Study on liquefaction of soil

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CERTIFICATE

This is to certify that the report entitled, “STUDY OF LIQUEFACTION OF SOIL” submitted by Ms. Amrita Biswas and Mr. Aditya Narayan Naik in partial fulfilment of the requirements for the award of Bachelor of Technology Degree in Civil Engineering at National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by them under our supervision and guidance.

To the best of our knowledge, the matter embodied in the project report has not been submitted to any other University/Institute for the award of any Degree or Diploma

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CONTENTS

ABSTRACT

Liquefaction is the phenomena when there is loss of strength in saturated and cohesion-less soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading.

In this paper the field datas of two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1999,a study of the SPT and CPT case datas has been undertaken. In this paper, some methods have been studied namely, Semi-empirical method of evaluating soil liquefaction potential, Practical reliability based method for assessing soil liquefaction, Robertson method, Olsen method and Juang method. A comparative study has been done using all the above mentioned methods and the error percentages have been calculated for each of them with respect to the actual on field test results to conclude which of the models is better for both SPT and CPT case datas.

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Chapter 1

INTRODUCTION

GENERAL INTRODUCTION

1.1 Definition

Liquefaction is the phenomena when there is loss of strength in saturated and cohesion-less soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. It is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading.

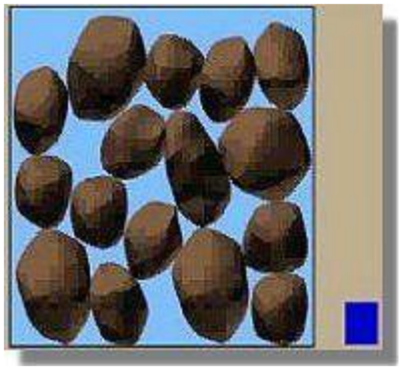
Liquefaction occurs in saturated soils and saturated soils are the soils in which the space between individual particles is completely filled with water. This water exerts a pressure on the soil particles that. The water pressure is however relatively low before the occurrence of earthquake. But earthquake shaking can cause the water pressure to increase to the point at which the soil particles can readily move with respect to one another.

Although earthquakes often triggers this increase in water pressure, but activities such as blasting can also cause an increase in water pressure. When liquefaction occurs, the strength of the soil decreases and the ability of a soil deposit to support the construction above it.

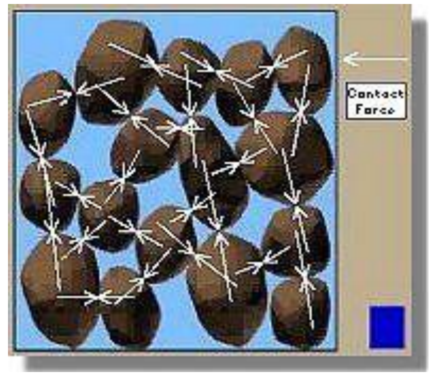
Soil liquefaction can also exert higher pressure on retaining walls, which can cause them to slide or tilt. This movement can cause destruction of structures on the ground surface and settlement of the retained soil.

1.2 Cause behind liquefaction

It is required to recognize the conditions that exist in a soil deposit before an earthquake in order to identify liquefaction. Soil is basically an assemblage of many soil particles which stay in contact with many neighboring soil. The contact forces produced by the weight of the overlying particles holds individual soil particle in its place and provide strength.



Soil grains in a soil deposit. The height of the blue column to the right represents the level of pore-water pressure in the soil.



The length of the arrows represents the size of the contact forces between individual soil grains. The contact forces are large when the pore-water pressure is low.

Occurrence of liquefaction is the result of rapid load application and break down of the loose and saturated sand and the loosely-packed individual soil particles tries to move into a denser configuration. However, there is not enough time for the pore-water of the soil to be squeezed out in case of earthquake. Instead, the water is trapped and prevents the soil particles from moving closer together. Thus, there is an increase in water pressure which reduces the contact forces between the individual soil particles causing softening and weakening of soil deposit. In extreme conditions, the soil particles may lose contact with each other due to the increased pore-water pressure. In such cases, the soil will have very little strength, and will behave more like a liquid than a solid - hence, the name "liquefaction".

1.3 Past records of liquefaction

Earthquakes accompanied with liquefaction have been observed for many years. In fact, written records dating back hundreds and even thousands of years have descriptions of earthquake effects that are now known to be associated with liquefaction. However, liquefaction has been so common in a number of recent earthquakes that it is often considered to be associated with them. Some of those earthquakes are

(1) Alaska, USA(1964)
(2) Niigata, Japan(1964)
(3) Loma Prieta, USA(1989)
(4) Kobe, Japan (1995)

1.4 Methods of reducing liquefaction hazards

There are basically three methods of reducing hazards liquefaction hazards:

1) *By Avoiding Liquefaction Susceptible Soils*

   Construction on liquefaction susceptible soils is to be avoided. It is required to characterize the soil at a particular building site according to the various criterias available to determine the liquefaction potential of the soil in a site

2) *Build Liquefaction Resistant Structures*

   The structure constructed should be liquefaction resistant i.e., designing the foundation elements to resist the effects of liquefaction if at all it is necessary to construct the structure on liquefiable soil because of favourable location, space restriction and other reasons.

3) *Improve the Soil*

   This involves mitigation of the liquefaction hazards by improving the strength, density and drainage characteristics of the soil. This can be done using variety of soil improvement techniques.

Chapter 2

LITERATURE REVIEW

2.1 General literature review

A more precise definition as given by Sladen et al (1985)[6] states that "Liquefaction is a phenomena wherein a mass of soil loses a large percentage of its shear resistance, when subjected to monotonic, cyclic, or shocking loading, and flows in a manner resembling a liquid until the shear stresses acting on the mass are as low as the reduced shear resistance"

Soils have the tendency to decrease in volume when they are subjected to shearing stresses. The soil grains tend to configure themselves into a more denser packing with less space in the voids, as water is forced to move out of the pore spaces. If the drainage of this pore water is obstructed then there is an increase in the pore water pressure with the shearing load. Therefore there is a transfer of stress i.e. there is decrease in effective stress and hence in the shearing resistance of the soil. If the static, driving shear stress is greater than the shear resistance of the soil, then it undergoes deformations which we term as liquefaction. Liquefaction of loose, cohesionless soils can be observed under monotonic as well as cyclic shear loads.

When dense sands are sheared monotonically, the soil gets compressed first, and then it gets dilated as sand particles move up and over one another. When dense saturated sands are sheared impeding the pore water drainage, their tendency of volume increase results in a decrease in pore water pressure and an increase in the effective stress and shear strength. When dense sand is subjected to cyclic small shear strains under undrained pore water conditions, excess pore water pressure may be generated in each load cycle leading to softening and the accumulation of deformations. However, at lager shear strains, increase in volume relieves the excess pore water pressure resulting in an increased shear resistance of the soil.

After initial liquefaction if large deformations are prevented because of increased undrained shear strength then it is termed," limited liquefaction" (Finn 1990)[7]. When dense saturated sands are subjected to static loading they have the tendency to progressively soften in undrained cyclic shear achieving limiting strains which is known as cyclic mobility(Castro 1975; Castro and Poulos 1979)[8]. Cyclic mobility should not be confused with liquefaction. Both can be distinguished from the very fact that a liquefied soil displays no appreciable increase in shear resistance regardless of the magnitude of deformation (Seed 1979)[9]. Soils undergoing cyclic mobility first soften subjected to cyclic loading but later when monotonically loaded without drainage stiffen because tendency to increase in volume reduce the pore pressures. During cyclic mobility, the driving static shear stress is less than the residual shear resistance and deformations get accumulated only during cyclic loading. However, in layman's language, a soil failure resulting from cyclic mobility is referred to as liquefaction.

According to Selig and Chang (1981)[10] and Robertson (1994)[11], a dilative soil can attain a state of zero effective stress and shear resistance. Cyclic loads may produce a reversal in the shear stress direction when the initial static shear stress is low i.e. the stress path passes through a condition which is known as state of zero shear stress. Under such conditions, a dilative soil may accumulate enough pore pressures which help to attain a condition of zero effective stress and large deformations may develop. However, deformations stabilize when cyclic loading comes to an end as the tendency to expand with further shearing increases the effective stresses and hence shear resistance. Robertson (1994)[11] termed this, "cyclic liquefaction". It involves some deformation occurring while static shear stresses exceed the shear resistance of the soil(when the state of zero effective stress is approached).However the deformations stop after cyclic loading ends as the tendency to expand quickly results in strain hardening. This type of failure in saturated, dense cohesionless soils is also referred to as "liquefaction" but with limited deformations.

Compiling all these ground failure mechanisms, Robertson (1994) and Robertson et al(1994)[11]have suggested a complete classification system to define "soil liquefaction". The latest put forward by Robertson and Fear (1996)[12] has been given below:

(1) Flow Liquefaction-The undrained flow of saturated, contractive soil when subjected to cyclic or monotonic shear loading as the static shear stress exceeds the residual strength of the soil
(2) Cyclic softening-Large deformations occurring during cyclic shear due to increase in pore water pressure that would tend to dilate in undrained, monotonic shear.
    Cyclic softening, in which deformations discontinue after cyclic loading stops, can be further classified as

- Cyclic liquefaction-It occurs when the initial, static shear stress is exceeded by the cyclic shear stresses to produce a stress reversal. This may help n attaining a condition of zero effective stress during which large deformations may develop.

- Cyclic mobility-Cyclic loads do not result in a reversal of shear stress and condition of zero effective stress does not occur. Deformations accumulate in each cycle of shear stress.

No definition or classification system appears entirely satisfactorily. Hence a broad definition of soil liquefaction will be adopted for our future study. As defined by the National Research Council's Committee on Earthquake Engineering (1985)[13],soil liquefaction is defined as the phenomena in which there is a loss of shearing resistance or the development of excessive strains as a result of transient or repeated disturbance of saturated cohesionless soils.

2.2 Susceptibility of Soils to Liquefaction in Earthquakes

Liquefaction is most commonly observed in shallow, loose, saturated cohesionless soils subjected to strong ground motions in earthquakes. Unsaturated soils are not subject to liquefaction because volume compression does not generate excess pore water pressure. Liquefaction and large deformations are more associated with contractive soils while cyclic softening and limited deformations are more likely with expansive soils. In practice, he liquefaction potential in a given soil deposit during an earthquake is often evaluated using in-situ penetration tests and empirical procedures.

Since liquefaction phenomena arises because of the tendency of soil grains to rearrange when sheared, any factor that prevents the movement of soil grains will increase the liquefaction resistance of a soil deposit. Particle cementation, soil fabric, and again are some of the important factors that can hinder soil particle movement.

Stress history is also crucial in determining the liquefaction resistance of a soil. For example, soil deposits with an initial static shear stress i.e. anisotropic consolidation conditions are generally more resistant to pore water pressure generation(Seed 1979)[9] although static shear stresses may result in greater deformations since liquefaction gets initiated.

Over consolidated soils (i.e. the soils that have been subjected to greater static pressures in the past) are more resistant to particle rearrangement and hence liquefaction as the soil grains tends to be in a more stable arrangement.

Liquefaction resistance of a soil deposit increases with depth as overburden pressure increases. That is why soil deposits deeper than about 15m are rarely found to have liquefied (Krinitzky et al.1993)[14]

Characteristics of the soil grains like distribution of shapes, sizes, shape, composition etc influence the susceptibility of a soil to liquefy (Seed 1979)[9]. While sands or silts are most commonly observed to liquefy, gravelly soils have also been known to have liquefied.

Rounded soil particles of uniform size are mostly susceptible to liquefaction (Poulus et al.1985)[15]. Well graded soils, due to their stable inter-locking configuration, are less prone to liquefaction. Natural silty sands tend to be deposited in a looser state, and hence are more likely to display contractive shear behaviour, than clear sands.

Clays with appreciable plasticity are resistant to relative movement of particles during shear cyclic shear loading and hence are usually not prone to pore water pressure generation and liquefaction. Soils with n appreciable plastic content are rarely observed to liquefy in earthquakes. Ishihara (1993)[16] gave the theory that non-plastic soil fines with dry surface texture do not create adhesion and hence do not provide appreciable resistance to particle rearrangement and liquefaction. Koester (1994)[17] stated that sandy soils with appreciable fines content may be inherently collapsible, perhaps because of greater compressibility of the fines between the sand grains.

Permeability also plays a significant role in liquefaction. When movement of pore water within the soil is retarded by low permeability, pore water pressures are likely to generate during the cyclic loading. Soils with large non-plastic fines content are more likely to get liquefied because the fines inhibit drainage of excess pore pressures. The permeability of surrounding soils also affects the vulnerability of the soil deposit. Less pervious soils such as clay can prevent the rapid dissipation of excess pore water pressures that may have generated in the adjacent saturated sand deposit. Sufficient drainage above or below a saturated deposit may inhibit the accumulation of excess pore water pressure and hence liquefaction. Gravelly soils are less prone to liquefaction due to a relatively high permeability unless pore water drainage is impeded by less pervious, adjoining deposits.

2.3 Ground Failure Resulting from Soil Liquefaction

The National Research Council (Liquefaction...1985)[13] lists eight types of ground failure commonly associated with the soil liquefaction in earthquakes:

- Sand boils resulting in land subsidence accompanied by relatively minor change.
- Failure of retaining walls due to increased lateral loads from liquefied backfill or loss of support from the liquefied foundation soils.
- Ground settlement, generally linked with some other failure mechanism.
- Flow failures of slopes resulting in large down slope movements of a soil mass.
- Buoyant rise of buried structures such as tanks.
- Lateral spreads resulting from the lateral movements of gently sloping ground.
- Loss of bearing capacity resulting in foundation failures.
- Ground oscillation involving back and forth displacements of intact blocks of surface soil.

The nature and severity of soil liquefaction damage can be said to be a function of both reduced shear strength and the magnitude of the static shear loads acting on the soil deposit. When the reduced strength of a liquefied soil deposit becomes less than the driving shear loads, there is a loss of stability resulting in extensive ground failures or flow slides. And if the shear strength is greater than the driving shear stresses, may be due to the expansion at larger strains, only limited shear deformations are likely to occur. On level ground with no shear stresses acting on it, excess pore water pressures may come out to the surface resulting in the formation of sand boils while the venting of liquefied soil deposits may result in settlements, damages are generally not extensive in the absence of static shear loads.

Ground failures associated with the phenomena of liquefaction under cyclic loading can be classified in a broader sense as (Liquefaction... 1985: Robertson et al.1992)[18]:

(1) Flow failures-It is observed when the liquefaction of loose, contractive soils (i.e. the soils where there is no increase in strength at larger shear strains) results in very large deformations.

(2) Deformation failures-It is observed when there is a gain in shear resistance of the liquefied soil at larger strain, resulting in limited deformations but no loss of stability.

However, putting an end to the confusion in terminology, all types of ground failure resulting from built-up pore water pressure and consequent loss in the shear strength of the soils during cyclic loading is commonly termed as liquefaction.

Chapter 3

FIELD DATAS

3.1 Field data collected

In 1999, two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) as given by Adel M. Hanna, Derin Ural and Gokhan Saygili, “Evaluation of liquefaction potential of soil deposits using artificial neural networks”[2] and Adel M. Hanna, Derin Ural, Gokhan Saygili, “Neural network model for liquefaction potential in soil deposits using Turkey and Taiwan earthquake data”, Soil Dynamics and Earthquake Engineering 27 (2007) 521–540[3]. The ground failure throughout the city of Adapazari (Turkey) and the cities of Wufeng, Nantou and Yuanlin (Taiwan) was attributed to the induced soil liquefaction.

After the 1999 Kocaeli, Turkey earthquake, a group of well respected research institutes around the world carried out a collaborative research program. These research institutes were Brigham Young University, University of California at Los Angeles, University of California at Berkeley, Sakarya University, ZETAS Corporation, Bogazici University and Middle East Technical University with the support of the US National Science Foundation, California Energy Commission, California Department of Transportation and Pacific Gas and Electric Company in the region. A total of 135 profiles were found out of which 46 soil borings with multiple SPT (at 0.8m spacing) and 19 were seismic CPT, which were completed in the city of Adapazari. Details of these investigations were made available by the “Pacific Earthquake Engineering Research Center” (PEER, 2002)[21] in the web site: http://peer.berkeley.edu/turkey/adapazari/ (Adel M. Hanna et al (2007)[4].

For Taiwan earthquake, a series of site investigation programs were done by Adel M. Hanna, Derin Ural and Gokhan Saygil in the year (in 2001-2002) with funding from PEER and by the National Center for Research in Earthquake Engineering (NCREE) in Taiwan in 2000. The PEER and NCREE investigation programs together found out a total of 92 CPT, out of which 98 soil borings with SPT (at 1.0m spacing) and 63 were seismic CPT. The tests were performed by “PEER” and “NCREE” in the cities of Nantou and Wufeng, whereas “NCREE” conducted tests majorly in the city of Yuanlin. Results of these investigations were made available by Stewart et al. (2001)[19] and "PEER" (2003)[20]

3.2 Overview of 1999 Kocaeli, Turkey earthquake and in-situ testing date

The epicenter of the Adapazari, Turkey earthquake was situated in the northwestern part of Turkey and affected a region with a population of nearly 15 million. The multiple rupture process in the 140km long western part of the 1200 km long NAF is the cause behind earthquake. Peak ground accelerations were recorded at approximately 0.4 g. Displacements in the range of 3–4m were measured over a significant length of the fault. The maximum displacement was 5.1 ft immediately east of Arifiye.

3.3 Overview of 1999 Chi-Chi, Taiwan earthquake and in-situ testing data

The epicenter was located near the town Chi-Chi and it had a shallow depth of 7 km. Ground shaking triggered hundreds of strong motion instruments across the island and exceeded 1.0 g in many places. More than 10,700 people were injured and death toll surpassed 2400. In the 1999 Chi-Chi, Taiwan earthquake, the seismic moment (a measure of the energy released by the earthquake) was 50% greater than the 1999 Kocaeli, Turkey Earthquake

Chapter 4

SEMI-EMPIRICAL PROCEDURES FOR EVALUATING LIQUEFACTION POTENTIAL

SEMI-EMPIRICAL PROCEDURES FOR EVALUATING LIQUEFACTION POTENTIAL DURING EARTHQUAKES

Evaluation of the liquefaction potential of saturated cohesionless soils during earthquakes were re-examined and revised using semi-empirical procedures for use in practice by I. M. Idriss, R. W. Boulanger[1]. The stress reduction factor (rd*)*, earthquake magnitude scaling factor for cyclic stress ratios (MSF*)*, overburden correction factor for cyclic stress ratios (K), and the overburden normalization factor for penetration resistances (C_N) were discussed and recently modified relations were presented. These modified relations were used in re-evaluations of the SPT and CPT case history databases. Based on these re-evaluations, revised SPT- and CPT-based liquefaction correlations were recommended for use in practice. In addition, shear wave velocity based procedures and the approaches used to evaluate the cyclic loading behavior of plastic fine-grained soils were also discussed.

Using this procedure, the some SPT and CPT cases of the two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1999, has been evaluated and compared with the liquefaction potential results obtained from the on-field test for both of them.

Basically, Semi-empirical field-based procedures for evaluating liquefaction potential during earthquakes have two essential components: (1) the development of an analytical framework to organize past case history experiences, and (2) the development of a suitable in-situ index to represent soil liquefaction characteristics. There has been a number of re-evaluations to the various components, but the original simplified procedure (Seed and Idriss 1971)[ ] for calculating earthquake induced cyclic shear stresses is still the essential component of this analysis framework.

The strength semi-empirical procedure is the use of both experimental findings together with the theoretical considerations for establishing the framework of the analysis procedure. It is far more advanced method of evaluation because it ties together the theory and the field observations.

The paper by I. M. Idriss, R. W. Boulanger [1] provides an update on the semi-empirical field-based procedures for evaluating liquefaction potential of cohesionless soils during earthquakes. This update includes recommended relations for each part of the analytical framework, including the:

- Stress reduction coefficient **rd** ,
- Magnitude scaling factor **MSF** ,
- Overburden correction factor **K** for cyclic stress ratios, and
- Overburden correction factor $\mathbf{C_N}$ for penetration resistances.

4.1 Overview of the framework for the use of Semi-empirical liquefaction procedures used in this paper:

A brief overview is provided for the framework that is used as the basis for most semi-empirical procedures for evaluating liquefaction potential of cohesionless soils during earthquakes as given by I. M. Idriss, R. W. Boulanger [1] is as follows

4.1.1 The Simplified Procedure for Estimating Cyclic Shear Stress Ratios Induced by Earthquake Ground Motions

The Seed-Idriss (1971) simplified procedure is used to estimate the cyclic shear stress ratios (***CSR***) induced by earthquake ground motions, at a depth ***z*** below the ground surface, using the following equation (1):

$$CSR = 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right) r_d$$

Where a_{max} -maximum horizontal acceleration at the ground surface

v_o - total vertical stress

$v_o{}^{'}$ -effective vertical stress at depth

z- depth

r_d -stress reduction coefficient that accounts for the flexibility of the soil column

4.1.2 *Adjustment for the Equivalent Number of Stress Cycles in Different Magnitude Earthquakes*

It has been customary to adjust the values of CSR calculated by equation (1) so that the adjusted values of CSR would pertain to the equivalent uniform shear stress induced by the earthquake ground motions generated by an earthquake having a moment magnitude M = 7½, i.e., (CSR)$_{M\text{-}7.5}$. Accordingly, the values of (CSR)$_{M\text{-}7.5}$are given by equation (2):

$$(CSR)_{M-7.5} = \frac{CSR}{MSF} = 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{MSF}$$

Where a_{max} -maximum horizontal acceleration at the ground surface

v_o - total vertical stress

$v_o^{'}$ -effective vertical stress at depth

z- Depth

r_d -stress reduction coefficient that accounts for the flexibility of the soil column

MSF- magnitude scaling factor

4.1.3 *Use of the SPT Blow Count and CPT Tip Resistance as Indices for Soil Liquefaction Characteristics*

The effective use of SPT blow count and CPT tip resistance as indices for soil liquefaction charactertics require that the effects of soil density and effective confining stress on penetration resistance be separated [Boulanger and Idriss (2004)]. Hence Seed et al (1975a) included the normalization of penetration resistances in sand to an equivalent σ'_{vo} of one atmosphere (1 ***Pa*** =1 tsf =101 kPa) as part of the semi-empirical procedure. This normalization currently takes the form:

$$(N_1)_{60} = C_N (N)_{60}$$

$$q_{C1} = C_N q_C$$

\

4.1.4. *Stress reduction coefficient, rd*

The stress reduction coefficient ***rd*** was introduced by Seed and Idriss (1971) as a parameter describing the ratio of cyclic stresses for a flexible soil column to the cyclic stresses for a rigid soil column. They obtained values of ***rd*** for a range of earthquake ground motions and soil profiles having sand in the upper 15± m (50 ft) and suggested an average curve for use as a function of depth. The average curve, which was extended only to a depth of about 12 m (40 ft), was intended for all earthquake magnitudes and for all profiles.

Idriss (1999) extended the work of Golesorkhi (1989) and performed several hundred parametric site response analyses and concluded that for the conditions of most practical interest, the parameter ***rd*** could be adequately expressed as a function of depth and earthquake magnitude (M***)***. The following relation was derived using those results:

$$Ln(r_d) = \alpha(z) + \beta(z)M$$

$$\alpha(z) = -1.012 - 1.126\sin\left(\frac{z}{11.73} + 5.133\right)$$

$$\beta(z) = 0.106 + 0.118\sin\left(\frac{z}{11.28} + 5.142\right)$$

These equations given above were considered for z<=34 m. for z >34 m the equation to be used is:

$$r_d = 0.12\exp(0.22M)$$

Where, z-depth

M- Magnitude of the earthquake

r_d - stress reduction coefficient

4.1.5 *Magnitude scaling factor, MSF*

The magnitude scaling factor, *MSF*, has been used to adjust the induced *CSR* during earthquake magnitude *M* to an equivalent *CSR* for an earthquake magnitude, *M* = 7½. The *MSF* is thus defined as:

$$MSF = CSR_M / CSR_{M-7.5}$$

The values of MSF are calculated by combining correlations of the number of equivalent uniform cycles versus earthquake magnitude and the laboratory based relations between the cyclic stress ratio required to cause liquefaction and the number of uniform stress cycles.

Idriss (1999)[22] re-evaluated the MSF derivation using results of cyclic tests on high quality samples obtained by frozen sampling techniques. The re-evaluated relation was slightly different from the simplified procedure (Seed et al 1975)[23]. The MSF relation produced by this reevaluation is given by:

$$MSF = 6.9 \exp\left(\frac{-M}{4}\right) - 0.058$$

Where M- magnitude of the earthquake

4.1.6 *Overburden correction factor, K*

By the studies by Boulanger and Idriss (2004)[1] it is found that overburden stress effects on CRR could be represented in either of two ways: (1) through the additional normalization of penetration resistances for relative state, thereby producing the quantities $(N_1)_{60}$ and q_{C1} ,or (2) through a K factor. The recommended K curves are expressed as (Boulanger and Idriss 2004):

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_{Vo}}{P_a}\right) \le 1.0$$

$$C_\sigma = \frac{1}{18.9 - 2.55\sqrt{(N_1)_{60}}}$$

$$C_\sigma = \frac{1}{37.3 - 8.27\left(q_{c1N}\right)^{0.264}}$$

4.1.7 *Normalization of penetration resistances, C_N*

One of the most commonly used expressions for the overburden correction was proposed by Liao and Whitman (1986), viz:

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{0.5}$$

But after re-evaluation Boulanger and Idriss (2004)[1] subsequently used the relations given below to obtain the following expressions for determining $C_{N:}$

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{\alpha} \leq 1.7$$

$$\alpha = 0.784 - 0.0768\sqrt{(N_1)_{60}}$$

$$C_N = \left(\frac{P_a}{\sigma'_{vo}}\right)^{\beta} \leq 1.7$$

$$\alpha = 1.338 - 0.249\left(q_{c1N}\right)^{0.264}$$

4.2 SPT-BASED PROCEDURE FOR EVALUATING LIQUEFACTION POTENTIAL OF COHESIONLESS SOILS

Semi-empirical procedures for the liquefaction potential analysis was developed using the Standard Penetration Test (SPT) for differentiating between liquefiable and non-liquefiable conditions in the 1964 Niigata earthquake, Japan. In this paper we have used the semi-empirical approach for differentiating between liquefiable and non-liquefiable conditions for 40 SPT cases the two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1999. Thus following the semi-empirical approach, the CSR and $(N_1)_{60}$ values were re-calculated using the revised r_d, MSF , K and C_N relations recommended herein.

4.2.1. Evaluation of CSR

The K factor is usually applied to the "capacity" side of the analysis during design but it must also be used to convert the CSR [Boulanger and Idriss (2004)[1] It is given as follows:

$$(CSR)_{M=7.5} = 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{MSF}\frac{1}{K_\sigma}$$

4.2.2. Evaluation of CRR

For the CRR value, at first the SPT penetration resistance was adjusted by Boulanger and Idriss (2004)[1] to an equivalent clean sand value:

$$(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60}$$

$$\Delta(N_1)_{60} = exp\left(1.63 + \frac{9.7}{FC} - \left(\frac{15.7}{FC}\right)^2\right)$$

The value of the CRR for a magnitude of earthquake=7.5 and an effective vertical stress of 1 atm can be calculated on the basis of the value of $(N_1)_{60cs}$ using the following expression:

$$CRR = exp\left\{\begin{array}{l}\dfrac{(N_1)_{60cs}}{14.1} + \left(\dfrac{(N_1)_{60cs}}{126}\right)^2 \\ -\left(\dfrac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\dfrac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\end{array}\right\}$$

4.2.3 Datas used:

The datas used were the SPT case datas from two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1994 as given by Adel M. Hanna, Derin Ural, Gokhan Saygili, "Neural network model for liquefaction potential in soil deposits using Turkey and Taiwan earthquake data", Soil Dynamics and Earthquake Engineering 27 (2007) 521–540[4]. 40 numbers of datas were analyzed using semi-empirical procedures for evaluating the liquefaction potential.

4.2.4 Calculation and Table

The following three tables gives the calculation for the values of CSR and CRR, followed by the assessment of liquefaction potential from these values found out by the semi-empirical method. Table 3 gives a comparison between these results and the actual on-field results as given in the paper by Adel M. Hanna, Derin Ural, Gokhan Saygili, "Neural network model for liquefaction potential in soil deposits using Turkey and Taiwan earthquake data", Soil Dynamics and Earthquake Engineering 27 (2007) 521–540[4].

Table 1: calculation of CSR by semi-empirical method using SPT case datas

Sl.no.	Z	r_d	MSF	$(N_1)_{60}$	C_σ	$\frac{\sigma'_{Vo}}{P_a}$	K	$\frac{\sigma_{vo}}{\sigma'_{vo}}$	CSR
1	1	0.779	1.027	6	0.079	0.139	1.156	1.164	0.199
2	1.8	0.778	1.027	8	0.086	0.204	1.137	1.5	0.26
3	2.6	0.777	1.027	7	0.082	0.27	1.107	1.67	0.297
4	3.4	0.778	1.027	5	0.076	0.337	1.083	1.774	0.319
5	4.2	0.777	1.027	5	0.076	0.411	1.068	1.827	0.332
6	5	0.778	1.027	3	0.069	0.473	1.052	1.885	0.349
7	6	0.777	1.027	3	0.069	0.553	1.041	1.936	0.361
8	7	0.777	1.027	19	0.128	0.65	1.055	1.95	0.359
9	8	0.776	1.027	26	0.17	0.745	1.05	1.96	0.362
10	9	0.776	1.027	48	0.811	0.842	1.139	1.968	0.335
11	1	0.779	1.027	3	0.069	0.156	1.128	1.139	.0197
12	1.8	0.778	1.027	5	0.076	0.224	1.114	1.451	0.253
13	3.4	0.778	1.027	2	0.065	0.358	1.067	1.724	0.314
14	4.2	0.777	1.027	10	0.092	0.428	1.078	1.792	0.323
15	6	0.777	1.027	4	0.072	0.567	1.041	1.911	0.357
16	7	0.777	1.027	11	0.096	0.654	1.041	1.941	0.362
17	8.5	0.776	1.027	39	0.336	0.782	1.083	1.977	0.354
18	10	0.777	1.027	25	0.163	0.935	1.011	1.977	0.38
19	1.8	0.778	1.027	7	0.082	0.192	1.135	1.562	0.268
20	2.8	0.778	1.027	4	0.072	0.276	1.093	1.749	0.310
21	3.7	0.777	1.027	6	0.079	0.357	1.081	1.828	0.328
22	5.6	0.777	1.027	5	0.076	0.505	1.052	1.959	0.362
23	6.5	0.777	1.027	17	0.119	0.585	1.064	1.98	0.361
24	8.5	0.776	1.027	49	0.952	0.784	1.232	1.984	0.312
25	4.1	0.777	1.027	7	0.082	0.409	1.073	1.794	0.325
26	5	0.778	1.027	3	0.069	0.482	1.05	1.858	0.344
27	6.6	0.777	1.027	18	0.124	0.616	1.06	1.932	0.354
28	8	0.777	1.027	32	0.223	0.753	1.063	1.943	0.355
29	9.5	0.776	1.027	75	-0.314	0.916	0.97	1.938	0.388
30	11	0.776	1.027	33	0.235	1.082	0.98	1.931	0.382
31	12.5	0.775	1.027	33	0.235	1.25	0.948	1.924	0.393
32	15	0.775	1.027	10	0,092	1.464	0.965	1.959	0.393
33	1.6	0.777	1.027	3	0.069	0.249	1.096	1.008	0.179
34	2.6	0.778	1.027	3	0.069	0.423	1.059	1.007	0.185
35	3.5	0.777	1.027	10	0.092	0.541	1.057	1.073	0.197
36	4.2	0.777	1.027	12	0.099	0.607	1.049	1.179	0.218
37	5	0.778	1.027	31	0.213	0.696	1.077	1.27	0.229
38	6.2	0.777	1.027	33	0.235	0.846	1.039	1.363	0.255
39	8	0.777	1.027	31	0.213	1.05	0.99	1.461	0.287
40	10.5	0.776	1.027	8	0.086	1.27	0.98	1.577	0.312

Table 2: calculation of CRR by semi-empirical method using SPT case datas

Sl.no.	FC	$\Delta(N_1)_{60}$	$(N_1)_{60}$	$(N_1)_{60cs}$	CRR
1	90	5.51	6	11.51	0.129
2	94	5.5	8	13.5	0.144
3	100	5.49	7	13.49	0.136
4	87	5.52	5	10.52	0.122
5	74	5.56	5	10.56	0.122
6	92	5.51	3	8.51	0.108
7	97	5.49	3	8.49	0.108
8	70	5.57	19	24.57	0.28
9	58	5.61	26	31.61	0.607
10	5	.0019	48	48.0019	162.26
11	74	5.56	3	5.56	0.108
12	86	5.53	5	1.053	0.122
13	85	5.53	2	7.53	0.102
14	93	5.51	10	15.51	0.16
15	99	5.49	4	9.49	0.115
16	85	5.53	11	16.53	0.17
17	8	0.365	39	390365	3.38
18	6	0.0273	25	25.0273	0.29
19	99	5.49	7	13.49	0.136
20	99	5.49	4	9049	0.115
21	79	5.55	6	11.55	0.129
22	96	5.5	5	10.5	0.122
23	88	5.52	17	22.52	0.24
24	9	0.715	79	49.715	499.25
25	97	5.49	7	12.49	0.136
26	98	5.49	3	8.49	0.108
27	92	5.51	18	23.51	0.25
28	66	5.59	32	37.59	2.036
29	8	0.365	75	75.365	$6*10^{20}$
30	10	1.145	33	34.145	0.935
31	7	0.133	33	33.133	0.93
32	100	5.49	10	15.49	0.16
33	96	5.5	3	8.5	0.108
34	82	5.54	3	8.54	0.108
35	21	4.63	10	14.63	0.153
36	14	2.9	12	14.9	0.153
37	29	5.32	31	36.32	1.48
38	5	0.0019	33	33.0019	0.759
39	5	0.0019	31	31.0019	0.607
40	100	5.49	8	13.49	0.144

Table 3: Assessment of liquefaction potential using semi-empirical method for SPT case datas

Sl.no.	CRR (R)	CSR(S)	Performance function Z = R-S	Liquefaction result	Actual liquefaction
1	0.129	0.199	-0.07	yes	No *
2	0.144	0.26	-0.166	yes	No*
3	0.136	0.297	-0.161	yes	No*
4	0.122	0.319	-0.197	yes	No*
5	0.122	0.332	-0.21	yes	Yes
6	0.108	0.349	-0.241	Yes	No*
7	0.108	0.361	-0.253	Yes	No*
8	0.28	0.359	-0.079	Yes	Yes
9	0.607	0.362	0.245	No	No
10	162.26	0.335	161.93	No	No
11	0.108	.0197	-0.089	Yes	Yes
12	0.122	0.253	-0.131	Yes	Yes
13	0.102	0.314	-0.212	Yes	No*
14	0.16	0.323	-0.163	Yes	Yes
15	0.115	0.357	-0.242	Yes	No*
16	0.17	0.362	-0.192	Yes	Yes
17	3.38	0.354	3.026	No	No
18	0.29	0.38	-0.09	Yes	No*
19	0.136	0.268	-0.133	Yes	No*
20	0.115	0.310	-0.195	Yes	No*
21	0.129	0.328	-0.199	Yes	Yes
22	0.122	0.362	-0.24	Yes	Yes
23	0.24	0.361	-0.121	Yes	Yes
24	499.25	0.312	498.938	No	No
25	0.136	0.325	-0.189	Yes	Yes
26	0.108	0.344	-0.236	Yes	No*
27	0.25	0.354	-0.104	Yes	No*
28	2.036	0.355	1.686	No	No
29	$6*10^{20}$	0.388	$6*10^{20}$	no	No
30	0.935	0.382	0.553	No	No
31	0.93	0.393	0.537	No	No
32	0.16	0.393	-0.233	Yes	No*
33	0.108	0.179	-0.071	Yes	No*
34	0.108	0.185	-0.077	Yes	No*
35	0.153	0.197	-0.044	Yes	Yes
36	0.153	0.218	-0.065	Yes	Yes
37	1.48	0.229	1.251	No	No
38	0.759	0.255	0.504	No	No
39	0.607	0.287	0.32	No	No
40	0.144	0.312	-0.168	Yes	No*

4.3.CPT-BASED PROCEDURE FOR EVALUATING LIQUEFACTION POTENTIAL OF COHESIONLESS SOILS

Seed and Idriss (1981)[24] as well as Douglas et al (1981)[25] proposed the use of correlations between the SPT and CPT to convert the then available SPT-based charts for use with the CPT. The CPT-based liquefaction correlation was re-evaluated by Idriss and Boulanger (2003)[26] using case history data compiled by Shibata and Teparaksa (1988)[27], Kayen et al (1992)[28], Boulanger et al (1995, 1997)[29], Stark and Olson (1995)[30], Suzuki et al (1997)[31] and Moss (2003)[32].

The re-evaluation of CPT cases will include the same adjustments and perimeter revisions as in case of SPT re-evaluation. The CSR adjustment remains same as in case of the SPT cases but the CRR $-q_{C1N}$ will be adjusted according to the different values of tip resistance (q_c).

4.3.1. Evaluation of CSR

The K factor is usually applied to the "capacity" side of the analysis during design but it must also be used to convert the CSR as given by Boulanger and Idriss (2004)[1]. It is given as follows:

$$(CSR)_{M-7.5} = 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{MSF}\frac{1}{K_\sigma}$$

4.3.2. Evaluation of CRR

The revised $CRR - q_{C1N}$ relation, derived using the considerations can be expressed as follows:

$$CRR = exp\begin{Bmatrix} \frac{q_{c1N}}{540} + \left(\frac{q_{c1N}}{67}\right)^2 \\ -\left(\frac{q_{c1N}}{80}\right)^3 + \left(\frac{q_{c1N}}{114}\right)^4 - 3 \end{Bmatrix}$$

Where,

$$q_{C1} = C_N \, q_C$$

4.3.3. Datas used

The datas used were the CPT case datas from two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1994 as given by Adel M. Hanna, Derin Ural, Gokhan Saygili, “Evaluation of liquefaction potential of soil deposits using artificial neural networks”[3]. 28 numbers of datas were analyzed using semi-empirical procedures for evaluating the liquefaction potential.

4.3.4 Calculation and Table

The following three tables gives the calculation for the values of CSR and CRR, followed by the assessment of liquefaction potential from these values found out by the semi-empirical method. Table 6 gives a comparison between these results and the actual on-field results as given in the paper by Adel M. Hanna, Derin Ural, Gokhan Saygili, “Evaluation of liquefaction potential of soil deposits using artificial neural networks”[3].

Table 4: calculation of CSR by semi-empirical method using CPT case datas

Sl.no.	Z	r_d	MSF	$\frac{\sigma_{vo}}{\sigma'_{vo}}$	CSR
1	3.6	0.78	1.027	1.76	0.348
2	4.8	0.78	1.027	1.97	0.389
3	5.8	0.78	1.027	1.65	0.326
4	3.6	0.796	0.97	1.42	0.136
5	17.8	0.793	0.97	1.82	0.174
6	7	0.796	0.97	1.75	0.355
7	3.2	0.797	0.97	1	0.203
8	9.6	0.795	0.97	1.82	0.651
9	8.6	0.795	0.97	1.81	0.646
10	4.4	0.778	1.027	1.81	0.357
11	3.6	0.777	1.027	1.09	0.214
12	11	0.776	1.027	1.89	0.371
13	9.8	0.795	0.97	2.01	0.428
14	11.6	0.795	0.97	1.15	0.11
15	7.8	0.795	0.97	1.8	0.173
16	15.8	0.793	0.97	1.87	0.378
17	3.4	0.797	0.97	1.77	0.359
18	9.8	0.776	0.97	1.48	0.516
19	4	0.778	1.027	1.78	0.587
20	8.8	0.777	1.027	1.97	0.388
21	12.8	0.794	1.027	1.65	0.332
22	12.6	0.794	0.97	1.74	0.167
23	6	0.796	0.97	1.87	0.18
24	2.6	0.796	0.97	1.0	0.203
25	7	0.795	0.97	1.63	0.33
26	2.6	0.796	0.97	1.57	0.561
27	5.2	0.796	0.97	1.8	0.643
28	8.6	0.795	0.97	1.64	0.585

Table 5: calculation of CRR by semi-empirical method using CPT case datas

Sl.no	C_N	q_{C1}	q_{C1N}	CRR
1	1.63	941.33	9.32	0.052
2	1.27	1533.53	15.18	0.054
3	1.37	1013.66	10.04	0.0517
4	1.54	2808.96	27.8	0.0599
5	0.74	1030.45	10.2	0.0518
6	1.16	2195.53	21.74	0.0565
7	1.4	1400.42	13.87	0.0531
8	1	5285.3	52.33	0.0798
9	1.02	4831.33	47.8	0.0754
10	1.5	2283.75	22.6	0.0569
11	1.32	6836.68	67.69	0.0968
12	0.96	9525.6	94.3	0.133
13	1.08	4469.5	44.25	0.0722
14	0.9	2797.83	27.7	0.0599
15	1.21	2230.88	22.088	0.0567
16	0.61	2506.49	24.82	0.0582
17	2.72	5254.77	52.03	0.0795
18	0.79	5322.39	52.7	0.08
19	2.46	5940.9	58.82	0.0866
20	1.22	2971.92	29.42	0.061
21	0.68	1832.87	18.15	0.0548
22	0.75	735.53	7.28	0.051
23	1.69	4375.41	43.32	0.0714
24	2.42	4271.78	42.29	0.0705
25	1.17	8063.87	79.84	0.1124
26	3.18	2993.65	29.64	0.061
27	1.75	3706.89	36.7	0.066
28	0.92	3329.2	32.96	0.0633

Sl.no.	CRR(R)	CSR	FS=CRR/CSR	Liquefaction result	Actual liquefaction
1	0.052	0.348	0.149	yes	No*
2	0.054	0.389	0.139	Yes	Yes
3	0.0517	0.326	0.159	Yes	No*
4	0.0599	0.136	0.44	Yes	Yes
5	0.0518	0.174	0.298	Yes	No*
6	0.0565	0.355	0.159	Yes	Yes
7	0.0531	0.203	0.262	Yes	No*
8	0.0798	0.651	0.117	Yes	Yes
9	0.0754	0.646	0.117	Yes	Yes
10	0.0569	0.357	0.159	Yes	Yes
11	0.0968	0.214	0.452	Yes	Yes
12	0.133	0.371	0.358	Yes	No*
13	0.0722	0.428	0.169	Yes	No*
14	0.0599	0.11	0.545	Yes	Yes
15	0.0567	0.173	0.328	Yes	Yes
16	0.0582	0.378	0.154	Yes	No*
17	0.0795	0.359	0.221	Yes	Yes
18	0.08	0.516	0.155	Yes	Yes
19	0.0866	0.587	0.148	Yes	No*
20	0.061	0.388	0.157	Yes	No*
21	0.0548	0.332	0.165	Yes	Yes
22	0.051	0.167	0.305	Yes	Yes
23	0.0714	0.18	0.397	Yes	No*
24	0.0705	0.203	0.347	Yes	No*
25	0.1124	0.33	0.341	Yes	Yes
26	0.061	0.561	0.109	Yes	No*
27	0.066	0.643	0.103	Yes	No*
28	0.0633	0.585	0.108	Yes	Yes

Table 6: Assessment of liquefaction potential using semi-empirical method for CPT case datas

Chapter 5

A Practical Reliability-Based Method for Assessing Soil Liquefaction Potential

A Practical Reliability-Based Method for Assessing Soil Liquefaction Potential

The methods which are used now-a-days for assessing the liquefaction potential works for depicting whether the soil is liquefiable or not. But these methods are unable to predict the liquefaction probability with respect to the safety factor obtained. This liquefaction probability can be found out by reliability analysis. In the paper by Hwang and Yang [2], reliability analysis method is based on the popular Seed'85 liquefaction analysis method. This method is being used this paper for finding out the liquefaction probability of some SPT case datas of the two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) in 1999 as given by Adel M. Hanna et al (2007)[4].

5.1 Reliability model for soil liquefaction

In simplified liquefaction potential assessment, if CSR (cyclic stress ratio) is denoted by S and CRR (cyclic resistance ratio) is denoted by R then we can say that the performance function is Z=R-S. Hence if Z is less than 0 then the performance function is 'failure', i.e., liquefaction will occur. If Z is greater than 0 then the performance function is 'safe', i.e, there is no liquefaction and if Z is equal to 0 then it is on the boundary between liquefaction and no liquefaction. We can treat R and S as random variables as there would be some inherent uncertainties in the values of both CRR and CSR respectively, hence the liquefaction performance function will also be a random variable. Hence, these three problem states can be thus said to occur with some probabilities.

A simplified calculation method involving statistics is being used in this paper by Hwang and Yang [2] for basic independent random variable, i.e, R and S in this case. According to the principle of statistics, the performance function Z is also a normal distribution random variable, if both R and S are independent random variables under normal distribution. The liquefaction probability P_f equals the probability of Z=R-S <=0. Hence, it can be expressed as

$$P_f = P(Z \leq 0) = \int_{-\infty}^{0} f_z(z)dz = F_z(0)$$

Where, $f_z(z)$ - probability density function (PDF) of Z

$F_z(z)$ - cumulative probability function (CPF) of Z

According to the first-order and second moment method, the mean value μ_z, the standard deviation σ_z and the covariance coefficient δ_z of Z are as follows:

$$\mu_Z = \mu_R - \mu_S$$

$$\sigma_z = \sqrt{\sigma_R^2 + \sigma_S^2}$$

$$\delta_Z = \frac{\sigma_Z}{\mu_Z} = \frac{\sqrt{\sigma_R^2 + \sigma_S^2}}{\mu_R - \mu_S}$$

Where,

Mean values of R and S= μ_R, μ_S

Standard deviation of R and S= σ_R, σ_S

By these equations, the Z can be simply calculated, using the statistics for the basic variables R and S. This is the advantage of the first order and second moment method. A reliability index β can be written as:

$$\beta = \frac{1}{\delta_z} = \frac{\mu_z}{\sigma_Z} = \frac{\mu_R - \mu_S}{\sqrt{\sigma_R^2 + \sigma_S^2}}$$

$$\mu_z = \beta\sigma_z$$

Now, if the liquefaction probability is denoted by P_f then we may say that β has a unique relation with P_f and can be used as an index for calculating the probability of liquefaction.

Hence we may write P_f as:

$$P_f = \int_{-\infty}^{0} f_z(z)dz = \int_{-\infty}^{0} \frac{1}{\sqrt{2\pi}\sigma_z} e^{-\frac{1}{2}(\frac{z-\mu_z}{\sigma_z})^2} dz .$$

Or, this equation can be written as

$$P_f = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{-\frac{\mu_z}{\sigma_z}} e^{-\frac{t^2}{2}} dz = \Phi(-\frac{\mu_z}{\sigma_z}) \ ; \ t = \frac{z-\mu_z}{\sigma_z} ,$$

Here Φ is the cumulative probability function for standard normal distribution.

Now, as $\beta = \mu_z / \sigma_z$, so

$$P_f = \Phi(-\beta) = 1.0 - \Phi(\beta)$$

But practically the basic engineering variables cannot be reasonably modeled by a normal distribution function because they are generally slightly skewed. Hence it was proposed by Rosenblueth and Estra (1972)[33] that they can be described by a log-normal distribution model. Hence in this paper we will consider R and S as log-normal distributions. Thus the reliability index β and the liquefaction probability P_f can be expressed as:

$$\beta = \frac{\mu_Z}{\sigma_Z} = \frac{\mu_{\ln R} - \mu_{\ln S}}{\sqrt{\sigma_{\ln R}^2 + \sigma_{\ln S}^2}} = \frac{\ln\left[\frac{\mu_R}{\mu_S}\left(\frac{\delta_S^2 + 1}{\delta_R^2 + 1}\right)^{1/2}\right]}{\left[\ln(\delta_R^2 + 1)(\delta_S^2 + 1)\right]^{1/2}}$$

$$P_f = \Phi(-\beta) = 1.0 - \Phi(\beta)$$

According to the safety factor-based design method, the safety factor for liquefaction is defined as the ratio of the mean values of R and S. Hence

$$FS = \frac{\mu_S}{\mu_R}$$

Thus we can find out a liquefaction probability for each value of factor of safety. Thus the reliability model can be prepared.

The steps to be followed can be well described by the following flowchart given by Hwang and Yang [2]. This flowchart has been implemented to find out the probability of liquefaction for the SPT case datas of two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake (magnitude Mw = 7.4) as given by Adel M. Hanna et al (2007)[4]. Here the model provides us with the liquefaction probability, using which it may be concluded that which soil sample would be more susceptible to liquefaction than others. Hence here the soil sample giving higher probability of liquefaction is being considered to be “failure” and the ones with lower probability are considered to be “safe”.

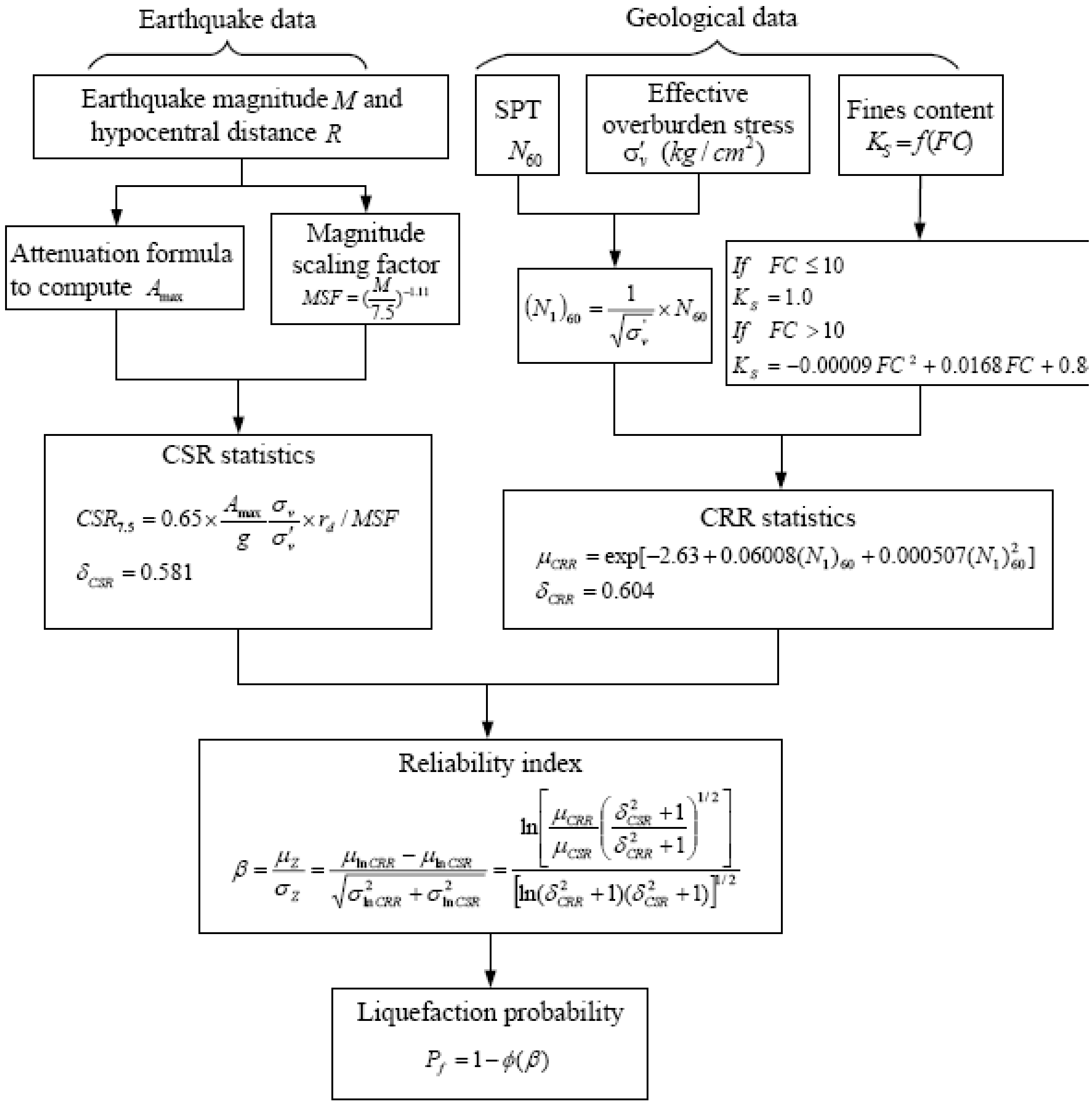


Fig1: Flow chart of the proposed reliability liquefaction analysis method.

5.2 Calculations and tables

The following tables give the mean values of CSR and CRR for the SPT case datas found by the proposed reliability liquefaction analysis method. From these values the factor of safety and the liquefaction probability has been calculated and a comparative assessment has been done considering the soil having more liquefaction probability to be susceptible to "failure" and the soil having less liquefaction probability to be "safe"

Table 7: calculation of μ_S and μ_R

Sl. No.	z	r_d	MSF	$\frac{\sigma_{vo}}{\sigma'_{vo}}$	CSR	μ_S	$(N_1)_{60}$	μ_R
1	1	0.779	1.015	1.164	0.0236	0.024	6	0.105
2	1.8	0.778	1.015	1.5	0.0303	0.0308	8	0.12
3	2.6	0.777	1.015	1.67	0.034	0.0345	7	0.113
4	3.4	0.778	1.015	1.774	0.0359	0.0364	5	0.099
5	4.2	0.777	1.015	1.827	0.0369	0.0375	5	0.099
6	5	0.778	1.015	1.885	0.0381	0.0387	3	0.087
7	6	0.777	1.015	1.936	0.0391	0.0397	3	0.087
8	7	0.777	1.015	1.95	0.0394	0.04	19	0.271
9	8	0.776	1.015	1.96	0.0395	0.0401	26	0.484
10	9	0.776	1.015	1.968	0.0397	0.0403	48	4.145
11	1	0.779	1.015	1.139	0.0231	0.0234	3	0.087
12	1.8	0.778	1.015	1.451	0.0294	0.0298	5	0.099
13	3.4	0.778	1.015	1.724	0.0349	0.0354	2	0.081
14	4.2	0.777	1.015	1.792	0.0362	0.0367	10	0.138
15	6	0.777	1.015	1.911	0.0386	0.0392	4	0.092
16	7	0.777	1.015	1.941	0.0392	0.0398	11	0.148
17	8.5	0.776	1.015	1.977	0.0399	0.0404	39	1.623
18	10	0.777	1.015	1.977	0.0399	0.0404	25	0.444
19	1.8	0.778	1.015	1.562	0.0316	0.0321	7	0.113
20	2.8	0.778	1.015	1.749	0.0354	0.0359	4	0.092
21	3.7	0.777	1.015	1.828	0.0369	0.0375	6	0.105
22	5.6	0.777	1.015	1.959	0.0369	0.0402	5	0.099
23	6.5	0.777	1.015	1.98	0.0399	0.0404	17	0.232
24	8.5	0.776	1.015	1.984	0.04	0.0406	79	4.624
25	4.1	0.777	1.015	1.794	0.0362	0.0367	7	0.113
26	5	0.778	1.015	1.858	0.0377	0.0383	3	0.087
27	6.6	0.777	1.015	1.932	0.039	0.0396	18	0.251
28	8	0.777	1.015	1.943	0.0393	0.0399	32	0.828
29	9.5	0.776	1.015	1.938	0.0391	0.0397	75	6.78
30	11	0.776	1.015	1.931	0.039	0.0396	33	0.909
31	12.5	0.775	1.015	1.924	0.0388	0.0394	33	0.909
32	15	0.775	1.015	1.959	0.0395	0.0401	10	0.138
33	1.6	0.777	1.015	1.008	0.0204	0.0207	3	0.087
34	2.6	0.778	1.015	1.007	0.0204	0.0207	3	0.087
35	3.5	0.777	1.015	1.073	0.0217	0.022	10	0.138
36	4.2	0.777	1.015	1.179	0.0238	0.0242	12	0.159
37	5	0.778	1.015	1.27	0.0257	0.0261	31	0.756
38	6.2	0.777	1.015	1.363	0.0257	0.0279	33	0.909
39	8	0.777	1.015	1.461	0.0295	0.0299	31	0.756
40	10.5	0.776	1.015	1.577	0.0318	0.0323	8	0.12

Table 8: calculation of P_f, FS and assessment of liquefaction probability

Sl. No.	μ_Z	β	$\Phi(\beta)$	P_f	F.S= $\frac{\mu_S}{\mu_R}$	Liquefaction probability result	Actual liquefaction
1	0.081	1.89	0.9706	0.029	0.229	No	No
2	0.089	1.74	0.9506	0.049	0.257	No	No
3	0.079	1.516	0.9357	0.0643	0.305	No	No
4	0.0626	1.277	0.9	0.1	0.368	Yes	No*
5	0.0615	1.238	0.89	0.11	0.379	Yes	Yes
6	0.0483	1.0309	0.8485	0.1515	0.445	yes	No*
7	0.0473	0.998	0.8413	0.1587	0.456	Yes	No*
8	0.231	2.453	0.9928	0.0072	0.148	No	Yes*
9	0.444	3.197	0.9993	0.0007	0.083	No	No
10	4.105	5.958			0.0097	Can’t say	No
11	0.0636	1.679	0.9535	0.0465	0.269	No	Yes*
12	0.0692	1.054	0.8531	0.1469	0.437	Yes	Yes
13	0.0456	1.054	0.8531	0.1469	0.437	Yes	No*
14	0.1013	1.609	0.9463	0.0537	0.284	No	Yes*
15	0.0528	1.086	0.8621	0.1379	0.426	Yes	No*
16	0.1082	1.68	0.9535	0.0465	0.269	No	Yes*
17	1.5826	4.746			0.025	Can’t say	No
18	0.4036	3.076	0.9989	0.0011	0.091	No	No
19	0.0809	1.609	0.9463	0.0537	0.284	No	No
20	0.0561	1.199	0.8849	0.1151	0.39	Yes	No*
21	0.0675	1.314	0.9049	0.0951	0.357	Yes	Yes
22	0.0588	1.149	0.8749	0.1251	0.406	Yes	Yes
23	0.1916	2.24	0.9874	0.0126	0.174	No	Yes*
24	4.58	6.089			0.0088	Can’t say	No
25	0.0763	1.437	0.9250	0.075	0.325	No	Yes*
26	0.0487	1.045	0.8531	0.1469	0.44	Yes	No*
27	0.211	2.367	0.9911	0.0089	0.158	No	No
28	0.788	3.895			0.048	Can’t say	No
29	6.74	6.611			0.0059	Can’t say	No
30	0.869	4.025			0.044	Can’t say	No
31	0.87	4.032			0.043	Can’t say	No
32	0.098	1.579	0.9429	0.0571	0.291	No	No
33	0.066	1.837	0.9671	0.0329	0.238	No	No
34	0.066	1.837	0.9671	0.0329	0.238	No	No
35	0.116	2.353	0.9906	0.0094	0.159	No	Yes*
36	0.135	2.413	0.9920	0.008	0.152	No	Yes*
37	0.73	4.325			0.035	Can’t say	No
38	0.881	4.476			0.031	Can’t say	No
39	0.726	4.15			0.04	Can’t say	No
40	0.0877	1.678	0.9535	0.0465	0.269	No	No

Chapter 6

ROBERTSON METHOD, OLSEN METHOD & JUANG METHOD

CPT penetration resistance has been commonly used for characterization of liquefaction resistance. Thus, the CPT parameters, namely CPT tip resistance(q_C), CPT sleeve friction(f_S), and CPT friction ratio(R_f) are used as an index parameters for liquefaction assessment.

Earthquake magnitude (M_v) and maximum horizontal acceleration at ground surface (a_{max}) characterize the nature of loading, intensity of seismic ground shaking induced by the earthquakes. These values were reported in the soil data of the respective site.

Now the factor of safety against liquefaction has been evaluated in this study using 3 methods namely:

1) Robertson method
2) Olsen method
3) Juang method

From the field data we calculate the CSR (cyclic stress ratio) first using the relation

$$CSR= 0.65\left(\frac{\sigma_{vo} a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{MSF}$$

Where r_d=1-0.00765 z, z ≤9.15m

=1.174-0.0267 z, 9.15< z ≤23m

MSF=(M_v/7.5)^n , where n=-2.56 (as suggested by Idriss)

Finally factor of safety is calculated as FS=CRR/CSR

Now, CRR (cyclic resistance ratio) can be evaluated by any of the methods mentioned above. Each method has different methodology.

6.1 Robertson method

CRR=0.833(q_{c1ncs}/1000) +0.05, if q_{c1ncs}< 50

=93(qc1ns/1000)^3+0.08, if 50 ≤q_{c1ncs}< 160

Mapping function PL=1/ (1+(FS/A)^B), where A=1.0 and B=3.3

Table 9: Robertson table1

z in mtr	v in atm	v' in atm	dw in mtr	qc in atm	fs in atm	Rf	Vs in m/s	at	Mv
3.6	0.666	0.379	0.78	5.775	0.061	1.06	85	0.027	7.4
4.8	1.228	0.624	0.77	12.075	0.356	2.95	173	0.062	7.4
5.8	0.882	0.535	1.34	7.399	0.243	3.28	85	0.021	7.4
3.6	0.605	0.427	1.78	18.24	0.157	0.86	158	0.091	7.6
17.8	3.324	1.823	2.5	13.925	0.314	2.25	172	0.028	7.6
7	1.311	0.751	0.99	18.927	0.441	2.33	205	0.083	7.6
3.2	0.515	0.515	5	10.003	0.059	0.59	161	0.105	7.6
9.6	1.838	1.008	1.14	52.858	0.824	1.56	239	0.086	7.6
8.6	1.744	0.964	0.75	47.366	1.843	3.89	209	0.072	7.6
4.4	0.811	0.449	0.78	15.225	0.234	1.54	85	0.022	7.4
3.6	0.626	0.576	3.1	51.793	0.243	0.47	185	0.128	7.4
11	2.062	1.092	1.3	99.225	0.733	0.74	180	0.045	7.4
9.8	1.75	0.872	0.85	41.384	0.539	1.3	161	0.038	7.4

Table 10: Robertson table2

z in mtr	v in atm	v' in atm	dw in mtr	qc in atm	fs in atm	Rf	Mv	amax	Yd
11.6	1.294	1.126	0.71	31.087	0.373	1.2	7.6	0.18	0.86428
7.8	1.506	0.838	0.99	18.437	0.343	1.86	7.6	0.38	0.94033
15.8	3.114	1.661	0.5	41.09	1.059	2.58	7.6	0.38	0.75214
3.4	0.657	0.372	3.6	19.319	0.373	1.93	7.6	0.67	0.97399
9.8	1.885	1.276	0.78	67.372	3.226	4.79	7.6	0.67	0.91234
4	0.735	0.413	0.77	24.15	0.468	1.94	7.4	0.4	0.9694
8.8	1.635	0.828	3.1	243.6	1.141	0.47	7.4	0.4	0.93268
12.8	2.457	1.487	2.5	26.954	0.496	1.84	7.4	0.4	0.83224
12.6	2.336	1.345	0.71	98.07	0.186	1.9	7.6	0.18	0.83758
6	1.117	0.598	5	25.89	0.275	1.06	7.6	0.18	0.9541
2.6	0.418	0.418	1.5	17.652	0.098	0.56	7.6	0.38	0.98011
7	1.401	0.862	0.75	68.922	0.716	1.04	7.6	0.38	0.94645
2.6	0.499	0.318	0.5	94.14	0.235	2.5	7.6	0.67	0.98011
5.2	1.038	0.576	1.57	211.822	0.441	2.08	7.6	0.67	0.96022
8.6	1.781	1.092	0.99	36.187	1.775	4.91	7.6	0.67	0.93421

MSF	CSR	qc1n	F	Ic	qc1ns	CRR	FS	PL
0.966661	0.120216	29.29609	1.251972	2.397671	63.89253	0.104257	0.867246	0.615391
0.966661	0.431802	20.14041	2.02587	2.649868	74.0333	0.117737	0.272664	0.986459
0.966661	0.360305	31.88242	2.788603	2.576904	99.42591	0.171407	0.475728	0.920678
0.966661	0.774981	31.67475	1.998714	2.488126	81.95375	0.131191	0.169282	0.997161
0.966661	0.6072	59.64226	4.926169	2.555159	177.4121	0.599317	0.987018	0.510778
1.03496	0.433401	37.57873	1.998719	2.4298	86.86926	0.140965	0.325253	0.976023
1.03496	0.462668	267.7085	0.471556	1.372906	230.076	1.212653	2.620998	0.039936
1.03496	0.345456	22.10384	2.024738	2.616809	75.33977	0.11977	0.346701	0.970564
0.966661	0.176071	84.56196	0.194288	1.624447	84.18184	0.13548	0.769462	0.703665
0.966661	0.215704	33.47969	1.11008	2.320559	64.25666	0.104674	0.485267	0.91576
0.966661	0.250437	27.30271	0.568643	2.255356	47.66779	0.090073	0.359664	0.966899
0.966661	0.393053	74.23424	1.060411	2.027134	101.1409	0.17622	0.448335	0.933845
0.966661	0.692887	166.9401	0.250958	1.392843	144.1744	0.358707	0.517699	0.897761
0.966661	0.779578	279.1	0.209219	1.158153	270.0232	1.910991	2.451314	0.049319
0.966661	0.686436	34.6291	5.158984	2.731641	154.3059	0.421689	0.614316	0.833121

Sl no.	Liquefaction result	Actual liquefaction
1	yes	No*
2	yes	Yes
3	yes	No*
4	yes	Yes
5	yes	No*
6	yes	Yes
7	yes	No*
8	yes	Yes
9	yes	Yes
10	yes	Yes
11	yes	Yes
12	yes	No*
13	yes	No*
14	yes	Yes
15	yes	Yes
16	yes	No*
17	yes	Yes
18	yes	Yes
19	yes	No*
20	no	No
21	yes	Yes
22	yes	Yes
23	yes	No*
24	yes	No*
25	yes	Yes
26	yes	No*
27	no	No
28	yes	Yes

6.2 Olsen method

$CRR=0.00128(q_c\ /\ (\sigma'_{vo})^{0.7})-0.025+0.17R_f-0.028*R_f^2+0.0016*R_f^3$

Where q_c and σ'_{vo} is measured in atm

$R_f = f_s/q_c$

$PL=1/\ (1+ (FS/A)^B)$ where A=1 and B=2.78

Table 11: Olsen table1

z	v	v'	qc	fs	Rf	amax	yd	Mw	MSF	CSR	CRR	FS	PL
3.6	0.666	0.379	5.775	0.061	1.06	0.4	0.9725	7.4	1.035	0.4293	0.14	0.3266	0.957327
4.8	1.228	0.624	12.075	0.356	2.95	0.4	0.9633	7.4	1.035	0.4762	0.295	0.6203	0.790439
5.8	0.882	0.535	7.399	0.243	3.28	0.4	0.9556	7.4	1.035	0.3958	0.302	0.7643	0.678572
3.6	0.605	0.427	18.24	0.157	0.86	0.18	0.9725	7.6	0.9667	0.1668	0.144	0.8627	0.601241
18	3.324	1.823	13.925	0.314	2.25	0.18	0.6987	7.6	0.9667	0.1542	0.246	1.5932	0.215045
7	1.311	0.751	18.927	0.441	2.33	0.38	0.9465	7.6	0.9667	0.4222	0.269	0.637	0.777923
3.2	0.515	0.515	10.003	0.059	0.59	0.38	0.9755	7.6	0.9667	0.2493	0.086	0.346	0.950269
9.6	1.838	1.008	52.858	0.824	1.56	0.67	0.9177	7.6	0.9667	0.7539	0.245	0.3255	0.957705
8.6	1.744	0.964	47.366	1.843	3.89	0.67	0.9342	7.6	0.9667	0.7614	0.369	0.4846	0.88225
4.4	0.811	0.449	15.225	0.234	1.54	0.4	0.9663	7.4	1.035	0.4385	0.21	0.4798	0.885111
3.6	0.626	0.576	51.793	0.243	0.47	0.4	0.9725	7.4	1.035	0.2655	0.146	0.5515	0.839499
11	2.062	1.092	99.225	0.733	0.74	0.4	0.8803	7.4	1.035	0.4176	0.206	0.4922	0.877683
9.8	1.75	0.872	41.384	0.539	1.3	0.18	0.9123	7.4	1.035	0.207	0.21	1.017	0.488312

Table 12: Olsen table2

z in mt	v in atm	v' in atm	qc in atm	fs in atm	Rf	Mv	amax	Yd	MSF	CSR	CRR	FS	PL
11.6	1.294	1.126	31.087	0.373	1.2	7.6	0.18	0.8643	0.9667	0.1202	0.178	1.481	0.251
7.8	1.506	0.838	18.437	0.343	1.86	7.6	0.38	0.9403	0.9667	0.4318	0.231	0.536	0.85
15.8	3.114	1.661	41.09	1.059	2.58	7.6	0.38	0.7521	0.9667	0.3603	0.292	0.809	0.643
3.4	0.657	0.372	19.319	0.373	1.93	7.6	0.67	0.974	0.9667	0.775	0.26	0.335	0.954
9.8	1.885	1.276	67.372	3.226	4.79	7.6	0.67	0.9123	0.9667	0.6072	0.395	0.651	0.767
4	0.735	0.413	24.15	0.468	1.94	7.4	0.4	0.9694	1.035	0.4334	0.269	0.62	0.791
8.8	1.635	0.828	243.6	1.141	0.47	7.4	0.4	0.9327	1.035	0.4627	0.405	0.875	0.592
12.8	2.457	1.487	26.954	0.496	1.84	7.4	0.4	0.8322	1.035	0.3455	0.229	0.663	0.758
12.6	2.336	1.345	98.07	0.186	1.9	7.6	0.18	0.8376	0.9667	0.1761	0.31	1.76	0.172
6	1.117	0.598	25.89	0.275	1.06	7.6	0.18	0.9541	0.9667	0.2157	0.173	0.803	0.648
2.6	0.418	0.418	17.652	0.098	0.56	7.6	0.38	0.9801	0.9667	0.2504	0.103	0.413	0.921
7	1.401	0.862	68.922	0.716	1.04	7.6	0.38	0.9465	0.9667	0.3931	0.221	0.563	0.832
2.6	0.499	0.318	94.14	0.235	2.5	7.6	0.67	0.9801	0.9667	0.6929	0.519	0.749	0.691
5.2	1.038	0.576	211.822	0.441	2.08	7.6	0.67	0.9602	0.9667	0.7796	0.621	0.796	0.653

Sl no.	**Liquefaction result**	**Actual liquefaction**
1	yes	No*
2	yes	Yes
3	yes	No*
4	yes	Yes
5	no	No
6	yes	Yes
7	yes	No*
8	yes	Yes
9	yes	Yes
10	yes	Yes
11	yes	Yes
12	yes	No*
13	no	No
14	yes	Yes
15	no	Yes
16	yes	No*
17	yes	Yes
18	yes	Yes
19	yes	No*
20	yes	No*
21	yes	Yes
22	yes	Yes
23	no	No
24	yes	No*
25	yes	Yes
26	yes	No*
27	yes	No*
28	yes	Yes

6.3 Juang method

$CRR=C_{sigma}*\exp(-2.9571+1.264(q_{c1ncs}/100)^{1.25})$

Where $C_{sigma}=-0.016(\sigma'_{vo}/100)^3+0.178(\sigma'_{vo}/100)^2-0.063(\sigma'_{vo}/100)+.903$

$q_{c1ncs}=q_{c1n}(2.429*I_c^4-16.943*I_c^3+44.551*I_c^2-51.497*I_c+22.802)$

$q_{c1n}=q_c/(\sigma'_{vo})^{0.5}$

Soil type index, $I_c=((3.47-\log(q_{c1n}))^2+(\log(F)+1.22)^2)^{0.5}$ where $F=f_s/(q_c-v)*100\%$

$PL=1/(1+(FS/A)^B)$, where A=0.96 and B=4.5

Table 13: Juang table 1

z in mtr	v in atm	v' in atm	dw in mtr	qc in atm	fs in atm	Mv	amax	yd	MSF
3.6	0.666	0.379	0.78	5.775	0.061	7.4	0.4	0.97246	1.03496
4.8	1.228	0.624	0.77	12.075	0.356	7.4	0.4	0.96328	1.03496
5.8	0.882	0.535	1.34	7.399	0.243	7.4	0.4	0.95563	1.03496
3.6	0.605	0.427	1.78	18.24	0.157	7.6	0.18	0.97246	0.966661
17.8	3.324	1.823	2.5	13.925	0.314	7.6	0.18	0.69874	0.966661
7	1.311	0.751	0.99	18.927	0.441	7.6	0.38	0.94645	0.966661
3.2	0.515	0.515	5	10.003	0.059	7.6	0.38	0.97552	0.966661
9.6	1.838	1.008	1.14	52.858	0.824	7.6	0.67	0.91768	0.966661
8.6	1.744	0.964	0.75	47.366	1.843	7.6	0.67	0.93421	0.966661
4.4	0.811	0.449	0.78	15.225	0.234	7.4	0.4	0.96634	1.03496
3.6	0.626	0.576	3.1	51.793	0.243	7.4	0.4	0.97246	1.03496
11	2.062	1.092	1.3	99.225	0.733	7.4	0.4	0.8803	1.03496
9.8	1.75	0.872	0.85	41.384	0.539	7.4	0.18	0.91234	1.03496

CSR	qc1n	F	Ic	qc1ncs	Csigma	CRR	FS	PL
0.429296	9.380638	1.193971	2.814434	51.08039	0.902764	0.080988	0.188653	0.999339
0.476229	15.28603	3.282013	2.870302	95.39501	0.902614	0.154464	0.324348	0.992483
0.39578	10.1157	3.72871	3.047282	96.9997	0.902668	0.158401	0.400224	0.980868
0.166767	27.91328	0.890275	2.337761	55.07627	0.902734	0.085469	0.512504	0.943975
0.154206	10.3134	2.961985	2.982671	84.63744	0.901911	0.130796	0.84819	0.635811
0.422166	21.84046	2.503406	2.675759	85.30279	0.902537	0.132215	0.313182	0.993572
0.249264	13.93884	0.621838	2.537078	39.91615	0.90268	0.070064	0.281082	0.996039
0.753859	52.64783	1.615053	2.25774	92.27742	0.902383	0.147116	0.195151	0.999231
0.761427	48.24232	4.039718	2.554877	143.4952	0.902409	0.341456	0.448443	0.968482
0.438484	22.72135	1.623422	2.552116	67.1842	0.902721	0.101215	0.230829	0.998364
0.265505	68.24327	0.474915	1.865535	81.74005	0.902643	0.125303	0.47194	0.960659
0.417586	94.95324	0.754402	1.852636	112.6241	0.902333	0.203262	0.486755	0.955053
0.206986	44.31739	1.359943	2.270881	79.10258	0.902464	0.12043	0.581829	0.904944

Table 14: Juang table 2

z in mtr	v in atm	v' in atm	dw in mtr	qc in atm	fs in atm	Rf	Mv	amax	Yd
11.6	1.294	1.126	0.71	31.087	0.373	1.2	7.6	0.18	0.86428
7.8	1.506	0.838	0.99	18.437	0.343	1.86	7.6	0.38	0.94033
15.8	3.114	1.661	0.5	41.09	1.059	2.58	7.6	0.38	0.75214
3.4	0.657	0.372	3.6	19.319	0.373	1.93	7.6	0.67	0.97399
9.8	1.885	1.276	0.78	67.372	3.226	4.79	7.6	0.67	0.91234
4	0.735	0.413	0.77	24.15	0.468	1.94	7.4	0.4	0.9694
8.8	1.635	0.828	3.1	243.6	1.141	0.47	7.4	0.4	0.93268
12.8	2.457	1.487	2.5	26.954	0.496	1.84	7.4	0.4	0.83224
12.6	2.336	1.345	0.71	98.07	0.186	1.9	7.6	0.18	0.83758
6	1.117	0.598	5	25.89	0.275	1.06	7.6	0.18	0.9541
2.6	0.418	0.418	1.5	17.652	0.098	0.56	7.6	0.38	0.98011
7	1.401	0.862	0.75	68.922	0.716	1.04	7.6	0.38	0.94645
2.6	0.499	0.318	0.5	94.14	0.235	2.5	7.6	0.67	0.98011
5.2	1.038	0.576	1.57	211.822	0.441	2.08	7.6	0.67	0.96022
8.6	1.781	1.092	0.99	36.187	1.775	4.91	7.6	0.67	0.93421

MSF	CSR	qc1n	F	Ic	qc1ns	Csigma	CRR	FS	PL
0.966661	0.120216	29.29609	1.251972	2.397671	63.89253	0.104257	0.011155	0.092792	0.999973
0.966661	0.431802	20.14041	2.02587	2.649868	74.0333	0.117737	0.014577	0.03376	1
0.966661	0.360305	31.88242	2.788603	2.576904	99.42591	0.171407	0.031249	0.086729	0.99998
0.966661	0.774981	31.67475	1.998714	2.488126	81.95375	0.131191	0.01827	0.023575	1
0.966661	0.6072	59.64226	4.926169	2.555159	177.4121	0.599317	0.414412	0.682498	0.822778
1.03496	0.433401	37.57873	1.998719	2.4298	86.86926	0.140965	0.021149	0.048797	0.999998
1.03496	0.462668	267.7085	0.471556	1.372906	230.076	1.212653	2.264798	4.895078	0.000655
1.03496	0.345456	22.10384	2.024738	2.616809	75.33977	0.11977	0.015116	0.043758	0.999999
0.966661	0.176071	84.56196	0.194288	1.624447	84.18184	0.13548	0.019512	0.110821	0.99994
0.966661	0.215704	33.47969	1.11008	2.320559	64.25666	0.104674	0.011257	0.05219	0.999998
0.966661	0.250437	27.30271	0.568643	2.255356	47.66779	0.090073	0.007723	0.03084	1
0.966661	0.393053	74.23424	1.060411	2.027134	101.1409	0.17622	0.033009	0.083981	0.999983
0.966661	0.692887	166.9401	0.250958	1.392843	144.1744	0.358707	0.137333	0.198204	0.999175
0.966661	0.779578	279.1	0.209219	1.158153	270.0232	1.910991	7.891824	10.1232	2.49E-05
0.966661	0.686436	34.6291	5.158984	2.731641	154.3059	0.421689	0.192693	0.280715	0.996062

Sl no.	Liquefaction result	Actual liquefaction
1	yes	No*
2	yes	Yes
3	yes	No*
4	yes	Yes
5	yes	No*
6	yes	Yes
7	yes	No*
8	yes	Yes
9	yes	Yes
10	yes	Yes
11	yes	Yes
12	yes	No*
13	yes	No*
14	yes	Yes
15	yes	Yes
16	yes	No*
17	yes	Yes
18	yes	Yes
19	yes	No*
20	yes	No*
21	no	Yes
22	yes	Yes
23	yes	No*
24	yes	No*
25	yes	Yes
26	yes	No*
27	yes	No*
28	no	Yes*

6.4 Liquefaction analysis:

FS > 1.00

The above factor of safety is appropriate, only if the design assumptions are conservative; site-specific, higher quality data are used; and the calculation methods chosen are shown to be valid and appropriate for the facility. It should be noted, however, that historically, occasions of liquefaction induced instability have occurred when factors of safety using these methods and assumptions were calculated to be greater than 1.00. Therefore, the use of a factor of safety against liquefaction higher than 1.00 may be warranted whenever:

- A failure would have a catastrophic effect upon human health or the environment,
- Uncertainty exists regarding the accuracy, consistency, or validity of data, and no opportunity exists to conduct additional testing to improve or verify the quality of the data,
- Large uncertainty exists about the effects that changes to the site conditions over time may have on the stability of the facility, and no engineered controls can be carried out that will significantly reduce the uncertainty.

Using a factor of safety less than 1.00 against liquefaction is not considered a sound engineering practice. This is because a factor of safety less than 1.00 indicates failure is likely to occur. Furthermore, performing a deformation analysis to quantify the risks and damage expected to the waste containment facility should liquefaction occur is not considered justification for using a factor of safety less than 1.00 against liquefaction. This is because the strains allowed by deformation analysis are likely to result in decreased performance and loss of integrity in the engineering components. Thus, any failure to the waste containment facility due to liquefaction is likely to be substantial and very likely to increase the potential for harm to human health and the environment. If a facility has a factor of safety against liquefaction less than 1.00, mitigation of the liquefiable layers will be necessary, or another site not at risk of liquefaction will need to be used.

The following five screening criteria, from the above reference, are recommended by Ohio EPA for completing a liquefaction evaluation:

- Geologic age and origin. If a soil layer is a fluvial, lacustrine or aeolian deposit of Holocene age, a greater potential for liquefaction exists than for till, residual deposits, or older deposits.
- Fines content and plasticity index. Liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil. Cohesionless soils having less than 15 percent (by weight) of particles smaller than 0.005 mm, a liquid limit less than 35 percent, and an in situ water content greater than 0.9 times the liquid limit may be susceptible to liquefaction (Seed and Idriss, 1982[34]).

- Saturation. Although low water content soils have been reported to liquefy, at least 80 to 85 percent saturation is generally deemed to be a necessary condition for soil liquefaction. The highest anticipated temporal phreatic surface elevations should be considered when evaluating saturation.
- Depth below ground surface. If a soil layer is within 50 feet of the ground surface, it is more likely to liquefy than deeper layers.

- Soil Penetration Resistance-Liquefaction has been shown to occur if the normalized CPT cone resistance (q_c) is less than 157 tsf (15 MPa) (Shibata and Taparaska, 1988[27]).

If three or more of the above criteria indicate that liquefaction is not likely, the potential for liquefaction can be dismissed. Otherwise, a more rigorous analysis of the liquefaction potential at a facility is required. However, it is possible that other information, especially historical evidence of past liquefaction or sample testing data collected during the subsurface investigation, may raise enough of a concern that a full liquefaction analysis would be appropriate even if three or more of the liquefaction evaluation criteria indicate that liquefaction is unlikely.

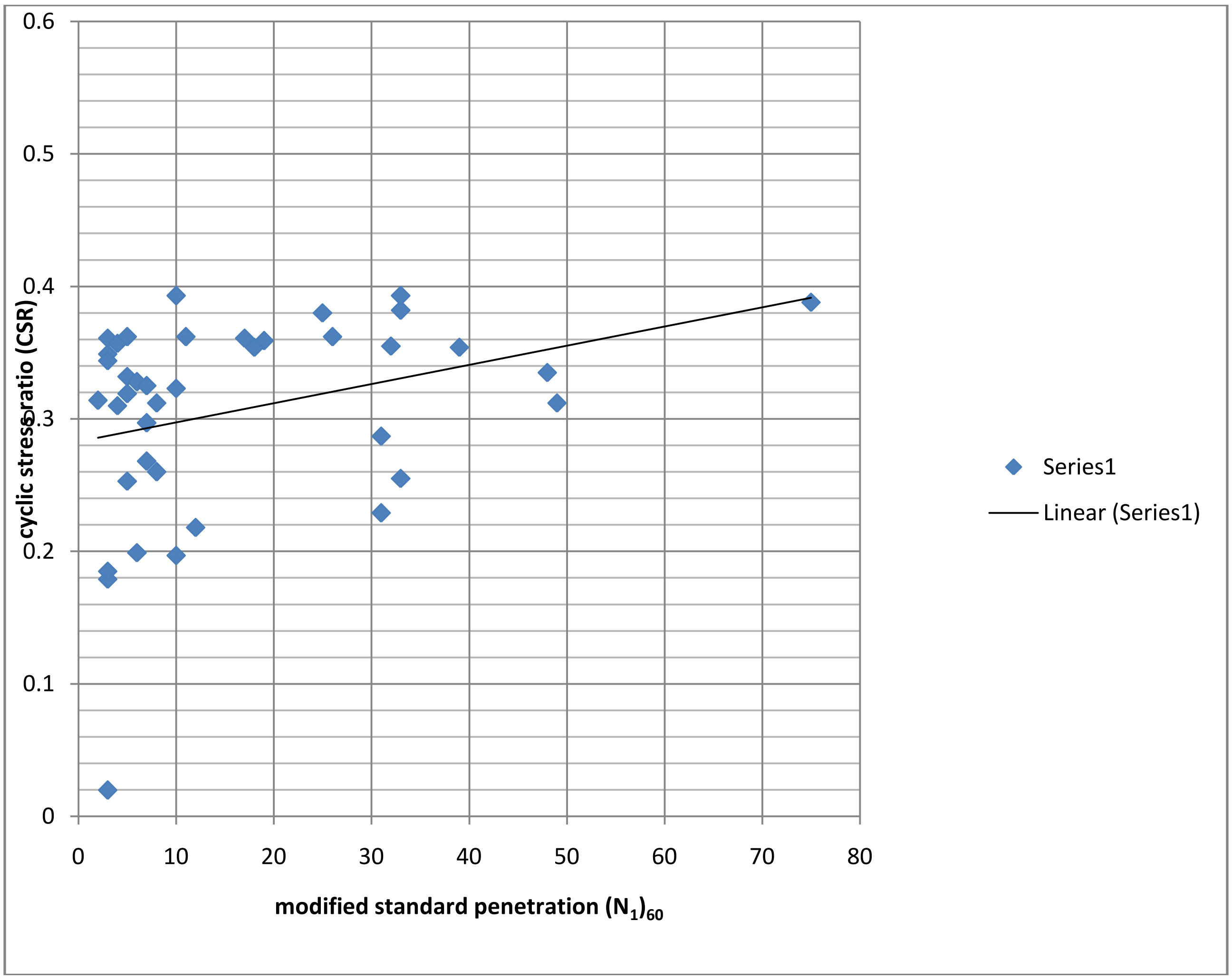
Chapter 7

RESULTS AND DISCUSSIONS

Results and discussions

From the tables and calculations it is seen that the semi-empirical method gives us 17 errors which means an error percentage of 42.5% for the SPT case datas of the two major earthquakes, namely Chi-Chi, Taiwan earthquake (magnitude Mw =7.6) and Kocaeli, Turkey earthquake.

The graph plotted between the modified standard penetrations and the cyclic stress ratio values clearly shows the border-line of liquefaction and the points lying above the line would be susceptible to liquefaction, the ones below it would be "no liquefaction" points and the ones lying over it shows the marginal results obtained.

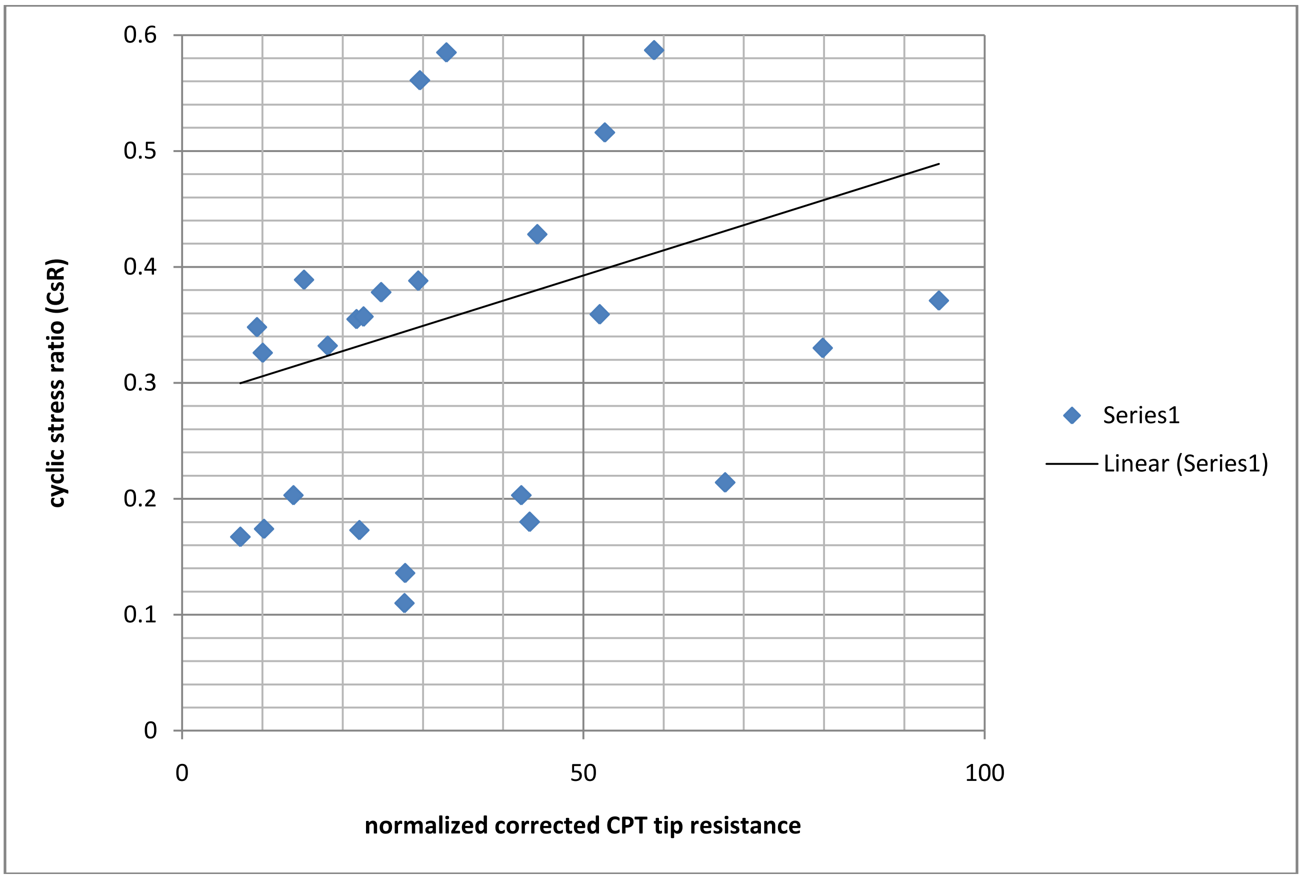


Graph 1:- <u>modified standard penetration Vs CSR(semi-empirical method)</u>

But from the tables and calculations of reliability liquefaction analysis method gives us 15 errors which mean an error percentage of 37.5 % for the same SPT case datas.

Similarly, from the tables and calculations of the semi-empirical method gives us 13 errors which means an error percentage of 46.4% for the CPT case datas of the same Taiwan and Turkey earthquake.

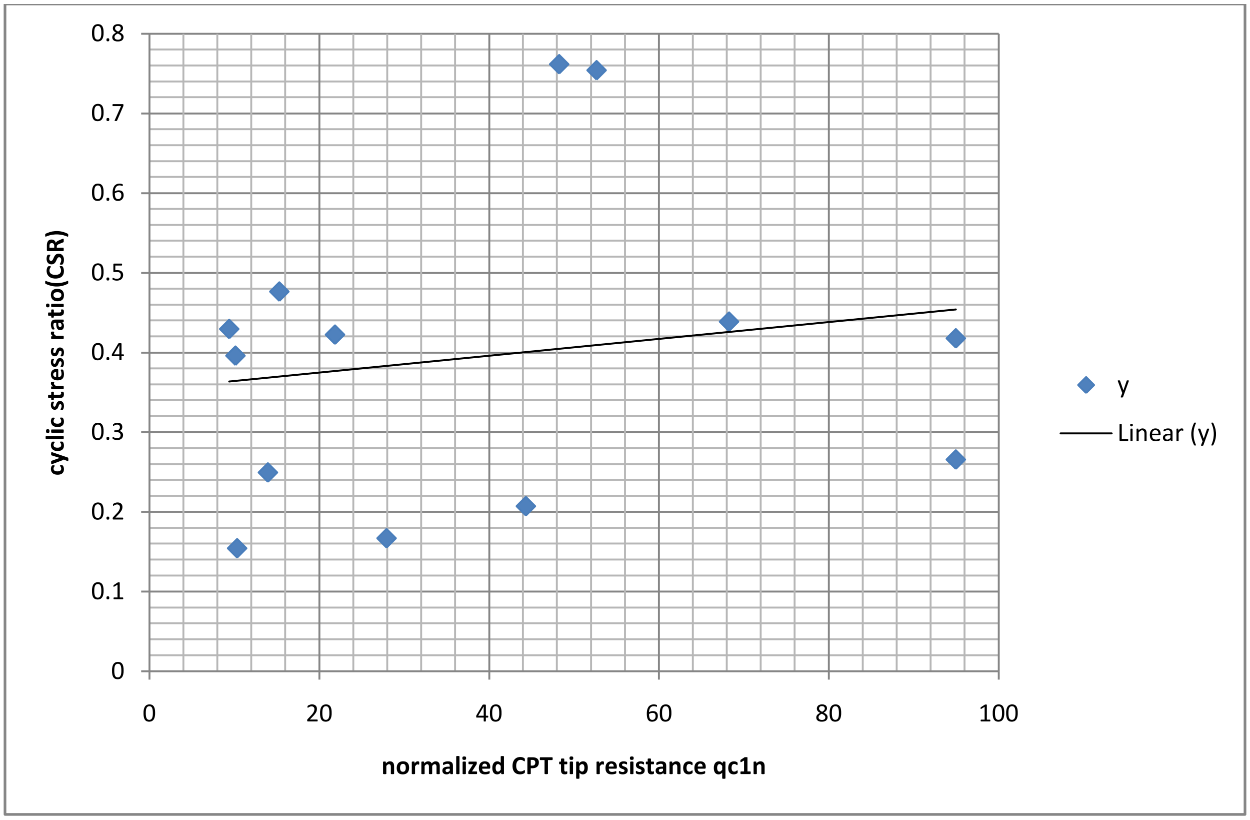
Here also the graph plotted between the normalized corrected CPT tip resistances and the cyclic stress ratio values clearly shows the border-line of liquefaction and the points lying above the line would be susceptible to liquefaction, the ones below it would be "no liquefaction" points and the ones lying over it shows the marginal results obtained.



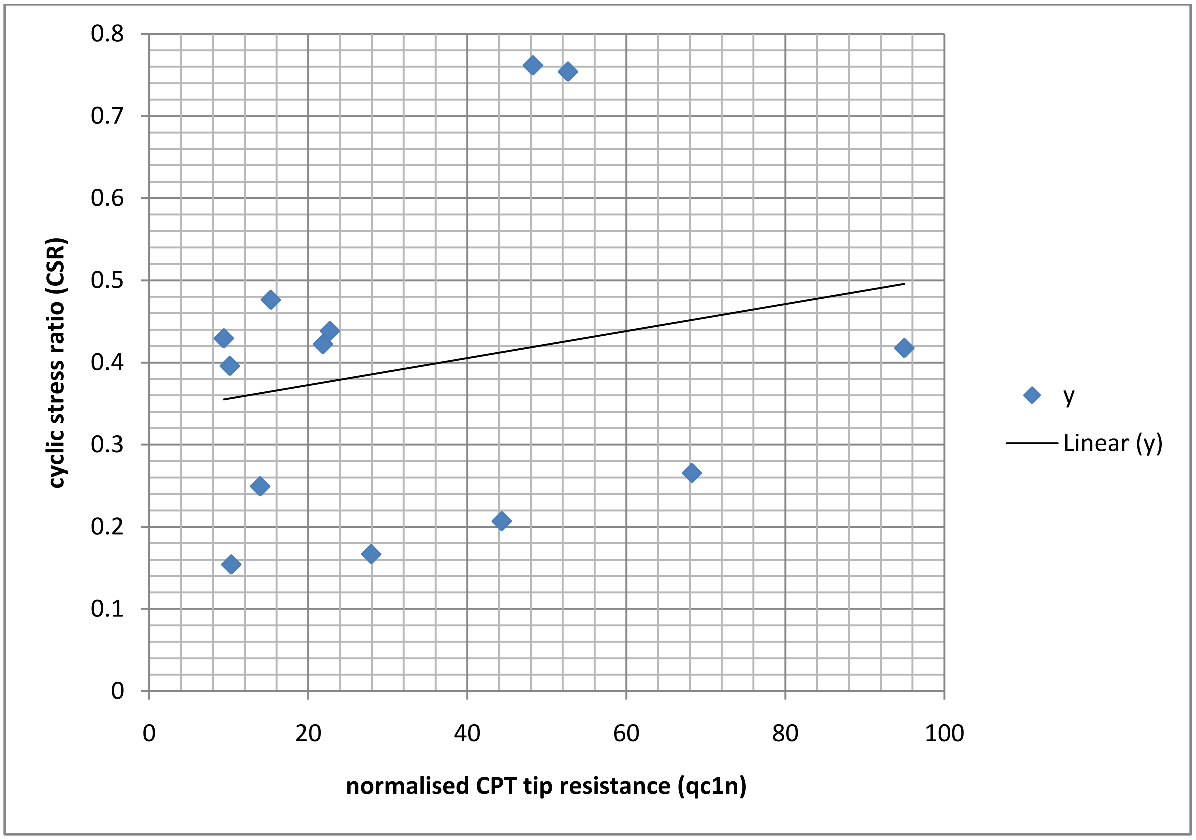
Graph 2: <u>normalized corrected CPT tip resistance Vs CSR(semi –empirical method)</u>

Whereas the tables and calculations of the Robertson method, Olsen method and Juang method gives us 11,10 and 14 errors respectively and error percentages being 39.3%,35.7% and 50% respectively.

Here also the graph plotted between the normalized corrected CPT tip resistances and the cyclic stress ratio values clearly shows the border-line of liquefaction and the points lying above the line would be susceptible to liquefaction, the ones below it would be "no liquefaction" points and the ones lying over it shows the marginal results obtained.



Graph 3: **<u>normalized corrected CPT tip resistance Vs CSR(Robertson method)</u>**



Graph 4: **normalized corrected CPT tip resistance Vs CSR(Juang method)**

Thus it can be said the Reliability liquefaction probability analysis model gives us lower error percentage for the SPT case datas (37.5%) and Olsen method gives lower error percentage for the CPT case datas. (35.7%)

From the error percentages of SPT and CPT case datas studied in this paper, it can be said that CPT datas gives better results concerning liquefaction potential.

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Chapter 8

CONCLUSIONS

CONCLUSIONS

Thus it can be concluded that the Reliability liquefaction probability analysis model gives us lower error percentage for the SPT case datas (37.5%) and Olsen method gives lower error percentage for the CPT case datas (35.7%). Hence, from the limited studies done in this paper we may state the above but for more accurate results more earthquake case datas and other methodologies are to be implemented.

From the error percentages of SPT and CPT case datas studied in this paper, it can be said that CPT datas gives better results concerning liquefaction potential but for practical purposes the above can’t be surely concluded. For accurate results, more earthquake case datas and other methodologies are to be implemented.

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