975
503	Niigata Koshigun	1877	6.1	Niigata	1.0	7.0	8	0.08	0.07	no	Seed et al. 1975
504	Niigata Koshigun	1877	6.1	Niigata	1.0	7.0	12	0.08	0.07	no	Seed et al. 1975
505	Mino-Owari	1891	7.9	Ogaki	8.0	13.7	17	0.32	0.35	yes	Seed et al. 1975
506	Mino-Owari	1891	7.9	Ginan	2.0	9.1	10	0.32	0.35	yes	Seed et al. 1975
507	Mino-Owari	1891	7.9	Unuma	1.9	6.0	17	0.32	0.33	yes	Seed et al. 1975
508	Mino-Owari	1891	7.9	Ogase Pond	2.4	6.0	13	0.32	0.29	yes	Seed et al. 1975
509	Mino-Owari	1891	7.9	Saya	0.5	9.0	13	0.23	0.31	yes	Davis and Berril 1981
510	Mino-Owari	1891	7.9	Biwajima	1.0	3.0	9	0.24	0.26	yes	Davis and Berril 1981
511	Tokyo	1894	7.5	Tone River	2.5	3.5	4	0.13	0.10	yes	Davis and Berril 1981
512	Tokyo	1894	7.5	Gyona	0.0	4.5	8	0.15	0.21	no	Davis and Berril 1981
513	Tokyo	1894	7.5	kasu	0.5	3.5	8	0.17	0.20	no	Davis and Berril 1981
514	Tokyo	1894	7.5	Kasukabe	1.0	8.0	3	0.24	0.28	yes	Davis and Berril 1981
515	Tokyo	1894	7.5	Ara River	1.0	7.5	4	0.40	0.47	yes	Davis and Berril 1981
516	San Francisco	1906	8.3	Foot of Market-b	2.4	7.6	16	0.31	0.34	yes	Yegian 1976
517	San Francisco	1906	8.3	So. of Market	1.5	4.6	7	0.31	0.34	yes	Yegian 1976
518	San Francisco	1906	8.3	Mission Creek	3.0	6.0	6	0.31	0.28	yes	Yegian 1976
519	San Francisco	1906	8.3	Salinas	0.5	5.5	8.5	0.20	0.28	yes	Yegian 1976
520	San Francisco	1906	8.3	Yerba Buena Cove	1.7	4.8	8	0.31	0.34	yes	Clough and Chameau 1983
521	San Francisco	1906	8.3	Telegraph Hill	1.7	4.8	14	0.31	0.34	no	Clough and Chameau 1983
522	Gono	1909	6.9	Saya	0.5	9.0	13	0.13	0.15	no	Davis and Berril 1981
523	Gono	1909	6.9	Biwajima	1.0	3.0	9	0.12	0.11	no	Davis and Berril 1981
524	Kanto	1923	7.9	Arakawa 7	4.0	8.0	1	0.20	0.18	yes	Tokimatsu and Yoshimi 1983
525	Kanto	1923	7.9	Arakawa 12	4.0	4.3	2	0.20	0.14	yes	Tokimatsu and Yoshimi 1983
526	Kanto	1923	7.9	Arakawa 21	1.0	8.0	16	0.20	0.24	yes	Tokimatsu and Yoshimi 1983

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	PENETRAT. RESISTANCE AT CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
1	2	3	4	5	6	7	8	9	CSRN τ_{av}/σ'_o	(liquefaction)	12
527	Kanto	1923	7.9	Arakawa 30	1.0	5.0	12	0.20	0.23	yes	Tokimatsu and Yoshimi 1983
528	Kanto	1923	7.9	Arakawa 49	3.0	8.0	2	0.20	0.19	no	Tokimatsu and Yoshimi 1983
529	Kanto	1923	7.9	Tone River	2.5	3.5	4	0.12	0.09	no	Tokimatsu and Yoshimi 1983
530	Kanto	1923	7.9	Gyoda	0.0	4.5	8	0.13	0.19	no	Tokimatsu and Yoshimi 1983
531	Kanto	1923	7.9	Kasu	0.5	3.5	8	0.13	0.17	no	Davis and Berril 1981
532	Kanto	1923	7.9	Kasukabe	1.0	8.0	3	0.14	0.18	yes	Davis and Berril 1981
533	Kanto	1923	7.9	Ara River	1.0	7.5	4	0.17	0.21	yes	Davis and Berril 1981
534	Kanto	1923	7.9	Ukita	1.9	2.5	4	0.18	0.14	yes	Davis and Berril 1981
535	Kanto	1923	7.9	Edogawa	1.3	2.2	6	0.17	0.15	yes	Davis and Berril 1981
536	Santa Barbara	1925	8.3	Sheffield Dam	4.6	7.6	3	0.20	0.13	yes	Seed et al. 1975
537	Nishi-Saitama	1931	7	Tone River	2.5	3.5	4	0.25	0.17	yes	Davis and Berril 1981
538	Nishi-Saitama	1931	7	Gyoda	0.0	4.5	8	0.25	0.32	yes	Davis and Berril 1981
539	Nishi-Saitama	1931	7	Kasu	0.5	3.5	8	0.18	0.20	no	Davis and Berril 1981
540	Nishi-Saitama	1931	7	Kasukabe	1.0	8.0	3	0.14	0.15	yes	Davis and Berril 1981
541	Nishi-Saitama	1931	7	Ara River	1.0	7.5	4	0.11	0.12	no	Davis and Berril 1981
542	Long Beach	1933	6.3	LNG Ter./Res.Pt-1	3.0	7.9	10	0.21	0.16	no	Yegian 1976
543	Long Beach	1933	6.3	LNG Ter./Res.Pt-2	5.5	11.0	8	0.21	0.14	no	Yegian 1976
544	Long Beach	1933	6.3	LNG Ter./Res.Pt-3	3.0	7.3	7	0.21	0.16	no	Yegian 1976
545	Long Beach	1933	6.3	LNG Ter./Res.Pt-4	3.0	6.4	13	0.21	0.16	no	Yegian 1976
546	Long Beach	1933	6.3	L.A. Pier A	1.8	8.2	8	0.21	0.17	no	Pyke et al. 1978
547	El Centro	1940	7	Brawley	4.6	4.6	9	0.25	0.14	yes	Seed et al. 1975
548	El Centro	1940	7	All-Am. Canal	6.1	7.5	4	0.25	0.16	yes	Seed et al. 1975
549	El Centro	1940	7	Solfataro Canal	1.6	6.1	1	0.25	0.24	yes	Seed et al. 1975
550	Tonankai	1944	8	Komei	2.0	2.4	1	0.20	0.15	yes	Seed et al. 1975
551	Tonankai	1944	8	Meiko St.	0.5	3.7	1	0.20	0.24	yes	Seed et al. 1975

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER	DEPTH	RESISTANCE	GROUND	STRESS	BEHAVIOR	
					TABLE		AT CRITICAL	SURFACE	RATIO		
					(m)	(m)	(N)	(gals)	τ_{av}/σ'_o	(liquefaction)	
	1	2	3	4	6	7	8	9	10	11	12
552	Tonankai	1944	8	Ienaga	2.6	3.0	2	0.20	0.15	yes	Seed et al. 1975
553	Tonankai	1944	8	Ginan	2.0	7.0	10	0.16	0.17	yes	Seed et al. 1975
554	Fukui	1948	7.3	Shonenji	1.2	3.0	3	0.40	0.36	yes	Yegian 1976
555	San Francisco	1955	5.4	Joaquin Aqueduct	2.4	17.1	22	0.04	0.03	no	Yegian 1976
556	San Francisco	1957	5.3	St. Francis Circle	4.6	6.1	4	0.14	0.07	no	Yegian 1976
557	San Francisco	1957	5.3	Lake Merced	2.4	3.0	5	0.18	0.09	yes	Yegian 1976
558	San Francisco	1957	5.3	Duboce&Sanchez	3.7	4.0	14	0.15	0.07	no	Yegian 1976
559	San Francisco	1957	5.3	Foot of Market-b	2.4	7.6	16	0.10	0.07	no	Yegian 1976
560	San Francisco	1957	5.3	So. of Market	1.5	4.6	24	0.12	0.08	no	Yegian 1976
561	San Francisco	1957	5.3	Mission Creek	1.5	6.1	6	0.14	0.10	no	Yegian 1976
562	San Francisco	1957	5.3	Polk&Golden Gate	4.6	6.1	20	0.10	0.06	no	Yegian 1976
563	San Francisco	1957	5.3	Polk & Market	2.4	4.6	20	0.10	0.06	no	Yegian 1976
564	San Francisco	1957	5.3	Weiden-a	0.9	1.2	4	0.14	0.07	no	Yegian 1976
565	San Francisco	1957	5.3	Weiden-b	0.9	1.2	8	0.14	0.07	no	Yegian 1976
566	San Francisco	1957	5.3	Weiden-d	1.2	4.3	8	0.14	0.10	no	Yegian 1976
567	San Francisco	1957	5.3	Mission & Spear-a	3.1	3.7	11	0.10	0.06	no	Yegian 1976
568	San Francisco	1957	5.3	Mission & Spear-b	3.1	4.0	10	0.10	0.06	no	Yegian 1976
569	San Francisco	1957	5.3	Park&Otis. AL-a	1.8	5.8	12	0.07	0.06	no	Yegian 1976
570	San Francisco	1957	5.3	Park&Otis. AL-b	1.2	5.8	16	0.07	0.06	no	Yegian 1976
571	San Francisco	1957	5.3	Singlenton, Alame	1.8	3.7	10	0.07	0.04	no	Yegian 1976
572	San Francisco	1957	5.3	Treasure Island-a	2.4	7.6	3	0.08	0.05	no	Yegian 1976
573	San Francisco	1957	5.3	Treasure Island-b	2.4	9.1	5	0.08	0.06	no	Yegian 1976
574	San Francisco	1957	5.3	Treasure Island-c	1.8	6.8	7	0.08	0.06	no	Yegian 1976
575	San Francisco	1957	5.3	Treasure Island-d	1.8	6.8	5	0.08	0.06	no	Yegian 1976
576	San Francisco	1957	5.3	Treasure Island-f	1.8	4.6	5	0.08	0.05	no	Yegian 1976

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER TABLE	DEPTH	RESISTANCE AT CRITICAL DEPTH	GROUND SURFACE ACCELERAT.	STRESS RATIO	BEHAVIOR	
					(m)	(m)	(N)	(gals)	CSRN τ_{av}/σ'_o	(liquefaction)	
	1	2	3	4	6	7	8	9	10	11	12
577	San Francisco	1957	5.3	Treasure Island-g	1.8	4.6	8	0.08	0.05	no	Yegian 1976
578	San Francisco	1957	5.3	Treasure Island-h	1.8	4.6	5	0.08	0.05	no	Yegian 1976
579	San Francisco	1957	5.3	Treasure Island-i	1.8	4.6	15	0.08	0.05	no	Yegian 1976
580	San Francisco	1957	5.3	W 5th/Ave D, Al-a	1.8	3.1	3	0.08	0.04	no	Yegian 1976
581	San Francisco	1957	5.3	W 5th/Ave D, Al-b	2.1	2.7	7	0.08	0.04	no	Yegian 1976
582	San Francisco	1957	5.3	Westline Ave, Al.	0.6	1.5	13	0.08	0.05	no	Yegian 1976
583	San Francisco	1957	5.3	Emeryville, Al.	1.2	4.3	7	0.08	0.04	no	Yegian 1976
584	San Francisco	1957	5.3	Westline M.C.-a	1.2	4.6	5	0.08	0.06	no	Yegian 1976
585	San Francisco	1957	5.3	Westline M.C.-b	1.2	3.7	12	0.08	0.06	no	Yegian 1976
586	San Francisco	1957	5.3	Yerba Buena Cove	1.7	4.6	6	0.10	0.07	no	Clough and Chameau 1983
587	San Francisco	1957	5.3	Telegraph Hill	1.7	4.6	14	0.10	0.07	no	Clough and Chameau 1983
588	Chile	1960	8.4	Puerto Montt-a	3.7	4.6	6	0.15	0.12	yes	Seed et al. 1975
589	Chile	1960	8.4	Puerto Montt-b	3.7	4.6	8	0.15	0.12	yes	Seed et al. 1975
590	Chile	1960	8.4	Puerto Montt-c	3.7	6.1	15	0.15	0.13	no	Seed et al. 1975
591	Chile	1960	7.5	Conception	3.7	7.0	10	0.15	0.12	no	Whitman 1971
592	Chile	1960	7.5	Huchipato	3.7	7.9	35	0.15	0.13	no	Whitman 1971
593	Alaska	1964	8.3	Snow River B605A	0.0	6.1	7	0.15	0.22	yes	Seed et al. 1975
594	Alaska	1964	8.3	Snow River B605	2.4	6.1	5	0.15	0.15	yes	Seed et al. 1975
595	Alaska	1964	8.3	Quartz Creek	0.0	7.6	42	0.12	0.16	no	Seed et al. 1975
596	Alaska	1964	8.3	Scott Glacier	0.0	8.1	10	0.16	0.23	yes	Seed et al. 1975
597	Alaska	1964	8.3	Valdez	1.5	8.1	13	0.25	0.29	yes	Seed et al. 1975
598	Niigata	1964	7.5	Niigata	3.7	7.8	8	0.18	0.15	no	Whitman 1971
599	Niigata	1964	7.5	Niigata	1.0	7.0	8	0.18	0.18	yes	Tokimatsu and Yoshimi 1983
600	Niigata	1964	7.5	Niigata	1.0	7.0	12	0.18	0.18	no	Tokimatsu and Yoshimi 1983
601	Niigata	1964	7.5	Niigata	1.8	7.0	18	0.18	0.18	no	Tokimatsu and Yoshimi 1983

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER	DEPTH	RESISTANCE	GROUND	STRESS	BEHAVIOR	
					TABLE		AT CRITICAL	SURFACE	RATIO	(Liquefaction)	
	1	2	3	4	(m)	(m)	(N)	(gals)	CSRN τ_{av}/σ'_o		12
					6	7	8	9	10	11	
602	Niigata	1964	7.5	Niigata	1.0	10.0	10	0.18	0.18	yes	Tokimatsu and Yoshimi 1983
603	Niigata	1964	7.5	Niigata	1.0	10.0	16	0.18	0.18	no	Tokimatsu and Yoshimi 1983
604	Niigata	1964	7.5	Niigata	2.0	10.0	20	0.18	0.18	no	Tokimatsu and Yoshimi 1983
605	Niigata	1964	7.5	Shown Bridge 2	0.0	4.3	4	0.18	0.21	yes	Tokimatsu and Yoshimi 1983
606	Niigata	1964	7.5	Shown Bridge 4	1.3	6.0	27	0.18	0.19	no	Tokimatsu and Yoshimi 1983
607	Niigata	1964	7.5	Road Site	2.5	6.0	12	0.18	0.16	no	Tokimatsu and Yoshimi 1983
608	Niigata	1964	7.5	River Site	0.6	4.5	6	0.18	0.18	yes	Tokimatsu and Yoshimi 1983
609	San Francisco	1965	4.9	Joaquin Aqueduct	2.4	17.1	22	0.05	0.03	no	Yegian 1976
610	Caracus	1967	6.3	Caraballeda	0.9	0.9	3	0.13	0.07	yes	Seed et al. 1975
611	Tokachi-Oki	1968	7.9	Nanahama, Hakad	1.0	4.0	5	0.20	0.22	yes	Yegian and Vitelli 1981
612	Tokachi-Oki	1968	7.9	Hachinohe P-1	1.0	2.9	14	0.23	0.24	no	Whitman 1971
613	Tokachi-Oki	1968	7.9	Hachinohe P-2	2.0	6.0	28	0.23	0.23	no	Whitman 1971
614	Tokachi-Oki	1968	7.9	Hachinohe P-4	1.0	4.0	16	0.23	0.25	no	Whitman 1971
615	Tokachi-Oki	1968	7.9	Hachinohe P-5	1.6	2.5	11	0.23	0.19	no	Whitman 1971
616	Tokachi-Oki	1968	7.9	Hachinohe P-6	1.2	2.0	1	0.25	0.20	yes	Yegian and Vitelli 1981
617	Tokachi-Oki	1968	7.9	Hachinohe Accel.	1.3	2.5	5	0.24	0.22	no	Yegian and Vitelli 1981
618	Saitama	1968	6.1	Saitama 101-2	8.0	10.0	14	0.08	0.04	no	Yegian and Vitelli 1981
619	Saitama	1968	6.1	Saitama 105-2	8.0	10.0	47	0.08	0.04	no	Yegian and Vitelli 1981
620	Saitama	1968	6.1	Saitama 119	2.0	6.3	10	0.08	0.06	no	Yegian and Vitelli 1981
621	Saitama	1968	6.1	Saitama 121	2.0	6.0	4	0.08	0.06	no	Yegian and Vitelli 1981
622	Saitama	1968	6.1	Saitama 130	3.5	6.5	5	0.08	0.05	no	Yegian and Vitelli 1981
623	Saitama	1968	6.1	Saitama 602	3.0	3.8	5	0.07	0.04	no	Yegian and Vitelli 1981
624	Santa Rosa	1969	5.7	Yerba Buena Cove	1.7	4.6	8	0.02	0.01	no	Clough and Chameau 1983
625	Santa Rosa	1969	5.7	Telegraph Hill	1.7	4.6	14	0.02	0.01	no	Clough and Chameau 1983
626	Godiz, Turkey	1970	7.1	Bursa	3.7	7.0	12	0.07	0.05	no	Yegian and Vitelli 1981

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
					WATER TABLE	DEPTH	RESISTANCE AT CRITICAL DEPTH	GROUND SURFACE ACCELERAT.			
	1	2	3	4	(m) 6	(m) 7	(N) 8	(gals) 9	CSRN τ_{av}/σ'_o 10	(liquefaction) 11	12
627	San Fernando	1971	6.6	Juvenile Hall	4.6	6.1	2	0.45	0.28	yes	Seed et al. 1975
628	San Fernando	1971	6.6	Jensen Plant	16.8	16.8	24	0.45	0.18	yes	Seed et al. 1975
629	San Fernando	1971	6.6	Van Norman Dam	3.0	5.0	9	0.45	0.30	yes	Tokimatsu and Yoshimi 1983
630	Yokohama	1972	7.3	Yokohama	3.1	16.0	10	0.01	0.01	no	Yegian 1976
631	Haicheng	1975	7.3	Shuang. Eardo Br.	2.0	6.0	9.5	0.10	0.09	no	Xie 1979
632	Haicheng	1975	7.3	Shenglitang	2.0	13.0	14.5	0.10	0.09	no	Xie 1979
633	Haicheng	1975	7.3	Ligobe Plant	5.0	5.2	5.5	0.10	0.09	yes	Xie 1979
634	Haicheng	1975	7.3	Panjin Storage	1.5	7.0	6	0.13	0.14	yes	Xie 1979
635	Haicheng	1975	7.3	Yinkou Paper Plan	1.5	8.2	11	0.20	0.20	yes	Xie 1979
636	Haicheng	1975	7.3	Nanbeyan Irr. Sta.	2.0	3.0	6	0.13	0.10	yes	Xie 1979
637	Haicheng	1975	7.3	Shuiyuan Commu	2.0	10.0	9	0.20	0.19	yes	Xie 1979
638	Haicheng	1975	7.3	Yinlou Gate	2.0	10.3	9	0.20	0.21	yes	Xie 1979
639	Haicheng	1975	7.3	Panjin Ch. Fertil.	1.5	9.1	8	0.13	0.13	yes	Seed et al. 1984
640	Haicheng	1975	7.3	Yinkou Glass Fibe	1.5	8.2	13	0.20	0.20	yes	Seed et al. 1984
641	Haicheng	1975	7.3	Shuang Tai Zi R.	1.5	8.2	9	0.10	0.10	no	Seed et al. 1984
642	Guatemala	1976	7.5	Amatitlan 1	1.5	8.8	3	0.13	0.16	yes	Tokimatsu and Yoshimi 1983
643	Guatemala	1976	7.5	Amatitlan 2	2.8	5.8	7	0.13	0.15	no	Tokimatsu and Yoshimi 1983
644	Guatemala	1976	7.5	Amatitlan 3	4.1	16.8	12	0.13	0.14	no	Tokimatsu and Yoshimi 1983
645	Guatemala	1976	7.5	Amatitlan 4	3.4	17.9	11	0.13	0.15	no	Tokimatsu and Yoshimi 1983
646	Tangaham	1976	7.8	Weigezhuang	1.4	2.3	11	0.20	0.17	yes	Xie 1979
647	Tangaham	1976	7.8	Lujiao Mine	1.0	7.0	4	0.35	0.41	yes	Xie 1979
648	Tangaham	1976	7.8	Tangshan City	3.0	5.3	30	0.60	0.41	no	Tokimatsu and Yoshimi 1983
649	Tangaham	1976	7.8	Qing Yin	0.9	5.3	17	0.35	0.38	yes	Tokimatsu and Yoshimi 1983
650	Tangaham	1976	7.8	Le Ting	1.5	2.0	10	0.20	0.15	yes	Tokimatsu and Yoshimi 1983
651	Tangaham	1976	7.8	Coastal Region	1.2	6.1	10	0.13	0.14	yes	Tokimatsu and Yoshimi 1983

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER TABLE	DEPTH	RESISTANCE AT CRITICAL DEPTH	GROUND SURFACE ACCELERAT.	STRESS RATIO	BEHAVIOR	
	1	2	3	4	(m) 6	(m) 7	(N) 8	(gals) 9	CSRN τ_{av}/σ'_o 10	(Liquefaction) 11	12
652	Tangaham	1976	7.8	Yao Yuan Village	0.0	4.2	6	0.20	0.21	yes	Tokimatsu and Yoshimi 1983
653	Tangaham	1976	7.8	Ma Feng	3.9	10.2	3	0.07	0.06	no	Tokimatsu and Yoshimi 1983
654	Tangaham	1976	7.8	Wang Zhuang	1.5	6.2	2	0.20	0.20	yes	Tokimatsu and Yoshimi 1983
655	San Juan Argentina	1977	7.4	Barrio Castro B-1	4.6	9.9	22	0.20	0.17	yes	Idriss et al. 1979
656	San Juan Argentina	1977	7.4	Barrio Castro B-2	4.7	11.0	8	0.20	0.17	yes	Idriss et al. 1979
657	San Juan Argentina	1977	7.4	Cauceta B-3	6.9	6.9	4	0.20	0.12	yes	Idriss et al. 1979
658	San Juan Argentina	1977	7.4	West of River B-4	1.2	8.1	7	0.20	0.23	no	Idriss et al. 1979
659	San Juan Argentina	1977	7.4	West of River B-5	2.1	3.2	18	0.20	0.15	no	Idriss et al. 1979
660	San Juan Argentina	1977	7.4	Fin. Santiago B-6	1.8	5.2	6	0.20	0.19	yes	Idriss et al. 1979
661	San Juan Argentina	1977	7.4	Escuela Normal 1	3.2	11.0	8	0.20	0.19	yes	Youd 1984
662	San Juan Argentina	1977	7.4	Escuela Normal 2	3.2	9.0	12	0.20	0.19	yes	Youd 1984
663	San Juan Argentina	1977	7.4	Escuela Normal 3	3.1	9.0	14	0.20	0.19	yes	Youd 1984
664	San Juan Argentina	1977	7.4	Escuela Normal 4	3.1	8.0	12	0.20	0.18	yes	Youd 1984
665	San Juan Argentina	1977	7.4	Escuela Normal 5	3.2	10.0	8	0.20	0.19	yes	Youd 1984
666	San Juan Argentina	1977	7.4	Escuela Normal 6	3.4	8.0	10	0.20	0.18	yes	Youd 1984
667	San Juan Argentina	1977	7.4	Airport (Rt 20) 1	2.7	9.0	17	0.20	0.19	no	Youd 1984
668	San Juan Argentina	1977	7.4	Airport (Rt 20) 2	2.6	12.0	10	0.20	0.19	no	Youd 1984
669	San Juan Argentina	1977	7.4	Santa Rosa 1	1.6	11.0	12	0.20	0.21	no	Youd 1984
670	San Juan Argentina	1977	7.4	Santa Rosa 2	1.6	9.0	6	0.20	0.20	no	Youd 1984
671	San Juan Argentina	1977	7.4	Santa Rosa 3	1.6	7.0	13	0.20	0.21	no	Youd 1984
672	Izu	1978	7	Mochikoshi	1.0	7.0	1	0.28	0.26	yes	Tokimatsu and Yoshimi 1983
673	Miyagiken-Oki-1	1978	6.7	Arahama	1.0	6.3	10	0.10	0.10	no	Tokimatsu and Yoshimi 1983
674	Miyagiken-Oki-1	1978	6.7	Nakamura 1	0.9	3.3	19	0.12	0.11	no	Tokimatsu and Yoshimi 1983
675	Miyagiken-Oki-1	1978	6.7	Nakamura 4	0.5	3.3	5	0.12	0.12	yes	Tokimatsu and Yoshimi 1983
676	Miyagiken-Oki-1	1978	6.7	Nakamura 5	1.3	3.3	7	0.12	0.10	no	Tokimatsu and Yoshimi 1983

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE	
					WATER	DEPTH	RESISTANCE	GROUND	STRESS	BEHAVIOR		
					TABLE		AT CRITICAL	SURFACE	RATIO	(liquefaction)		
	1	2	3	4	(m)	(m)	(N)	(gals)	CSR _N		12	
					6	7	8	9	τ_{av}/σ'_o	10	11	
677	Miyagiken-Oki-1	1978	6.7	Yoriageka 1	1.8	5.3	2	0.12	0.10	no	Tokimatsu and Yoshimi 1983	
678	Miyagiken-Oki-1	1978	6.7	Yoriageka 2	0.9	4.3	11	0.12	0.11	no	Tokimatsu and Yoshimi 1983	
679	Miyagiken-Oki-1	1978	6.7	Yoriageka 3	2.2	5.3	20	0.12	0.10	no	Tokimatsu and Yoshimi 1983	
680	Miyagiken-Oki-1	1978	6.7	Yuriage Bridge 1	1.7	4.3	4	0.12	0.09	no	Tokimatsu and Yoshimi 1983	
681	Miyagiken-Oki-1	1978	6.7	Yuriage Bridge 2	1.3	3.3	13	0.12	0.10	no	Tokimatsu and Yoshimi 1983	
682	Miyagiken-Oki-1	1978	6.7	Yuriage Bridge 3	0.3	4.3	8	0.12	0.13	no	Tokimatsu and Yoshimi 1983	
683	Miyagiken-Oki-1	1978	6.7	Yuriage Bridge 5	1.3	7.3	17	0.12	0.11	no	Tokimatsu and Yoshimi 1983	
684	Miyagiken-Oki-1	1978	6.7	Oiira 1	4.3	6.3	9	0.14	0.09	no	Tokimatsu and Yoshimi 1983	
685	Miyagiken-Oki-1	1978	6.7	Oiira 2	2.4	6.3	8	0.14	0.12	no	Tokimatsu and Yoshimi 1983	
686	Miyagiken-Oki-1	1978	6.7	Kitawabu 2	3.0	3.3	11	0.14	0.08	no	Whitman 1971	
687	Miyagiken-Oki-1	1978	6.7	Kitawabu 3	3.0	6.0	23	0.14	0.10	no	Whitman 1971	
688	Miyagiken-Oki-1	1978	6.7	Shiomi 2	2.5	6.0	10	0.14	0.11	no	Tokimatsu and Yoshimi 1983	
689	Miyagiken-Oki-1	1978	6.7	Shiomi 6	2.5	4.0	6	0.14	0.10	no	Tokimatsu and Yoshimi 1983	
690	Miyagiken-Oki-1	1978	6.7	Hiyori 5	2.5	7.0	21	0.14	0.11	no	Tokimatsu and Yoshimi 1983	
691	Miyagiken-Oki-1	1978	6.7	Hiyori 18	2.5	5.0	9	0.14	0.11	no	Tokimatsu and Yoshimi 1983	
692	Miyagiken-Oki-1	1978	6.7	Nakajima 2	2.5	4.5	10	0.14	0.10	no	Tokimatsu and Yoshimi 1983	
693	Miyagiken-Oki-1	1978	6.7	Nakajima 18	2.5	6.0	12	0.14	0.11	no	Tokimatsu and Yoshimi 1983	
694	Miyagiken-Oki-1	1978	6.7	Sondaikou 1	2.4	6.0	15	0.12	0.10	no	Tokimatsu and Yoshimi 1983	
695	Miyagiken-Oki-1	1978	6.7	Sondaikou 4	3.6	7.0	17	0.12	0.09	no	Tokimatsu and Yoshimi 1983	
696	Miyagiken-Oki-1	1978	6.7	Ishinomaki 2	1.4	4.0	4	0.12	0.10	no	Tokimatsu and Yoshimi 1983	
697	Miyagiken-Oki-1	1978	6.7	Ishinomaki 4	1.4	6.0	15	0.12	0.11	no	Tokimatsu and Yoshimi 1983	
698	Miyagiken-Oki-2	1978	7.4	Arahama	1.0	6.3	10	0.20	0.22	yes	Tokimatsu and Yoshimi 1983	
699	Miyagiken-Oki-2	1978	7.4	Nakamura 1	0.9	3.3	19	0.32	0.32	no	Tokimatsu and Yoshimi 1983	
700	Miyagiken-Oki-2	1978	7.4	Nakamura 4	0.5	3.3	5	0.32	0.34	yes	Tokimatsu and Yoshimi 1983	
701	Miyagiken-Oki-2	1978	7.4	Nakamura 5	1.3	3.3	7	0.32	0.30	yes	Tokimatsu and Yoshimi 1983	

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER TABLE	DEPTH	RESISTANCE AT CRITICAL DEPTH	GROUND SURFACE ACCELERAT.	STRESS RATIO	BEHAVIOR	
	1	2	3	4	(m) 6	(m) 7	(N) 8	(gals) 9	CSRN τ_{av}/σ'_o 10	(liquefaction) 11	12
702	Miyagiken-Oki-2	1978	7.4	Yoriageka 1	1.0	5.3	2	0.24	0.23	yes	Tokimatsu and Yoshimi 1983
703	Miyagiken-Oki-2	1978	7.4	Yoriageka 2	0.9	4.3	11	0.24	0.25	yes	Tokimatsu and Yoshimi 1983
704	Miyagiken-Oki-2	1978	7.4	Yoriageka 3	2.2	5.3	20	0.24	0.21	no	Tokimatsu and Yoshimi 1983
705	Miyagiken-Oki-2	1978	7.4	Yuriage Bridge 1	1.7	4.3	4	0.24	0.21	yes	Tokimatsu and Yoshimi 1983
706	Miyagiken-Oki-2	1978	7.4	Yuriage Bridge 2	1.3	3.3	13	0.24	0.22	yes	Tokimatsu and Yoshimi 1983
707	Miyagiken-Oki-2	1978	7.4	Yuriage Bridge 3	0.3	4.3	8	0.24	0.29	yes	Tokimatsu and Yoshimi 1983
708	Miyagiken-Oki-2	1978	7.4	Yuriage Bridge 5	1.3	7.3	17	0.24	0.26	no	Tokimatsu and Yoshimi 1983
709	Miyagiken-Oki-2	1978	7.4	Oiira 1	4.3	6.3	9	0.24	0.18	yes	Tokimatsu and Yoshimi 1983
710	Miyagiken-Oki-2	1978	7.4	Oiira 2	2.4	6.3	8	0.24	0.22	yes	Tokimatsu and Yoshimi 1983
711	Miyagiken-Oki-2	1978	7.4	Kitawabu 2	3.0	3.3	11	0.28	0.18	yes	Tokimatsu and Yoshimi 1983
712	Miyagiken-Oki-2	1978	7.4	Kitawabu 3	3.0	6.0	23	0.28	0.23	no	Tokimatsu and Yoshimi 1983
713	Miyagiken-Oki-2	1978	7.4	Shiomi 2	2.5	6.0	10	0.24	0.21	no	Tokimatsu and Yoshimi 1983
714	Miyagiken-Oki-2	1978	7.4	Shiomi 6	2.5	4.0	6	0.24	0.19	yes	Tokimatsu and Yoshimi 1983
715	Miyagiken-Oki-2	1978	7.4	Iiyori 5	2.5	7.0	21	0.24	0.22	no	Tokimatsu and Yoshimi 1983
716	Miyagiken-Oki-2	1978	7.4	Iiyori 18	2.5	5.0	9	0.24	0.20	yes	Tokimatsu and Yoshimi 1983
717	Miyagiken-Oki-2	1978	7.4	Nakajima 2	2.5	4.5	10	0.24	0.20	no	Tokimatsu and Yoshimi 1983
718	Miyagiken-Oki-2	1978	7.4	Nakajima 18	2.5	6.0	12	0.24	0.21	yes	Tokimatsu and Yoshimi 1983
719	Miyagiken-Oki-2	1978	7.4	Sondaikou 1	2.4	6.0	15	0.24	0.21	no	Tokimatsu and Yoshimi 1983
720	Miyagiken-Oki-2	1978	7.4	Sondaikou 4	3.6	7.0	17	0.24	0.20	no	Tokimatsu and Yoshimi 1983
721	Miyagiken-Oki-2	1978	7.4	Ishinomaki 2	1.4	4.0	4	0.20	0.18	yes	Tokimatsu and Yoshimi 1983
722	Miyagiken-Oki-2	1978	7.4	Ishinomaki 4	1.4	6.0	15	0.20	0.20	no	Tokimatsu and Yoshimi 1983
723	Thessaloniki	1978	8.5	Greek Church	4.0	5.0	27	0.32	0.19	no	Gazetas and Botsis 1981
724	Thessaloniki	1978	8.5	White Tower	1.5	8.7	7	0.32	0.30	no	Gazetas and Botsis 1981
725	Guerrero	1979	7.6	Enmedio Zone 1	3.5	5.5	20	0.30	0.24	no	Jaime et al. 1981
726	Guerrero	1979	7.6	Enmedio Zone 2	1.2	2.7	16	0.30	0.27	yes	Jaime et al. 1981

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER	DEPTH	RESISTANCE	GROUND	STRESS	BEHAVIOR	
					TABLE		AT CRITICAL	SURFACE	RATIO	(liquefaction)	
	1	2	3	4	(m)	(m)	(N)	(gals)	CSRN		12
					6	7	8	9	τ_{av}/σ'_o	10	11
727	Montenegro	1979	6.9	Boca Kotorska	1.0	7.0	6	0.25	0.28	yes	Talaganov et al. 1980
728	Imperial Valley	1979	6.6	Heber Rd. 1/A1	1.8	3.7	28	0.55	0.40	no	Tokimatsu and Yoshimi 1983
729	Imperial Valley	1979	6.6	Heber Rd. 4,6/A2	1.8	3.7	1	0.55	0.40	yes	Tokimatsu and Yoshimi 1983
730	Imperial Valley	1979	6.6	Heber Rd. 7/A3	1.8	4.3	13	0.55	0.42	no	Tokimatsu and Yoshimi 1983
731	Imperial Valley	1979	6.6	River Park A	0.2	1.8	3	0.24	0.23	yes	Tokimatsu and Yoshimi 1983
732	Imperial Valley	1979	6.6	River Park C	0.2	4.3	11	0.24	0.24	yes	Tokimatsu and Yoshimi 1983
733	Imperial Valley	1979	6.6	Wildlife 1Ns	1.2	3.4	3	0.16	0.14	no	Tokimatsu and Yoshimi 1983
734	Imperial Valley	1979	6.6	Wildlife 2Ng1	1.2	3.4	5	0.16	0.14	no	Tokimatsu and Yoshimi 1983
735	Imperial Valley	1979	6.6	Wildlife 2Ng3	1.2	3.1	6	0.16	0.13	no	Tokimatsu and Yoshimi 1983
736	Imperial Valley	1979	6.6	Wildlife 3Ns	1.2	4.9	4	0.16	0.15	no	Tokimatsu and Yoshimi 1983
737	Imperial Valley	1979	6.6	Wildlife 3Ns	1.2	4.3	9	0.16	0.14	no	Tokimatsu and Yoshimi 1983
738	Imperial Valley	1979	6.6	Wildlife 5Ng	1.2	3.8	5	0.16	0.14	no	Tokimatsu and Yoshimi 1983
739	Imperial Valley	1979	6.6	Vail V2a-b	2.7	4.0	18	0.14	0.09	no	Tokimatsu and Yoshimi 1983
740	Imperial Valley	1979	6.6	Vail TV2a-b	2.7	4.0	13	0.14	0.09	no	Tokimatsu and Yoshimi 1983
741	Imperial Valley	1979	6.6	Kornbloom K3	2.5	4.3	3	0.10	0.09	no	Tokimatsu and Yoshimi 1983
742	Imperial Valley	1979	6.6	Kornbloom TK4a	2.5	3.0	8	0.10	0.08	no	Tokimatsu and Yoshimi 1983
743	Imperial Valley	1979	6.6	Kornbloom TK4b	2.5	3.7	7	0.10	0.07	no	Tokimatsu and Yoshimi 1983
744	Imperial Valley	1979	6.6	Kornbloom SK4-5	2.5	2.9	1	0.10	0.06	no	Tokimatsu and Yoshimi 1983
745	Imperial Valley	1979	6.6	Radio Tower R2	2.1	3.4	2	0.15	0.10	yes	Tokimatsu and Yoshimi 1983
746	Imperial Valley	1979	6.6	Radio Tower R3	2.1	2.3	11	0.15	0.09	no	Tokimatsu and Yoshimi 1983
747	Imperial Valley	1979	6.6	McKim TM6-7	1.5	2.3	7	0.50	0.34	yes	Tokimatsu and Yoshimi 1983
748	Imperial Valley	1979	6.6	McKim SM7	1.5	2.3	3	0.50	0.34	yes	Tokimatsu and Yoshimi 1983
749	Imperial Valley	1979	6.6	SNorthend SN2a	4.1	6.0	15	0.16	0.10	no	Tokimatsu and Yoshimi 1983
750	Imperial Valley	1979	6.6	SNorthend SN2b	4.1	4.6	16	0.16	0.09	no	Tokimatsu and Yoshimi 1983
751	Imperial Valley	1979	6.6	Young Y5	2.7	4.3	9	0.10	0.07	no	Tokimatsu and Yoshimi 1983

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DEPTH	CRITICAL	PENETRAT.	MAXIMUM	CYCLIC	FIELD	REFERENCE
					WATER TABLE	DEPTH	RESISTANCE AT CRITICAL DEPTH	GROUND SURFACE ACCELERAT.	STRESS RATIO	BEHAVIOR	
1	2	3	4	(m)	(m)	(N)	(gals)	τ_{mv}/σ'_v	CSRN	(liquefaction)	12
6	7	8	9	10	11	12	13	14	15	16	17
752	Mexicali Val.	1980	6.7	Town of Delta	2.0	5.0	5	0.58	0.52	yes	Seed et al. 1984
753	Mid-Chiba	1980	6.1	Owi Island 1	1.0	6.0	5	0.10	0.09	no	Tokimatsu and Yoshimi 1983
754	Mid-Chiba	1980	6.1	Owi Island 1	1.0	14.3	4	0.10	0.08	no	Tokimatsu and Yoshimi 1983
755	Westmoreland	1981	5.6	Heber Rd. 1/A1	1.8	3.7	28	0.02	0.01	no	Seed et al. 1984
756	Westmoreland	1981	5.6	Heber Rd. 4,6/A2	1.8	3.7	1	0.02	0.01	no	Seed et al. 1984
757	Westmoreland	1981	5.6	Heber Rd. 7/A3	1.8	4.3	13	0.02	0.01	no	Seed et al. 1984
758	Westmoreland	1981	5.6	River Park A	0.2	1.8	3	0.18	0.14	no	Seed et al. 1984
759	Westmoreland	1981	5.6	River Park C	0.2	4.3	11	0.18	0.15	no	Seed et al. 1984
760	Westmoreland	1981	5.6	Wildlife 1Ns	1.2	3.4	3	0.26	0.18	yes	Seed et al. 1984
761	Westmoreland	1981	5.6	Wildlife 2Ng1	1.2	3.4	5	0.26	0.18	yes	Seed et al. 1984
762	Westmoreland	1981	5.6	Wildlife 2Ng3	1.2	3.1	6	0.26	0.18	yes	Seed et al. 1984
763	Westmoreland	1981	5.6	Wildlife 3Ns	1.2	4.9	4	0.26	0.20	yes	Seed et al. 1984
764	Westmoreland	1981	5.6	Wildlife 3Ns	1.2	4.3	9	0.26	0.19	yes	Seed et al. 1984
765	Westmoreland	1981	5.6	Wildlife 5Ng	1.2	3.8	5	0.26	0.19	yes	Seed et al. 1984
766	Westmoreland	1981	5.6	Vail V2a-b	2.7	4.0	15	0.26	0.14	no	Seed et al. 1984
767	Westmoreland	1981	5.6	Vail TV2a-b	2.7	4.0	13	0.26	0.14	no	Seed et al. 1984
768	Westmoreland	1981	5.6	Kornbloom K3	2.5	4.3	3	0.28	0.16	yes	Seed et al. 1984
769	Westmoreland	1981	5.6	Kornbloom TK4a	2.5	3.0	3	0.28	0.14	yes	Seed et al. 1984
770	Westmoreland	1981	5.6	Kornbloom TK4b	2.5	3.7	7	0.28	0.15	yes	Seed et al. 1984
771	Westmoreland	1981	5.6	Kornbloom SK4-5	2.5	2.9	1	0.28	0.14	yes	Seed et al. 1984
772	Westmoreland	1981	5.6	Radio Tower R2	2.1	3.4	2	0.18	0.10	yes	Seed et al. 1984
773	Westmoreland	1981	5.6	Radio Tower R3	2.1	2.3	11	0.18	0.09	no	Seed et al. 1984
774	Westmoreland	1981	5.6	McKim TM6-7	1.5	2.3	7	0.12	0.07	no	Seed et al. 1984
775	Westmoreland	1981	5.6	McKim SM7	1.5	2.3	3	0.12	0.07	no	Seed et al. 1984
776	Westmoreland	1981	5.6	SNorthend SN2a	4.1	6.0	15	0.26	0.13	no	Seed et al. 1984

TABLE 14. (continued)

CASE N	EARTHQUAKE	DATE	MAGNT.	SITE	DEPTH WATER TABLE	CRITICAL DEPTH	PENETRAT. RESISTANCE AT CRITICAL DEPTH	MAXIMUM GROUND SURFACE ACCELERAT.	CYCLIC STRESS RATIO CSRN τ_{av}/σ'_o	FIELD BEHAVIOR (liquefaction)	REFERENCE
	1	2	3	4	(m) 6	(m) 7	(N) 8	(gals) 9	10	11	12
777	Westmoreland	1981	5.6	SNorthend SN2b	4.1	4.6	16	0.26	0.12	no	Seed et al. 1984
778	Westmoreland	1981	5.6	Young Y5	2.7	4.3	9	0.28	0.15	no	Seed et al. 1984

TABLE 15. Tokimatsu et al. (1994) Data Set

CASE N	EARTHQUAKE	DATE	MAGNIT.	SITE	DISTANCE FROM SOURCE OF ENERGY RELEASE (km)	DEPTH WATER TABLE (m)	CRITICAL DEPTH (m)	PENETRATION RESISTANCE AT CRITICAL DEPTH $(N_1)_{60}$	CYCLIC STRESS RATIO CSRN τ_{av}/σ'_o	FIELD BEHAVIOR (liquefaction)	REFERENCE
	1	2	3	4	5	6	7	8	9	12	13
801	Luzon	1990	7.8	Perez Boulevard	25	1-2	7.5	6	0.214	yes	Tokimatsu et al. 1994
802	Luzon	1990	7.8	Perez Boulevard	25	1-2	4.6	12	0.195	yes	Tokimatsu et al. 1994
803	Luzon	1990	7.8	Perez Boulevard	25	1-2	10.0	14	0.200	yes	Tokimatsu et al. 1994
804	Luzon	1990	7.8	Perez Boulevard	25	1-2	2.9	20	0.181	yes	Tokimatsu et al. 1994
805	Luzon	1990	7.8	Perez Boulevard	25	1-2	7.8	25	0.200	no	Tokimatsu et al. 1994
806	Luzon	1990	7.8	Perez Boulevard	25	1-2	9.3	31	0.200	no	Tokimatsu et al. 1994
807	Luzon	1990	7.8	Fernandez Ave.	25	1-2	4.3	12	0.190	no	Tokimatsu et al. 1994
808	Luzon	1990	7.8	Fernandez Ave.	25	1-2	5.0	13	0.190	yes	Tokimatsu et al. 1994
809	Luzon	1990	7.8	Fernandez Ave.	25	1-2	10.0	16	0.190	yes	Tokimatsu et al. 1994
810	Luzon	1990	7.8	Fernandez Ave.	25	1-2	10.4	16	0.204	yes	Tokimatsu et al. 1994
811	Luzon	1990	7.8	Fernandez Ave.	25	1-2	10.0	23	0.190	no	Tokimatsu et al. 1994

TABLE 16. Arulanandan et al. (1994) Data Set

CASE N	EARTHQUAKE	DATE	MAGNT.	SITE	DISTANCE FROM SOURCE OF ENERGY RELEASE	DEPTH WATER TABLE	CRITICAL DEPTH	CONE RESIST.	PENET. RESISTANCE AT CRITICAL DEPTH	CYCLIC STRESS RATIO	FIELD BEHAVIOR	REFERENCE
	1	2	3	4	(km) 5	(m) 6	(m) 7	q_{c1} (kg/cm^2) 8	$(N_1)_{60}$ 9	τ_{av}/σ'_o 10	(liquefaction) 11	12
901	Haicheng	1975	7.3	Paper Mill	60	1.00	3.00	26.06	4.20	0.005	yes	Arulanandan et al. 1986
902	Haicheng	1975	7.3	Glass Fibre	60	0.75	5.50	79.60	5.30	0.024	yes	Arulanandan et al. 1986
903	Haicheng	1975	7.3	Construction Building	60	1.50	7.50	11.54	5.30	0.068	yes	Arulanandan et al. 1986
904	Haicheng	1975	7.3	Fisheries and Shipbuilding	60	0.50	6.00	55.23	5.30	0.106	yes	Arulanandan et al. 1986
905	Haicheng	1975	7.3	Middle School	60	1.00	9.50	8.48	3.20	0.387	no	Arulanandan et al. 1986
906	Haicheng	1975	7.3	Chemical Fibre	60	1.50	5.00	26.34	5.30	0.058	yes	Arulanandan et al. 1986

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