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

A Thesis

by

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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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Co-Chairs of Advisory Committee: Dr. Derek V. Morris
Dr. Albert T. Yeung

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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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 (CSRN), the mean particle size (D50), the normalized penetration resistance for the standard penetration test (N₁0), and the normalized cone resistance for the cone penetration test (qₑ₁).

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 Poulus (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 monotomic 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.

![Graph](image)


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 \( \sigma_3 \) 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).

![Diagram showing the effect of drainage on cyclic shear strength.](image)


### 1.4.6 Grain Characteristics

At low relative densities, poorly graded water-deposit sand was found to have a lower cyclic strength that 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, possible, 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 show in Fig. 3.

![Graph](image)

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, \( \tau_{\text{avg}} \), induced at any point in a soil deposit using the relationship:

\[
\tau_{\text{avg}} = 0.65 \gamma h a_{\text{max}} r_d
\]

In which \( a_{\text{max}} \) = maximum horizontal ground surface acceleration; \( \gamma \) = 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, \( \tau_{\text{avg}} \), 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 $0.65 \tau_{\text{max}}$ ($\tau_{\text{max}}$ 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.

![Graph showing stress ratio vs. number of cycles](image)

The data presented in Fig. 4 shows the extent of variation in the equivalent number of uniform cycles computed from recorded acceleration time histories using a consistent procedure. The extent 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):

$$F_L = \frac{R}{L}$$

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

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

(3)

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

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

(4)

Where $N$ is the number of blows of the standard penetration test, $\sigma'_o$ 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:

\[ L = \frac{\tau_{\text{max}}}{\sigma'_o} = \frac{a_{\text{max}} \sigma_o}{g \sigma'_o} r_d \] .........................(5)

Where \( \tau_{\text{max}} \) is the maximum shear stress (kg/cm\(^2\)), \( a_{\text{max}} \) is the maximum acceleration at the ground surface, \( g \) is the acceleration of the gravity, \( \sigma_o \) 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 \( r_d \):

\[ r_d = 1.0 - 0.015 Z \] .........................(6)

Where \( Z \) is the depth in meters. The value of \( a_{\text{max}} \) can be estimated in view of the input bedrock motion and response characteristics of the ground layer. According to Iwasaki (1986) the value of \( a_{\text{max}} \) 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:

\[ a_{\text{max}} = 18.4 \times 10^{0.302M} D^{-0.8} \] .........................(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).
\[ I_L = \int_0^{20} F W(z) \, dz \]  \hspace{1cm} (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>
<thead>
<tr>
<th>( I_L )</th>
<th>Liquefaction risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Very low</td>
</tr>
<tr>
<td>0 &lt; ( I_L ) ≤ 5</td>
<td>Low</td>
</tr>
<tr>
<td>5 &lt; ( I_L ) ≤ 15</td>
<td>High</td>
</tr>
<tr>
<td>( I_L ) &gt; 15</td>
<td>Very high</td>
</tr>
</tbody>
</table>

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 \( (S_{pu}) \) computed from the results of each consolidated-undrained triaxial test.
\[ S_{mu} = q_s \cos \phi_s \]
\[ \sin \phi_s = \frac{q_s}{\sigma_{3e} + q_s} = \frac{q_s}{(\sigma_{3e} - \Delta \mu_s) + q_s} \]
\[ q_s = \frac{\sigma_{3e} - \sigma_{3s}}{2} \]

In which \( \sigma_{1s} - \sigma_{3s} \) is the principal stress difference at the steady state from the triaxial test, \( \sigma_{3e} \) is the effective minor principal stress at the steady state; \( \sigma_{3e} \) is the effective minor principal stress at star of shear (after consolidation); \( \Delta \mu_s \) is the pore pressure induced in the specimen at the steady state of deformation; and \( \phi_s \) 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 (\( \tau_d \)) 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:

\[ F_L = \frac{S_{mu}}{\text{shear stress required to maintain static equilibrium}} = \frac{S_{mu}}{\tau_d} \]

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).

![Diagram](image)

**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 \( \sigma_{cl}/(2\sigma'_o) \) causing liquefaction under 20 cycles of uniform loading must be converted to the stress ratio in terms of the maximum shear stress, \( \tau_{\max}/\sigma'_o \). The conversion can be done simply by dividing the cyclic stress ratio by the reduction factor, \( R_r \), 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 \( \mu/\sigma'_o \) is determined from the following relationship \( \tau_{\max}/\sigma'_o \) is calculated with the equation:

\[
\frac{\tau_{\max}}{\sigma'_o} = \frac{3}{1 + 2R_r} \frac{\tau_{\max,1}}{\sigma'_o} \tag{11}
\]

(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 \( \tau_{\max}/\sigma'_o \) can be computed with the following equation:

\[
\tau_{\max}/\sigma'_o = \frac{a_{\max}}{g} r_j h \left( \frac{Z}{H} \right) \tag{12}
\]

(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, \( F_s \), may be defined as:

\[
F_s = \frac{\tau_{\max}/\sigma'_v}{\tau_{\max}/\sigma'_o} \tag{13}
\]
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**

<table>
<thead>
<tr>
<th>Field Testing Technique</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Standard Penetration Test</td>
<td>popular testing tool, large data base</td>
<td>equipment variable, blow count average/12&quot;</td>
</tr>
<tr>
<td>(SPT)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cone Penetration Test (CPT)</td>
<td>continuous reading, economical, fast, standardized test</td>
<td>no sample, limited data base</td>
</tr>
<tr>
<td>Pressuremeter (PMT)</td>
<td>test soils in “undisturbed state”</td>
<td>present approach required lab testing</td>
</tr>
<tr>
<td>Dilatometer (DMT)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shear Wave Velocity</td>
<td>no drilling necessary, test difficult sites (gravels, etc)</td>
<td>no sample, limited use, limited reliability</td>
</tr>
</tbody>
</table>

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/ft$^2$. 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 \( (D_{50}) \).

![Graph showing comparison of curves proposed by different researchers](image)


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 or 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):

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

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 aspect 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 \(q_{ot}\), 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 Teparska (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.

![Diagram](image-url)

**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, \( q_{\text{crit}} \), separating liquefiable from non-liquefiable conditions to a depth of 15m is determined by:

\[
q_{\text{cr}} = q_{\infty}[1-0.065(H_\infty-2)][1-0.05(H_o-2)]\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(15)
\]

Where \( H_\infty \) is the depth to groundwater table (m), \( H_o \) is the thickness of cohesive overburden (m); \( q_{\infty} \) is the reference critical CPT value in MPa for liquefiable conditions when \( H_o=2 \text{ m} \) and \( H_\infty=2 \text{ m} \) 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 \( C_2=D_{50}/0.25 \).

\[
(q_{\text{cl}})_{\text{cr}} = C_2 \left[ 50 + \frac{\left( -\frac{\tau}{\sigma_0} - 0.1 \right)}{\left( -\frac{\tau}{\sigma_0} + 0.1 \right)} \right] \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(16)
\]

### 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, \( \psi \). The relationship is shown in Fig. 8. The dilation angle is derived from 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.

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 ($u$) 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, Vancuver, 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, $K_r$; 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 $(\tau/\sigma_v)$ and the horizontal stress index $K_d$.

\[
\frac{\tau}{\sigma_v} = \frac{K_d}{10} \tag{17}
\]

Marchetti has shown that $K_d$ appears to increase with increases in $K_r$, 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.
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.


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

$$P(Y=1) = \int_{\Omega} \left[ 1 - \exp(-\lambda T) \right] \psi^P(Y=1/\Omega, \Psi) g(\Psi) d\Psi \right] g(\Omega) d(\Omega). \quad (18)$$
For the usual case of small values resulting from the integral over $\Psi$ (which is the probability that any earthquake will result in liquefaction), $P$ is well approximated by:

$$P(Y=1) = \lambda T \int \Omega^{\Psi} P(Y=1/\Omega, \Psi) f(\Omega) g(\Psi) d\Psi d(\Omega) \ldots \ldots \ldots \ldots \ldots \ldots (19)$$

Where $\Psi$ is a vector of earthquake load parameters, $\Omega$ 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 represents the fact that site conditions are often spatially variable and inhomogeneous, and $\lambda$ 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 $P(Y=1/O, \Psi)$ 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 Kavananzian 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: $P(Y=I/Dr,A)=1$ 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 satisfies 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 simple as:

$$P[F_L] = P[F_L|E]P[E] \quad \text{(20)}$$

In which $P[F_L]$ 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 at 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:

$$P[F_L] = \sum E P[F_L|E]P[E] \quad \text{(21)}$$

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 $\Omega$ and $\Psi$ do not need to be estimated and that $P(Y=1/\Omega, \Psi)$ is directly evaluated with no need to appeals 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 use, 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 \( q_e \) 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 Ritcher 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:

\[
\text{CSR} \approx 0.65 \frac{a}{g} \frac{\sigma_v}{\sigma_d} \tag{22}
\]

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

\[
\text{CSR}_N \approx 0.65 \frac{a}{g} \frac{\sigma_v}{\sigma_d} \frac{r_d}{r_M} \tag{23}
\]

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 $rd$ (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:

$$r_d = 1.0 - 0.00765z \quad z \leq 9.15m$$

$$r_d = 1.174 - 0.0276z \quad z > 9.15m$$

$$r_M = 0.032M^2 - 0.631M + 3.934$$

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 form one of two attenuation relationships. For cases in California, The Joyner and Boore equation was used:

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

Where $M$ is the moment magnitude and $r=\left(d^2+7.3^2\right)^{1/2}$, in which $d$ is defined as the closest distance (km) to surface projection of the fault rupture. For Japanese earthquakes, the relationship used is due to Kawashima for soft alluvium or reclaimed ground:
\[ a = \frac{0.4109 \times 10^{0.263M}}{g} \left( \frac{R_E + 30}{10^W} \right)^{1.209} \]  

(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):

\[ N_1 = C_N N \]  

(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:

\[ (N_1)_e = C_E N_1 = C_E C_N N \]  

(29)

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

\[ C_N = \left[ \frac{(\sigma' v_0)}{(\sigma' v_0)} \right]^n \]  

(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):

\[ C_N = \sqrt{\frac{1}{\sigma}} \]  

(31)
The values of the correction factor $C_E$ are based on the recommendations of Seed et al. (1985), and are calculated as:

$$C_E = \frac{ER}{60} C_{IAP} C_{ROD} C_{LIN}$$ \hspace{1cm} (32)

In the above equation, $ER$ is the energy ratio defined as the percent of the theoretical free-fall energy transmitted to the rods form 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 $ER=60\%$. $C_{IAP}$ 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 form 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 this corrections is to attempted 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 showed.

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:

$$q_{e1} = c_s q_e$$ \hspace{1cm} (33)

Where $q_{e1}$ is the normalized cone end bearing stress, $q_e$ is the measured cone end bearing stress, $c_s = 1/(\sigma'_0)^n$, $\sigma'_0$ 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:

\[
q_e = C_0 \frac{\sigma_0}{(\sigma'_0)^2} \text{constant} \tag{34}
\]

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 form 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 of 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 to 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>
<thead>
<tr>
<th>Source Catalog</th>
<th>Number of Cases</th>
<th>Earthquake Load Parameters</th>
<th>Site Soil Parameters</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tokimatsu and Yoshimi 1983</td>
<td>51</td>
<td>a, M, CSR</td>
<td>N, N1, D, z, zw, zL, σ₀, FC, CC, GC, D₅₀, UC</td>
<td>Liquefaction occurrence classified in 4 states: extensive, moderate, marginal and no liquefaction</td>
</tr>
<tr>
<td>Shibata and Teparaksa 1988</td>
<td>84</td>
<td>a, M, CSR</td>
<td>q₀, q₁, D, z, zₗ₀, zL, σ₀, D₅₀</td>
<td>Analysis the liquefaction in the same boring under different depths</td>
</tr>
<tr>
<td>Liao 1986</td>
<td>114</td>
<td>a, D, M, R, CSR, CSRN</td>
<td>N, N₁, (N₁)₅₀, D₀, σ₀, zₗ₀, FC, CC, GC, D₅₀, UC</td>
<td>Extensive compilation, show minimum N instead of average penetration resistance.</td>
</tr>
<tr>
<td>Reyna 1991</td>
<td>37</td>
<td>a, M, CSR</td>
<td>q₀, q₁, D, z, zₗ₀, zL, σ₀, D₅₀</td>
<td>Compilation of recent events, only CFT test.</td>
</tr>
<tr>
<td>Tokimatsu et al. 1994</td>
<td>11</td>
<td>a, M, CSR</td>
<td>N, N₁, D, z, zₗ₀, σ₀</td>
<td>Few details of the earthquake characteristics.</td>
</tr>
</tbody>
</table>

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; $S_i$ the covariance matrix within group; $|S_i|$ the determinant of $S_i$; $S$ the pooled covariance matrix; $x$ a vector containing the variable of an observation; $DF^i$ the degree of freedom of the $i$ group; $q_i$ 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:

$$D^2_t(x) = g_1(x,t) + g_2(t)$$  \hspace{1cm} (35)

Where

$$g_1(x,t) = (x - m_t)^T S_i^{-1} (x - m_t) + \log |S_i|$$  \hspace{1cm} (36)

If the within-group covariance matrices are used, or

$$g_1(x,t) = (x - m_t)^T S^{-1} (x - m_t)$$  \hspace{1cm} (37)

If the pooled covariance matrix is used, and

$$g_2(t) = -2 \log(q_t)$$  \hspace{1cm} (38)

If the prior probabilities are not all equal, or

$$g_2(t) = 0$$  \hspace{1cm} (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

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

(40)

An observation is classified into group $\mu$ 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:

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

(41)

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

(42)

$$\text{DF} = 0.5(K-1)P(P+1)$$

(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:

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

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

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

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):

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

Where $\Phi$ 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 $D_1$ in equation 46:

$$\frac{N_1 + N_2 - p - 3}{N_1 + N_2 - 2} D^2$$

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:

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

$$\text{logit}[P] = \ln\left[\frac{P}{1-P}\right] = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_m X_m$$

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 $\beta_k$'s are regression coefficients to be obtained by fitting equation (57) with data. Methods to obtain $\beta_k$ values include a least-squares formulation and the maximum likelihood approach. The second one will be use due to its statistical properties: consistency, asymptotic normality distributed and asymptotic efficiency (Cook and Gross 1968), the last two properties are specially 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:

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

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:

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

This is the likelihood function that need to be seek to maximize with respect to the parameters $\beta_k$. 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 $\beta$ estimates, since the logarithmic function is a monotonic one-to-one transformation. Hence the $L$ denote the log-likelihood function and write as:

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

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 $\beta_k$ (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:

$$\{\beta_k\}_{r+1} = \{\beta_k\}_r - \left[ \frac{\partial^2 L}{\partial \beta_j \partial \beta_k} \right]^{-1}_r \left[ \frac{\partial L}{\partial \beta_k} \right]_r \tag{53}$$
Where
\[
\frac{\partial L}{\partial \beta_k} = \sum_{i=1}^{n} x_{ik}(Y_i - P_i) \\
\frac{\partial^2 L}{\partial \beta_j \partial \beta_k} = \sum_{i=1}^{n} -x_{ij}x_{ik}(1 - P_i)P_i \tag{54}
\]

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 \(\chi^2\) goodness of fit tests.

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

\[
\sum_{j=1}^{s} \frac{(o_j - n_j \bar{\rho}_j)^2}{n_j \rho_j (1 - \rho_j)} \approx \chi^2 \tag{55}
\]

Where \(o_j\) is the actual frequency of the group \(j\), \(n_j\) is the predicted frequency, and \(\bar{\rho}_j\) is the average value of \(\rho\) in group \(j\). Expression 55 is approximately \(\chi^2\) 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.
Where \( R \) is the modified resistance equal to \( q_{c1} \) 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 shown in Appendix. The discriminant analysis was applied in order to obtain the discriminant expression (model) for the modified cone penetration resistance \((q_{c1})\) as a function of normalized cyclic stress ratio \((\text{CSRN})\), and mean diameter \((D_{50})\). Following the procedure described in section 3.4.1, for the first model the within-group covariance matrix \( S_w \) was calculated as:

\[
S_t = \begin{bmatrix}
\text{Var}(\ln \text{CSRN}) & \text{Cov}(\ln \text{CSRN}, \ln q_{c1}) & \text{Cov}(\ln \text{CSRN}, \ln D_{50}) \\
\text{Cov}(\ln \text{CSRN}, \ln q_{c1}) & \text{Var}(\ln q_{c1}) & \text{Cov}(\ln q_{c1}, \ln D_{50}) \\
\text{Cov}(\ln \text{CSRN}, \ln D_{50}) & \text{Cov}(\ln q_{c1}, \ln D_{50}) & \text{Var}(\ln D_{50}) \\
\end{bmatrix}
\]

\( \text{..........................(56)} \)

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

\[
S_{(1,1)} = \frac{\text{Var}(\ln \text{CSRN})^{(1)}\text{DF}^{(1)} + \text{Var}(\ln \text{CSRN})^{(2)}\text{DF}^{(2)}}{\text{DF}^{(1)} + \text{DF}^{(2)}} \text{..........................(57)}
\]
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 (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 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 $D_{30}$, 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 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:

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

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, $q_{k1}$ is obtained replacing the values obtained form 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

<table>
<thead>
<tr>
<th>Model</th>
<th>Expression</th>
<th>$\chi^2$ value</th>
<th>Prob&gt;$\chi^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$q_d \leq 273.99 \text{CSR}N^{0.6857} D_{50}^{0.4555}$</td>
<td>2.3526</td>
<td>0.8846</td>
</tr>
<tr>
<td>2</td>
<td>$q_d \leq 157.62 \text{CSR}N^{0.6821}$</td>
<td>2.2563</td>
<td>0.5209</td>
</tr>
<tr>
<td>3</td>
<td>$q_d \leq 140.7547\text{CSR}N+142.8568D_{50}+6.6805$</td>
<td>4.9972</td>
<td>0.5442</td>
</tr>
<tr>
<td>4</td>
<td>$q_d \leq 133.9881\text{CSR}N+34.3380$</td>
<td>3.5703</td>
<td>0.3117</td>
</tr>
<tr>
<td>5</td>
<td>$q_d \leq 40.5967\ln(\text{CSR}N)+142.3080D_{50}+104.3617$</td>
<td>5.3798</td>
<td>0.4961</td>
</tr>
<tr>
<td>6</td>
<td>$q_d \leq \exp(2.436797\text{CSR}N+2.516253D_{50}+2.940123)$</td>
<td>2.2643</td>
<td>0.8939</td>
</tr>
</tbody>
</table>

The probability of identifying a site as non-liquefiable, $P_n$, 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 $P_n$ 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), logit(P) become equal to 0. The expressions obtained are shown in table 6.

**TABLE 6. Logistic Models for CPT Data**

<table>
<thead>
<tr>
<th>Model</th>
<th>Equation</th>
<th>Goodness-of-fit-statistics</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$q_{cl} \leq 605.44 \text{CSRN}^{0.75146}D_{50}^{0.72118}$</td>
<td>3.6781</td>
<td>0.8160</td>
</tr>
<tr>
<td>2</td>
<td>$q_{cl} \leq 226.73 \text{CSRN}^{0.7075}$</td>
<td>4.1914</td>
<td>0.7575</td>
</tr>
<tr>
<td>3</td>
<td>$q_{cl} \leq 176.0507 \text{CSRN} + 189.36D_{50} + 5.8427$</td>
<td>11.736</td>
<td>0.1096</td>
</tr>
<tr>
<td>4</td>
<td>$q_{cl} \leq 164.85 \text{CSRN} + 46.3894$</td>
<td>7.063</td>
<td>0.4223</td>
</tr>
<tr>
<td>5</td>
<td>$q_{cl} \leq 147.44 \ln(\text{CSRN}) + 185.327D_{50} + 121.94$</td>
<td>13.695</td>
<td>0.0596</td>
</tr>
<tr>
<td>6</td>
<td>$q_{cl} \leq \exp(2.6968 \text{CSRN} + 3.30D_{50} + 3.00)$</td>
<td>6.7132</td>
<td>0.4593</td>
</tr>
</tbody>
</table>

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 normalize penetration resistance \((N_1)_{60}\) as a function of normalized cyclic stress ratio (CSRNT), 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:

\[
S_1 = \begin{bmatrix}
\text{Var}(\ln \text{CSRNT}) & \text{Cov}(\ln \text{CSRNT}, \ln(N_1)_{60}) & \text{Cov}(\ln \text{CSRNT}, \ln D_{50}) \\
\text{Cov}(\ln \text{CSRNT}, \ln(N_1)_{60}) & \text{Var}(\ln(N_1)_{60}) & \text{Cov}(\ln(N_1)_{60}, \ln D_{50}) \\
\text{Cov}(\ln \text{CSRNT}, \ln D_{50}) & \text{Cov}(\ln(N_1)_{60}, \ln D_{50}) & \text{Var}(\ln D_{50}) \\
\end{bmatrix}
\]

\[
(59)
\]

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

\[
S_{(1,1)} = \frac{\text{Var}(\ln \text{CSRNT})^{(1)}DF^{(1)} + \text{Var}(\ln \text{CSRNT})^{(2)}DF^{(2)}}{DF^{(1)} + DF^{(2)}}
\]

\[
(60)
\]

For the other five models the pooled covariance matrix as the same characteristics. The data analysed is summarize in Fig. 14. The values of cyclic stress ratio are normalized to a magnitude of 7.5 event (CSRNT) following the Seed and Idriss formulation (equation 25) and the values of \(q_{ci}\) correspond to the critical depth (depth with the least value of \(q_{ci}\)). 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**

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

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**

<table>
<thead>
<tr>
<th>Model</th>
<th>Expression</th>
<th>$\chi^2$ value</th>
<th>Prob-$\chi^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$(N_i)^{1.337}\leq 124.337$ CSRN$^{1.337}D_{50}^{0.099}$</td>
<td>64.1130</td>
<td>0.0001</td>
</tr>
<tr>
<td>2</td>
<td>$(N_i)^{1.3193}= 116.87$ CSRN</td>
<td>56.2886</td>
<td>0.0001</td>
</tr>
<tr>
<td>3</td>
<td>$(N_i)^{1.3193}= 92.8013$ CSRN$^{1.0944}D_{50}^{1.397}$</td>
<td>25.3466</td>
<td>0.0003</td>
</tr>
<tr>
<td>4</td>
<td>$(N_i)^{1.3193}= 95.1368$ CSRN $-4.5377$</td>
<td>12.8467</td>
<td>0.0005</td>
</tr>
<tr>
<td>5</td>
<td>$(N_i)^{1.3193}= 12.07312$ ln(CSRN)$+6.189683D_{50}^{0.6134}$</td>
<td>75.0750</td>
<td>0.0001</td>
</tr>
<tr>
<td>6</td>
<td>$(N_i)^{1.3193}= \exp(9.763077$ CSRN$+0.27394D_{50}^{0.3884}$</td>
<td>22.5624</td>
<td>0.0010</td>
</tr>
</tbody>
</table>
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, 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 considered equal for liquefaction and non-liquefaction ($P=0.5$), logit($P$) become equal to 0 and the logit regression results of models are shown in Table 9.

**TABLE 9. Logistic Model for SPT Data**

<table>
<thead>
<tr>
<th>Model</th>
<th>Equation</th>
<th>Goodness-of-fit-statistics</th>
<th>$\rho$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$N_{50} \leq 214.1657\text{CSRN}^{1.61705}D_{50}^{0.4095}$</td>
<td>10.963</td>
<td>0.2038</td>
</tr>
<tr>
<td>2</td>
<td>$N_{50} \leq 203.568\text{CSRN}^{1.69380}$</td>
<td>9.1003</td>
<td>0.3339</td>
</tr>
<tr>
<td>3</td>
<td>$N_{50} \leq 93.6194\text{CSRN}+6.60644D_{50} - 6.5385$</td>
<td>11.736</td>
<td>0.1096</td>
</tr>
<tr>
<td>4</td>
<td>$N_{50} \leq 96.3572\text{CSRN}+5.5716$</td>
<td>7.3225</td>
<td>0.5023</td>
</tr>
<tr>
<td>5</td>
<td>$N_{50} \leq 17.58659\ln(\text{CSRN})+6.9985D_{50} + 41.5207$</td>
<td>4.8703</td>
<td>0.7713</td>
</tr>
<tr>
<td>6</td>
<td>$N_{50} \leq \exp(9.167\text{CSRN}+0.81757D_{50} + 0.4368)$</td>
<td>6.7132</td>
<td>0.4593</td>
</tr>
</tbody>
</table>

From the statistic goodness-of-fit values, model 4 is 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 shown 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 (CSRN). Range where the results coincide with most of the methods available, the discrepancy is in the range of 0.4 to 0.6 of CSRN (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 CSRN higher than 0.4 corresponding to values of $q_{c1}$ approximately of 120 kg/cm$^2$. 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 $D_{50}=0.25$ 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 database 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 $\chi^2$ 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 (CSRN) when is analyzed the SPT data
base. Most of the non-liquefaction cases have a CSRN 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 \((N_t)_{50}\) higher than 30, Seed et al. (1983) consider and asymptotic curve
at point 28 of \((N_t)_{60}\) 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 discriminat 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 (CSRN), 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 \(q_{ct}\) 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 kg/cm² ($N_i$)$_{60}$.

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 perform 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.
REFERENCES


TABLE 10. Seed and Idriss (1971) Data Set

<table>
<thead>
<tr>
<th>CASE N.</th>
<th>EARTHQUAKE</th>
<th>DATE</th>
<th>MAGNITUDE</th>
<th>SITE</th>
<th>DEPTH WATER TABLE (m)</th>
<th>CRITICAL DEPTH (m)</th>
<th>AVERAGE PENETRATION RESISTANCE CRITICAL DEPTH (N)</th>
<th>MAXIMUM GROUND SURFACE ACCELERATION (gals)</th>
<th>CYCLIC STRESS RATIO CSRN $\tau_{ay}/\sigma_0'$</th>
<th>FIELD BEHAVIOR</th>
<th>REFERENCE</th>
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<td>101</td>
<td>Niigata</td>
<td>1802</td>
<td>6.6</td>
<td>Niigata</td>
<td>0.9</td>
<td>6.1</td>
<td>6</td>
<td>0.12</td>
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<th>PENETRAT. RESISTANCE AT CRITICAL DEPTH (N)</th>
<th>MAXIMUM GROUND SURFACE ACCELERAT. (gala)</th>
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## TABLE 15. Tokimatsu et al. (1994) Data Set

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VITA

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