Geotechnical Earthquake Engineering and Soil Dynamics

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Edited by **Agnes Nolan**

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Preface

This book was inspired by the evolution of our times; to answer the curiosity of inquisitive minds. Many developments have occurred across the globe in the recent past which has transformed the progress in the field.

Various aspects of geotechnical earthquake engineering and soil dynamics are highlighted in this all-inclusive book. The current progress in the field of earthquake engineering has been discussed with primary focus on the seismic safety of dams and underground monuments, Bryan's effect, and the mitigation plans against landslide and fire whirlwind. The book discusses various interesting researches that have been contributed by researchers and experts from many countries. The researches presented in this book will be helpful for graduates, researchers and scientists working in these areas of structural and earthquake engineering. It will also be of significance to civil engineers working on building and reconstruction of structures such as dams, buildings, roads and others.

This book was developed from a mere concept to drafts to chapters and finally compiled together as a complete text to benefit the readers across all nations. To ensure the quality of the content we instilled two significant steps in our procedure. The first was to appoint an editorial team that would verify the data and statistics provided in the book and also select the most appropriate and valuable contributions from the plentiful contributions we received from authors worldwide. The next step was to appoint an expert of the topic as the Editor-in-Chief, who would head the project and finally make the necessary amendments and modifications to make the text reader-friendly. I was then commissioned to examine all the material to present the topics in the most comprehensive and productive format.

I would like to take this opportunity to thank all the contributing authors who were supportive enough to contribute their time and knowledge to this project. I also wish to convey my regards to my family who have been extremely supportive during the entire project.

Editor

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Review on Liquefaction Hazard Assessment

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1. Introduction

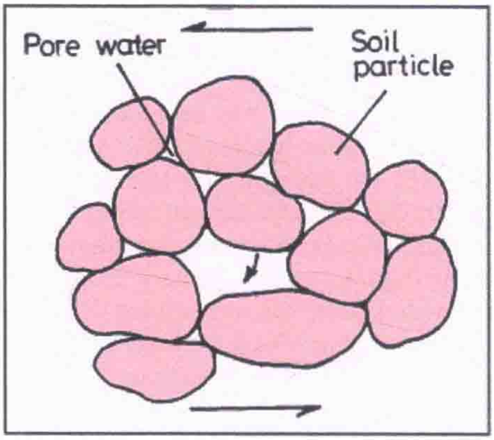
Experience from past earthquakes has demonstrated the vulnerability of structures to seismically induced ground deformation. During earthquake, soil can fail due to liquefaction with devastating effect such as land sliding, lateral spreading, or large ground settlement. The phenomenon of liquefaction of soil had been observed for many years, but was brought to the attention of engineers after Niigata (1964) Alaska earthquakes (1964). Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking or other rapid loading. Liquefaction and related phenomena have been responsible for tremendous amounts of damage in historical earthquakes around the world (Borchardt, 1991). During the Bhuj earthquake on 26th January 2001 (M=7.7) lot of damages had been occurred due to liquefaction and other ground failures (Rao and Mohanty, 2001a). From these investigations it was observed that a vast majority of liquefaction occurrences were associated with sandy soils and silty sands of low plasticity.

For the last four decades, investigations on understanding the liquefaction phenomena were carried out and have resulted in several different perspectives in describing various liquefaction-related phenomena. Liquefaction is looked upon as the condition at which the effective stress reaches (temporarily) a value of zero by few, while others consider liquefaction to have occurred when the soil deforms to large strains under constant shearing resistance. The first phenomenon is referred to as cyclic mobility and the second as flow liquefaction which may result in significant lateral deformations by either of them. To date, most research into liquefaction hazards has concentrated on the question of liquefaction potential, i.e., whether or not liquefaction will occur. The influence of liquefaction on the performance of structures, however, depends on the *effects* of liquefaction. While estimation of liquefaction effects has been improved by development of empirical procedures, the uncertainty involved in predicting these effects is still extremely high. More reliable prediction of structural performance requires more accurate prediction of liquefaction effects (Steven L Kramer, et.al; 2001).

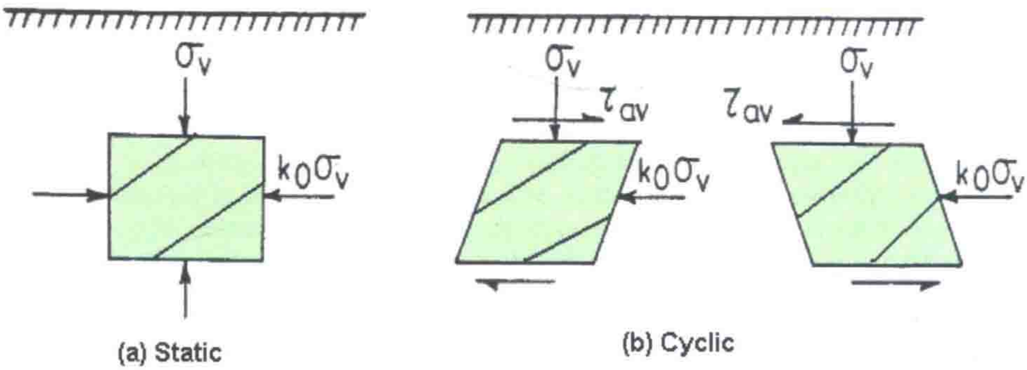
2. Mechanism of soil liquefaction

It is necessary to understand the mechanism of soil liquefaction, where it occurs and why it occurs so often during earthquakes. Figure 1 clearly depicts the mechanism of soil liquefaction. Liquefaction of soil is a process by which sediments below water table temporarily lose shear strength and behave more as a viscous liquid than as a solid. The water in the soil voids exerts pressure upon the soil particles. If the pressure is low enough, the soil stays stable. However, once the water pressure exceeds a certain level, it forces the soil particles to move relative to each other, thus causing the strength of the soil to decrease and failure of the soil follows. During earthquake when the shear wave passes through saturated soil layers, it causes the granular soil structure to deform and the weak part of the soil begins to collapse.

The collapsed soil fills the lower layer and forces the pore water pressure in this layer to increase. If increased water pressure cannot be released, it will continue to build up and after a certain limit effective stress of the soil becomes zero. If this situation occurs then the soil layer losses its shear strength and it can not certain the total weight of the soil layer above, thus the upper layer soils are ready to move down and behave as a viscous liquid. It then is said that soil liquefaction has occurred.



$$\sigma' = (\sigma - u)$$



(a) Static (b) Cyclic

Fig. 1. Mechanism of Soil Liquefaction

2.1 Stress condition at liquefaction

The basic difference between solid state and liquid state of a substance is that the substance in its solid state shows resistance to deformation when subjected to external forces, whereas the substance in its liquid state does not have this property. Therefore, the process of transformation of any substance from solid state into a liquid state is substantially a process of diminishing in shear resistance of the material. The shear resistance of cohesion less soil is mainly proportional to the intergranular pressure and the co-efficient of friction between solid particles which is usually given by the following relationship

$$\tau_s = \sigma' \tan\phi \tag{1}$$

Since $\sigma' = (\sigma - u)$

$$\tau_s = (\sigma - u)\tan\phi \tag{2}$$

where, τ_s is shear resistance, σ' is the effective normal stress, σ is the total normal stress, u is the pore water pressure and ϕ' is the angle of internal friction in terms of effective stress.

The condition for liquefaction is $\sigma' = 0$ then u will be equal to σ. This can be defined in terms of Lambe parameters as

$$q = \tfrac{1}{2}(\sigma_1 - \sigma_3) = \tfrac{1}{2}(\sigma_1' - \sigma_3') \tag{3}$$

$$p = \tfrac{1}{2}(\sigma_1 + \sigma_3) \tag{4}$$

$$p' = 1/2\,(\sigma_1' + \sigma_3') \tag{5}$$

where, σ_1 and σ_3 are maximum and minimum total principal stress, σ_1' and σ_3' maximum and minimum effective principal stress respectively . Then the condition for liquefied soil will be $q = 0$, $p' = 0$ and $p = u$.

Although the condition of soil liquefaction obeys the same condition as explained above, the mechanism of liquefaction process is different. Three typical mechanisms of soil liquefaction are identified as explained below:

2.2 Liquefaction caused by seepage pressure only: Sand boils

If the pore water pressure in a saturated sand deposit reaches and excesses the overburden pressure, the sand deposits will float or boil and lose entirely its bearing capacity. This process is nothing to do with the density and volumetric contraction of sand. Therefore, it has been usually considered as a phenomenon of seepage instability. However, according to the mechanism behavior of the material, it also belongs to the category of soil liquefaction.

2.3 Liquefaction caused by monotonous loading or shearing: Flow slide

The concept of critical void ratio has been suggested by Casagrande. The skeleton of loose saturated sand exhibits irreversible contraction in bulk volume under the action of monotonous loading or shearing, which cause increase of pure water pressure and decrease of effective stress and finally brings about an unlimited flow deformation.

2.4 Liquefaction caused by cyclic loading or shearing: Cyclic mobility

With various experimental techniques and testing apparatus it has been revealed that cohesion less soil always show volumetric contraction at low shear strain level, but may dilate at higher shear strain level depending upon the relative density of soil. Therefore, under the action of cyclic shearing a saturated cohesion less soil could show liquefaction at time intervals when shear strain is low, but may regain shear resistance in time intervals when the shear strain level is higher. A sequence of such sort of intermittent liquefaction would bring about the phenomenon of cyclic mobility with limited flow deformation. If the saturated cohesion less soil was loose enough to keep contraction at high shear strain level, then it also could come out to be an unlimited flow deformation.

3. Evaluation of liquefaction potential

The liquefaction potential of any given soil deposit is determined by a combination of the soil properties, environmental factors and characteristics of the earthquake to which it may be subjected. Specific factors that any liquefaction evaluation desirably takes into account include the following:

SOIL PROPERTIES

- Dynamic shear modulus
- Damping characteristics
- Unit weight
- Grain size characteristics
- Relative density
- Soil structure

ENVIRONMENTAL FACTORS

- Method of soil formation
- Seismic history
- Geologic history (aging, cementation)
- Lateral earth pressure coefficient
- Depth of water table
- Effective confining pressure

EARTHQUAKE CHARACTERISTICS

- Intensity of ground shaking
- Duration of ground shaking

Some of these factors cannot be determined directly, but their effects can be included in the evaluation procedure by performing cyclic loading tests on undisturbed samples or by measuring the liquefaction characteristics of the soil by means of some in-situ tests. The evaluation of liquefaction potential is based on two approaches (i) macroscopic evaluation and (ii) microscopic evaluation. Evaluation of liquefaction potential can be done using field tests or laboratory tests. The preliminary assessment of the liquefaction potential of a soil deposit over a large area in a seismically active region can be done using the following indices, which are characteristics of liquefiable soils:

- Mean grain size, D_{50} = 0.02- 1.0 mm
- Fines content < 10%
- Uniformity coefficient < 10

- Relative density < 75%
- Plasticity index < 10
- Intensity of an earthquake > VI
- Depth < 15m

There are three different ways to predict liquefaction susceptibility of a soil deposit in a particular region. They are (a) Historical criteria, (b) Geological and Geomorphological criteria and (c) Compositional criteria (Kramer, 2000). According to historical criteria soils that have liquefied in past can liquefy in future also. With the help of past earthquake records one can predict the liquefaction in future.

The type of geological processes that created a soil deposit has strong influence on its liquefaction susceptibility. Deposits formed by rivers, lakes, and wind and by man made deposits particularly those created by the process of hydraulic filling are highly susceptible to liquefaction. It also depends on soil type. Uniform graded soils are highly susceptible than well-graded soil deposits also, soils with angular particles are less susceptible than soils with rounded particles.

Cohesive soils with the following properties are vulnerable to significant strength loss under relatively minor strains (Seed et al.1983) i.e. if percent finer than (0.002 mm) is less than 30 percent, liquid limit less than 35 percent and if the moisture content of the insitu soil is greater than 0.9 times the liquid limit (i.e., sensitive clays).

In addition to sandy and silty soils, some gravely soils are potentially vulnerable to liquefaction. Most gravely soils drain relatively well, however gravelly soils are also liquefiable when the voids are filled with fine particles and if it is surrounded by less pervious soils, drainage can be impended and may be vulnerable to cyclic pore pressure generation and liquefaction.

Gravels tend to be deposited in a more turbulent depositional environment than sands or silts, tend to be fairly dense, and so generally resist liquefaction. Accordingly, conservative preliminary methods may often suffice for evaluation of their liquefaction potential. For example, gravely deposits that can be shown to be pre-Holocene in age (older than about 11,000 years) are generally not considered susceptible to liquefaction. Andrus and Stokoe (2000) compiled 225 liquefaction case histories from the United States, Taiwan, Japan and China. Among these case history sites 90% of the liquefied soils had a critical layer thickness of less than 7 m, an average depth below land surface of less than 8 m, and water table depth is at less than 4 m below ground surface.

3.1 Field methods

The use of insitu testing is the dominant approach in common engineering practice for quantitative assessment of liquefaction potential. Calculation of two variables is required for evaluation of liquefaction resistance of soils. They are as follows:

1. The seismic demand on a soil layer, expressed in terms of CSR and
2. The capacity of the soil to resist liquefaction, expressed in terms of *CRR*.

The models proposed by Seed and Idriss (1971), Seed and Peacock (1971), Iwasaki (1978) and Robertson and Wride (1998) methods are extensively used for predicting liquefaction potential using field data. Youd et al. (2001) reviewed in detail the available field methods available for the evaluation of liquefaction potential of soils.

3.1.1 SPT based methods

Standard penetration test is widely used as an economical, quick and convenient method for investigating the penetration resistance of non-cohesive soils. This test is an indirect means to obtain important design parameters for non-cohesive soils. The use of SPT as a tool for evaluation of liquefaction potential began to evolve in the wake of a pair of devastating earthquakes that occurred in 1964; the 1964 Great Alaskan Earthquake (M=9.2) and 1964 Niigata Earthquake (M=7.5), both of which produced significant liquefaction related damage.

It should be ensured that the energy of the falling weight is not reduced by friction between the drive weight and the guides or between rope and winch drum. The rods to which the sampler is attached for driving should be straight, tightly coupled and straight in alignment. For driving the casing, a hammer heavier than 63.5 kg may be used. Standard Penetration Test set up and accessories are Standard split spoon sampler, 65 kg hammer, guide pipe assembly, anvil and drill rod.

In the standard penetration test, a standard split spoon sampler is driven into the soil at the bottom of a borehole by giving repeated blows (30-40 blows per minute), using a 65 kg hammer released from a height of 75 cm. The blow count is found for every 150 mm penetration. If full penetration is obtained, the blows for the first 150 mm are ignored as those required for the seating drive. The number of blows for the next 300 mm of penetration is recorded as the Standard Penetration Resistance, called the 'N' value.

- If number of blows to drive 15 centimeters exceeds 50, the test has to be repeated.
- If the stratification is homogeneous and denseness is not very erratic, the spacing for the test depth can be increased suitably beyond a depth of 6 meters or so.
- Wide variations in N-value at given depth along the section would show heterogeneity of the subsoil and denseness.

3.1.1.1 Factors affecting test results

i. Effective over burden pressure: Effective over burden pressure affects results considerably.

- Desai (1968) reported that N value at shallow depths underestimates relative density. N-value corrected for the surcharge effects represent normal pressure is changed.
- It was proved that positive or negative pore pressure developed in fine sands and silty sands depend upon the denseness of the sub-soil and thus the effective normal pressure against split spoon sampler is altered.
- It was observed that the removal of 5 meters of soil affected N value considerably. In cohesive soils N-value do not reflect the precompression load and shear strengths if soils are partly saturated.

ii. Grain size and shape

- Gravels have reduced friction and penetration resistance, which will block the SPT sampler and gives erratic results.

- The particle size effect on N value is prominent if 30 % soil particles are less than 0.1 mm and soils is saturated or dry.
- In case of silty fine sands and very fine sand, positive or negative pore pressure can be generated depending on the state of compactness and N values will change according

iii. Degree of saturation

- N values will be reduced by 15 % due to saturation and this is more pronounced in case of loose soils.
- Penetration resistance increased due to saturation in case of denser soils while in case of loose, fine and silty sand N value is considerably reduced.

3.1.1.2 Corrections applied in standard penetration test

i. Corrections due to Overburden

It is an established fact that SPT blows are greatly affected by the overburden pressure at the test point. The effect of length or weight of the driving rods is not so pronounced and may be neglected. According to their investigations, Terzaghi-Peck correlation between SPT blows and density index is valid under an overburden pressure of approximately 280 kPa.

The curves are based on results for air dry and partially wetted, cohesion less sands and are considered conservatively reliable in all sands, saturated or unsaturated. But it is generally felt that the corrections provide over-estimate of density index.

For interpretation and correlations of SPT results the current thinking is to adopt 100kPa ($1 kg/cm^2$) as the reference overburden pressure and the N blows corrected for this pressure are called the normalized or corrected values, Nc.

$$Nc = Cn * Nr \tag{6}$$

$$Cn = 0.77 \log_{10}\left(\frac{20}{\sigma'}\right), \text{ where } \sigma' > 0.25 \ kg/cm^2 \tag{7}$$

where, Nr= Observed N value in the field and Cn = Correction factor

The another simple relation for the correction factor Cn that greatly cover more research work on the correction factor carried in the USA was given in Eqn 8 as below.

$$Cn = \sqrt{100/\sigma'} \tag{8}$$

where, σ′ in kPa

ii. Corrections due to Dilatancy

In submerged very fine or silty sands below the water table, the observed value of N may be too great (compared to the penetration resistance of permeable submerged soils of equal density index) if the void ratio is below the critical voids ratio which corresponds approximately to N=15. Submerged fine sands and silty sands offer increased resistance due to excess pore water set up during driving and unable to dissipate immediately (dilatancy effect). The corrected value of N is defined in IS: 2131(1981) is as follows

$$N'=15+\frac{1}{2}(Nc-15) \tag{9}$$

where, Nc is corrected value after over burden correction

Wherever both the overburden and submerged corrections are necessary, the overburden correction is applied first.

3.1.1.3 Seed and Idriss (1971) method

The initial approach for evaluating behavior of soils in the field during dynamic loading was developed by Seed and Idriss (1971). The procedure is referred to as the simplified procedure, and involves the comparison of the seismic stresses imparted onto a soil mass during an earthquake (Cyclic Stress Ratio, CSR) to the resistance of the soil to large magnitude strain and strength loss (Cyclic Resistance Ratio, CRR). The CSR estimation is based on the estimated ground accelerations generated by an earthquake, the stress conditions present in the soil, and correction factors accounting for the flexibility of the soil mass (Youd and Idriss 1997). Seed and Idriss developed this empirical method by combining the data on earthquake characteristics and in-situ properties of soil deposits, which is widely used all over the world for the assessment of liquefaction hazard. For earthquakes of other magnitudes, the appropriate cyclic strength is obtained by multiplying with a factor called magnitude scaling factor MSF. The factor of safety against liquefaction, F_L can then be estimated as the ratio of CSR and CRR.

3.1.1.4 Seed and Peacock (1971) method

In the Seed and Peacock (1971) method, the average shear stress τ_{av} will be computed same as in Seed and Idriss method. Using corrected SPT 'N' value and the proposed chart by Seed and Peacock, τ_Z can be calculated at the desired depth of the soil strata. If $\tau_{av} > \tau_Z$ then soil will liquefy at that zone.

3.1.1.5 Iwasaki et al. (1982) method

Iwasaki et al. (1982) proposed a simple geotechnical method as outlined in the Japanese Bridge Code (1991). In this method, soil liquefaction capacity factor R, is calculated along with a dynamic load L, induced in a soil element by the seismic motion. The ratio of both is defined as 'liquefaction resistance'. The soil liquefaction capacity is calculated by the three factors, which take into account the overburden pressure, grain size and fine content. In this method it is assumed that the severity of liquefaction should be proportional to the thickness of the liquefied layer, proximity of the liquefied layer to the surface, and the factor of safety of the liquefied layer.

The prediction by the liquefaction potential index is different than that made by the simplified procedure of Seed and Idriss (1971). According to Toprak and Holzer (2003), the simplified procedure predicts what will happen to a soil element whereas the index predicts the performance of the whole soil column and the consequences of liquefaction at the ground surface. Sonmez (2003) modified this method by accepting the threshold value of 1.2 of factor of safety as the limiting value between the categories of marginally liquefiable to non-liquefiable soil.

The NCEER workshops in 1996 and 1998 resulted in a number of suggested revisions to the SPT based procedure. Cetin et al. (2000) reexamined and expanded the SPT case history database. The data set by Seed et al. (1984) had 125 cases of liquefaction/ no liquefaction in 19 earthquakes, of which 65 cases pertain to sands with fines content ≤ 5%, 46 cases had fines content between 6 and 34% and 14 cases had ≥ 35%. Cetin et al. (2000) used their expanded data set and site response calculations for estimating CSR to develop revised relationships. Idriss and Boulanger (2004) presented a revised curve between CSR and modified SPT value based on the reexamination of the available field data.

3.1.2 CPT based method

The CPT test has become one of the most common and economical methods of subsurface exploration. The cone penetrometer is pushed into the ground at a standard velocity of 2 cm/sec and data is recorded at regular intervals (typically 2 or 5 cm) during penetration. The results provide excellent stratigraphic detail and repeatability provided proper care has been taken in calibration of the equipment (transducers and electronics). The cone penetrometer is instrumented to record a number of different parameters, with the most common being the force of the tip, the force of the sleeve, and the pore pressure behind the tip. Cone penetrometers have also been used to provide or measure electrical properties, shear wave velocities, visual images of the soil, acoustic emissions, temperature and water samples.

The CPT is a versatile sounding method that can be used to determine the materials in a soil profile and their engineering properties. The equipment consists of a 60^0 cone, with 10 cm^2 base area and a 150 cm^2 friction sleeves located above the cone. A sensor is attached for measuring tip resistance, pore pressure and sleeve resistance. To evaluate the potential for soil liquefaction it is important to determine soil stratification and in-situ soil state. The CPT is an ideal in-situ test to evaluate the potential for soil liquefaction because of its repeatability, reliability, continuous data and cost effectiveness.

3.1.2.1 Robertson and Wride (1998) method

A simplified method to estimate cyclic shear resistance (CSR) was developed by Seed and Idriss (1971) based on maximum ground acceleration at the site as under:

$$CSR = \tau_{av} / \sigma_0' = 0.65 \, (MWF) \, (\sigma_0 / \sigma_0') \, (a_{max} / g) \, r_d \tag{10}$$

$$MWF = (M)^{2.56} / 173 \tag{11}$$

where, MWF is the magnitude weighting factor and M is the earthquake magnitude, commonly M = 7.5

Seed et al. (1985) also developed a method to estimate the cyclic resistance ration (CRR) for clean sands and silty sands based on the CPT using normalized penetration resistance.

The cone penetration resistance q_c can be normalized as

$$q_{c1N} = C_Q (q_c / p_a) \tag{12}$$

$$C_Q = (P_a / \sigma_0')^n \tag{13}$$

where, C_Q is normalized factor for cone penetration resistance, P_a is the atmosphere of pressure in the same units as σ_0'and n ia an exponent that varies with soil type (= 0.5 for sands and 1 for clays) and q_c is the field cone penetration resistance at tip. The normalized penetration resistance (q_{c1N}) for silty sands is corrected to an equivalent clean sand value $(q_{c1N})_{CS}$ as

$$(q_{c1N})_{CS} = K_C \, q_{c1N} \tag{14}$$

where, K_C is the correction factor for grain characteristics and is defined as below by Robertson and Wride (1998).

$$K_C = 1.0 \qquad \text{for } I_C \leq 1.64 \tag{15}$$

$$K_C = -0.403 I_C^4 + 5.581 I_C^3 - 21.63 I_C^2 + 33.75 I_C - 17.88 \quad \text{for } I_C > 1.64 \tag{16}$$

If $I_C > 2.6$, the soil in this range are likely to clay rich or plastic to liquefy. I_C is the soil behavior type index and is calculated as

$$I_C = [(3.47 - \log Q)^2 + (1.22 + \log F)^2]^{0.5} \tag{17}$$

where Q is normalized penetration resistance

$$= [(q_c - \sigma_0)/P_a][P_a/\sigma_0']^n \tag{18}$$

$$F = [f_s / (q_c - \sigma_0)] * 100\% \tag{19}$$

where f_s being the sleeve friction stress

$$CRR_{7.5} = 0.833[\frac{(q_{c1N})_{cs}}{1000}] + 0.05 \qquad \text{if } (q_{c1N})_{cs} < 50 \tag{20}$$

$$CRR_{7.5} = 93[\frac{(q_{c1N})_{cs}}{1000}]^3 + 0.08 \qquad \text{if } 50 \leq (q_{c1N})_{cs} < 160 \tag{21}$$

where, $(q_{c1N})_{cs}$ is clean sand cone penetration resistance normalized to approximately 100 kPa (1atm). Then, using the equivalent clean sand normalized penetration resistance $(q_{c1N})_{cs}$, CRR can be estimated from the Fig. 2.

The CPT based liquefcation correlation was reevaluated by Idriss and Boulanger (2004) using case history data compiled by Shibata and Teparaksa (1988), Kayen et al. (1992), Boulanger (2003) and Moss (2003).

Moss (2003) has provided a most comprehensive compilation of field data and associated interpretations. He used friction ratio R_f instead of the parameter I I_C, soil behavior type index and examined for the cohesion less soils with fines content greater than or equal to 35%.