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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 26<sup>th</sup> 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 loses its shear strength and it can not sustain 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.

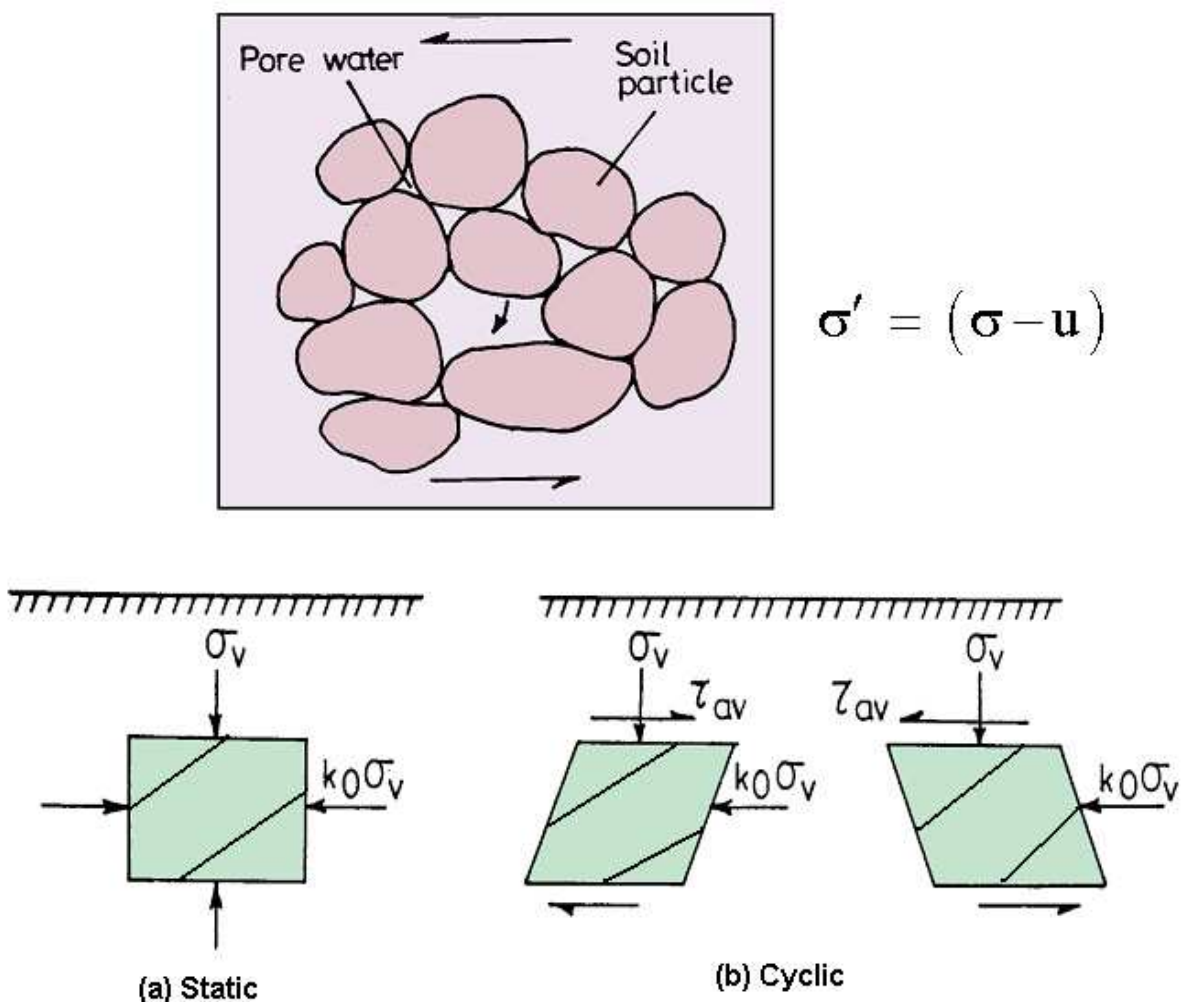


Fig. 1. Mechanism of Soil Liquefaction

## 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

$$\tau_s = \sigma' \tan \phi \quad (1)$$

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

$$\tau_s = (\sigma - u) \tan \phi \quad (2)$$

where,  $\tau_s$  is shear resistance,  $\sigma'$  is the effective normal stress,  $\sigma$  is the total normal stress,  $u$  is the pore water pressure and  $\phi'$  is the angle of internal friction in terms of effective stress.

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

$$q = \frac{1}{2} (\sigma_1 - \sigma_3) = \frac{1}{2} (\sigma_1' - \sigma_3') \quad (3)$$

$$p = \frac{1}{2} (\sigma_1 + \sigma_3) \quad (4)$$

$$p' = \frac{1}{2} (\sigma_1' + \sigma_3') \quad (5)$$

where,  $\sigma_1$  and  $\sigma_3$  are maximum and minimum total principal stress,  $\sigma_1'$  and  $\sigma_3'$  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 exceeds 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 pore 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 gravelly soils are potentially vulnerable to liquefaction. Most gravelly 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, gravelly 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 (1kg/cm<sup>2</sup>) as the reference overburden pressure and the N blows corrected for this pressure are called the normalized or corrected values, N<sub>c</sub>.

$$N_c = C_n * N_r \quad (6)$$

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

where, N<sub>r</sub> = Observed N value in the field and C<sub>n</sub> = Correction factor

The another simple relation for the correction factor C<sub>n</sub> that greatly cover more research work on the correction factor carried in the USA was given in Eqn 8 as below.

$$C_n = \sqrt{100 / \sigma'} \quad (8)$$

where,  $\sigma'$  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

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

where,  $N_c$  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  $\tau_{av}$  will be computed same as in Seed and Idriss method. Using corrected SPT 'N' value and the proposed chart by Seed and Peacock,  $\tau_z$  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  $\leq 5\%$ , 46 cases had fines content between 6 and 34% and 14 cases had  $\geq 35\%$ . 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° cone, with 10 cm<sup>2</sup> base area and a 150 cm<sup>2</sup> 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:

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

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

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 ratio (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

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

$$C_Q = (P_a / \sigma_0')^n \quad (13)$$

where,  $C_Q$  is normalized factor for cone penetration resistance,  $P_a$  is the atmosphere of pressure in the same units as  $\sigma_0'$  and  $n$  is 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 ( $q_{c1N}$ )<sub>CS</sub> as

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

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

$$K_C = 1.0 \quad \text{for } I_C \leq 1.64 \quad (15)$$

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

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

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

where  $Q$  is normalized penetration resistance

$$Q = [(q_c - \sigma_0) / P_a] [P_a / \sigma_0']^n \quad (18)$$

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

where  $f_s$  being the sleeve friction stress

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

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

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 liquefaction 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  $I_C$ , soil behavior type index and examined for the cohesion less soils with fines content greater than or equal to 35%.

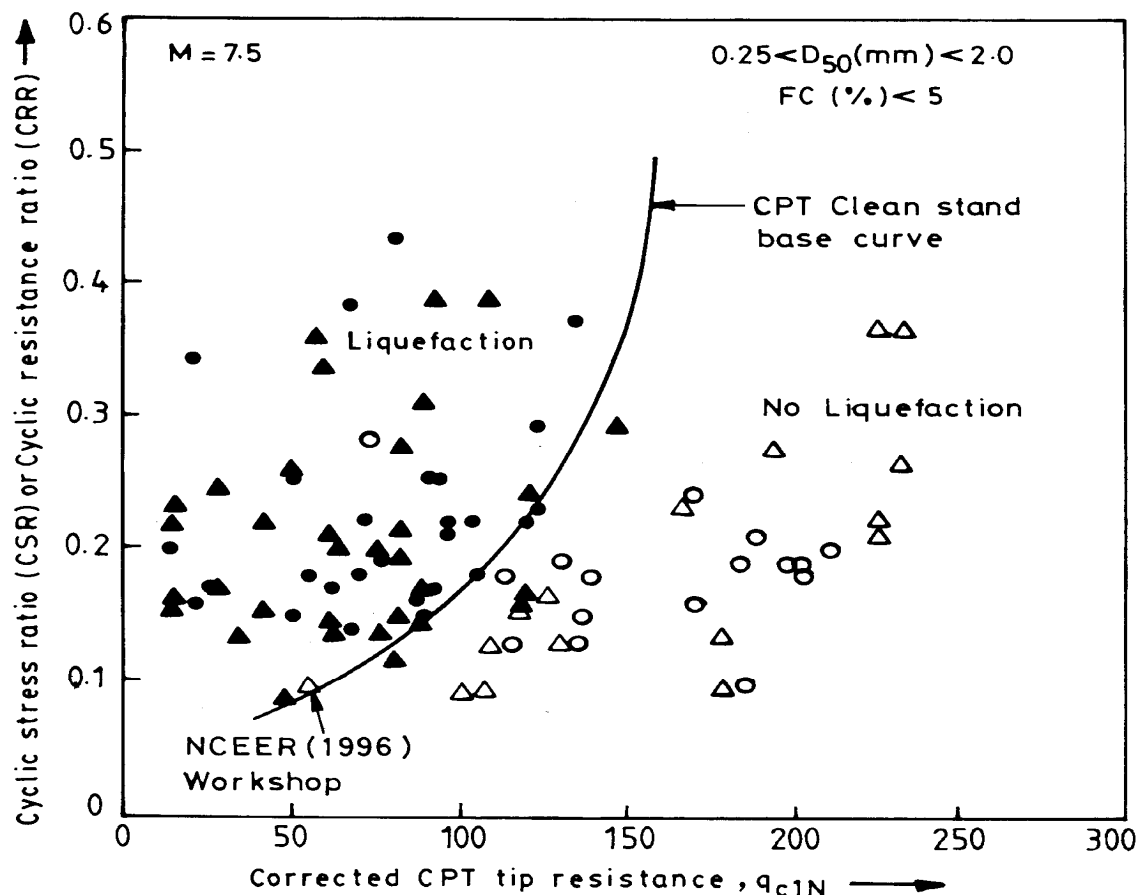


Fig. 2. Calculation of CRR from CPT  $q_{c1N}$  (Youd et al., 2001)

### 3.1.3 Shear wave velocity ( $V_s$ ) based methods

The shear wave velocity based procedure has advanced significantly in recent years, with improved correlations and more databases as summarized by Andrus and Stokoe (2000). Shear wave velocity can be determined either by subsurface geophysical method or by surface geophysical method as explained earlier. Liquefaction potential can be evaluated from the shear wave velocity ( $V_s$ ) using the following three methods. These procedures can be useful particularly for sites where it is difficult to penetrate or sample soils.

#### 3.1.3.1 Andrus and Stokoe (2000) method

Andrus and Stokoe have carried extensive research into the use of shear wave velocity as an index of liquefaction resistance. Various investigators have developed relationships between shear wave velocity and liquefaction resistance. This section presents detailed guidelines for applying the procedure described in Andrus and Stokoe that was developed using suggestions from two workshops and following the general format of the Seed and Idriss (1971) simplified procedure. Andrus and Stokoe (2000) developed liquefaction resistance criteria from field measurements of shear wave velocity  $V_s$ . This method forms the basis for the currently accepted shear wave velocity criteria for liquefaction potential assessment. Liquefaction hazard assessment based on this method is done and a liquefaction hazard map of Delhi region hazard map is prepared.

Shear wave velocity  $V_s$  is corrected similar to SPT 'N' value using the atmospheric pressure  $P_a$  and initial effective vertical stress,  $\sigma_0'$ . The cyclic resistance ratio (CRR) is determined empirically at various depths using the correlation developed between CRR and the shear wave velocity for the liquefaction assessment. The detailed analysis procedure is outlined in the later chapter.

### 3.1.3.2 Hatanaka, Uchida and Ohara (1997) method

Hatanaka et al. (1997) performed a systematic research relating the undrained cyclic shear strength of high quality undisturbed gravel samples to the shear wave velocity measured insitu  $V_{S1}$  is used for correcting the effect of effective confining stress on  $V_s$  by using the following equation.

$$V_{S1} = \frac{V_s}{\left(\frac{\sigma_v'}{98}\right)^{3/8}} \quad (22)$$

The value of 3/8 in the above equation is the average value of 0.5 and 0.25 which covered the test result. It is also important to know the  $K_O$  value for converting the liquefaction strength obtained in laboratory  $R_{LAB}$  to that in the field  $R_{INSITU}$  based on the following equation

$$R_{INSITU} = 0.9 \left\{ \frac{(1 + 2K_o)}{3} \right\} R_{LAB} \quad (23)$$

The variation of shear wave velocity with confining pressure for various soil conditions was given by Hardin (1963).

### 3.1.3.3 Tokimatsu, Yamazaki, and Yoshimi (1986) method

This method is proposed by Tokimatsu et al. (1986). The working principle in this procedure is that the liquefaction strength has a good correlation with elastic shear modulus for a given soil under a given confining pressure. The procedure is outlined in Fig. 3. The procedure shown on the left corresponds to the shear wave velocity measurement in-situ. Based on the measured shear wave velocity,  $V_s$  the elastic shear modulus in the field  $G_{OF}$  can readily be determined. The procedure on the right involves laboratory tests on a specimen reconstituted from the sample obtained at the site. Both liquefaction test, the shear modulus at small shear strain  $G_{OL}$ , of the specimen is measured and compared with  $G_{OF}$ . If they are equal, the liquefaction test has to be done on the same specimen. If they are not equal that usually means that  $G_{OF}$  is larger than  $G_{OL}$ , cyclic shear stresses are applied to the specimen until the shear modulus reaches the value; then the liquefaction test is conducted. In order to take the effects of void ratio ( $e$ ) and confining stress ( $\sigma_c'$ ) on the  $G_O$  is normalized with respect to  $e$  and  $\sigma_c'$  as described as

$$G_O = \frac{G_o}{\left\{ F(e_{min})(\sigma')^n \right\}} \quad (24)$$

$$F(e_{min}) = (2.17 - e_{min})^2 (1 + e_{min}) \quad (25)$$

where,  $n$  is a factor whose value equals to 2/3.

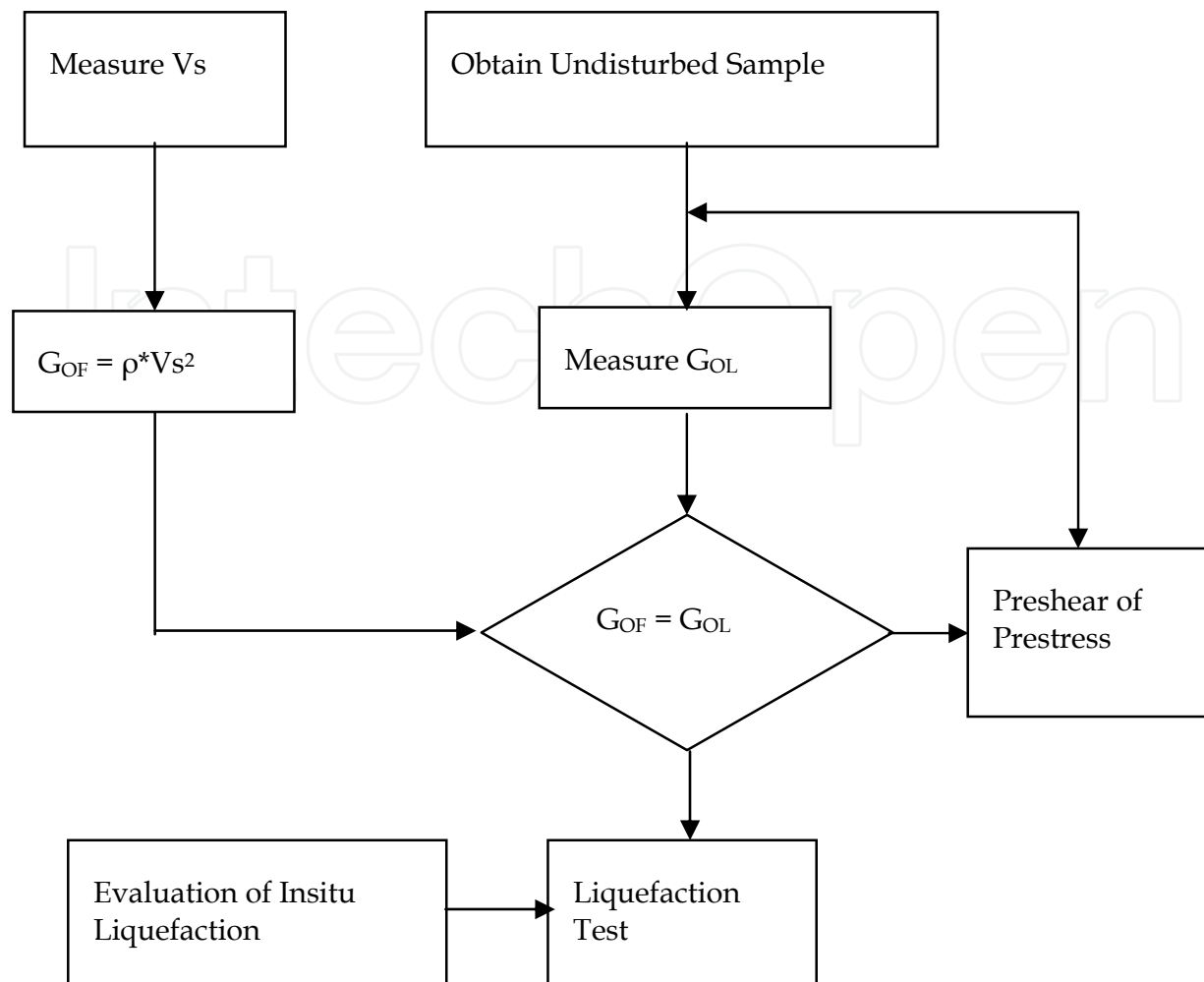


Fig. 3. Outline of Method (Tokimatsu et al., 1986)

In this method, it is important to determine  $G_{OL}$  from laboratory test.  $G_{OL}$  value highly depends on the confining stress at laboratory. As a result it is an important work to reasonably estimate the  $k_0$  value of insitu soils.

Tsurumaki et al. (2003) verified the simplified procedure using new data for different kinds of gravelly soils by undrained cyclic triaxial tests on undisturbed sample of gravelly soils obtained by the in-situ freezing sampling method. The undrained cyclic strength increases with increasing shear wave velocity.

### 3.2 Laboratory methods

#### 3.2.1 Cyclic triaxial test

The cyclic triaxial test is the most commonly used test for the measurement of dynamic soil properties at high strain levels. This test simulates the liquefaction phenomenon during earthquakes by applying cyclic shear to the saturated sandy soil under undrained condition. Axial loading is applied in steps to the specimen and the shear strain and shear stress at various levels of loading. Five to ten levels of shear strain amplitudes are chosen from the range of  $10^{-5}$  to  $10^{-2}$  for testing. Dynamic deformation characteristics are influenced by the



effective confining pressure during the test. When an undisturbed sample of normally consolidated soil is obtained, the effective vertical pressure at the depth of sampling is isotropically applied by cell pressure to avoid the influence of over consolidation. In order to obtain in-situ shear modulus of the soil from the laboratory test results, correction of these results is necessary so that a shear modulus corresponding to the average effective principal stress at the sample depth is obtained.

Seed and Lee (1966) were the first to reproduce liquefaction in a cyclic load triaxial test on loose and dense sands and concluded that liquefaction occurs more easily in sandy soils having higher void ratios and void ratio remaining constant, lower the effective confining pressure higher the liquefaction susceptibility.

### **3.2.2 Cyclic direct simple shear test**

The cyclic direct simple shear test is capable of reproducing earthquake stress conditions much more accurately than the cyclic triaxial test. It is most commonly used for liquefaction testing. In this test, a short cylindrical specimen is restrained against lateral expansion by rigid boundary platens, a wire reinforced membrane or with a series of stacked rings. By applying cyclic horizontal shear stresses to the top or bottom of the specimen, the test sample is deformed in the same way as an element of soil subjected to vertically propagating S waves. In recent years, simple shear devices that allow independent control of vertical and horizontal stresses have been developed. To better simulate actual earthquake conditions, Pyke (1979) used a large-scale simple shear apparatus. It is concluded that cyclic strength is related to the relative density of the soil and cyclic stresses that cause liquefaction in simple shear were less than those causing liquefaction in triaxial shear.

### **3.2.3 Cyclic torsional shear test**

Many of the difficulties with cyclic triaxial and cyclic shear test can be overcome with cyclic torsional shear test. This is mostly used to determine stiffness and damping characteristics over a wide range of strain levels. It allow isotropic or an isotropic initial stress conditions and can impose cyclic shear stresses on horizontal planes with continuous rotation of principal axes. Dobry et al.(1995) used strain controlled cyclic torsional loading along with stress controlled axial loading of solid specimens and has proven effective for measurement of liquefaction behavior. Torsional testing of solid specimens, however produces shear stresses that range from zero along the axis of the specimen to a maximum value at the outer edge. To increase the radial uniformity of shear strains, a hollow cylindrical cyclic torsional shear apparatus were also developed. While hollow cylinder tests offer perhaps the best uniformity and control over stresses and drainage. Ishihara and Li (1972) developed a torsional triaxial shear test and conducted strain controlled tests on solid cylinders of saturated sands. These tests helped in establishing relationship between cyclic triaxial tests, cyclic simple shear tests and the torsional triaxial test.

### **3.2.4 Shake table test**

Shake table tests of many sizes are being used for liquefaction studies on saturated soil samples prepared in a container, fixed to a shaking platform and vibrated at the desired

frequency for a prescribed time. A surcharge is placed on the sample to provide the confining pressure. The measurements of acceleration, pore water pressure and settlements are made during the test. Shaking tables utilize a single horizontal translation degree of freedom, but shake table with multiple degrees of freedom have also been developed. Kokusho (1987) developed a numerical model based on shake table test.

#### 4. Magnitude scaling factor

Seed and Idriss (1982) introduced a correction factor termed as magnitude scaling factor (MSF). This factor can be used to scale up or scale down the CRR based curves upward or downward, depending upon the earthquake magnitude. Figure 4 gives the curves proposed by various authors for different earthquake magnitudes.

##### 4.1 Seed and Idriss (1982) scaling factor

Seed and Idriss (1982) developed a set of MSF from average number of loading cycles for various earthquake magnitude and laboratory test results. These MSF have been routinely applied in engineering practice.

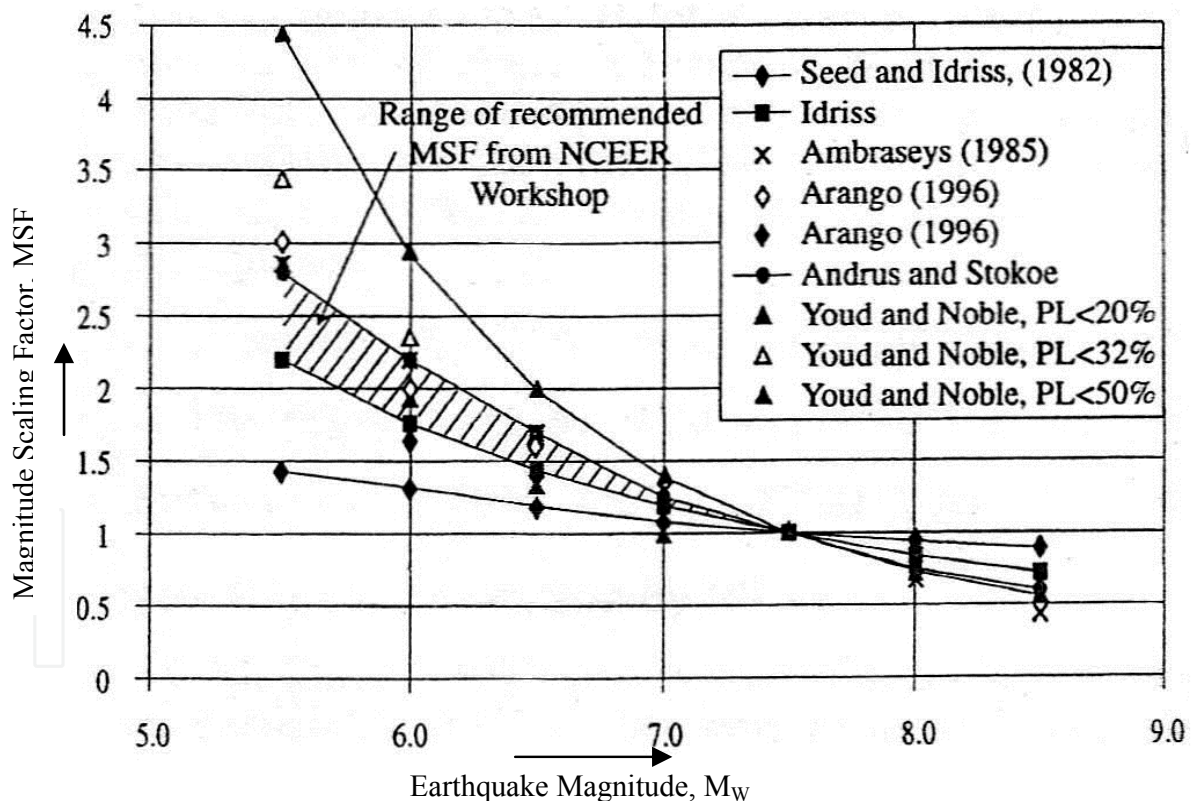


Fig. 4. Magnitude Scaling Factor Derived by Various Investigations (Youd et al., 2001)

Idriss (1995) developed a revised set of magnitude scaling factor and was defined as

$$MSF = \left( \frac{10^{2.24}}{M_w^{2.56}} \right) \quad (26)$$

The revised scaling factors were higher than the original scaling factors for magnitude < 7.5 and somewhat lower than the original factors for magnitude > 7.5 relative to the original scaling factors the revised factors lead to a reduced calculated liquefaction hazard for magnitude < 7.5 but increase calculated hazard for magnitude > 7.5. National Center for Earthquake Engineering Research (NCEER, 2001) workshop participants suggested this revised scaling factors as lower bound for MSF values.

#### 4.2 Ambraseys (1988) scaling factor

Ambraseys (1988) analyzed the liquefaction data and calculated cyclic stress for sites that did or did not liquefy versus  $N_{60}$ . Based on this he developed an empirical equation that define CRR as a function of  $N_{60}$  and moment magnitude. For magnitude less than 7.5, MSF s suggested by Ambraseys are significantly larger than both the scaling factors developed by Seed and Idriss. For magnitudes > 7.5, Ambraseys factors are significantly lower and much more conservative. It is not recommended for hazard evaluation because of few data constrain.

#### 4.3 Andrus and Stokoe (1997) scaling factor

Andrus and Stokoe developed scaling factors by drawing bounding curves for sites where surface effects of liquefaction were or were not observed for earthquake magnitudes of 6, 6.5 and 7.0. MSF for magnitude < 6 and > 7.5 were extrapolated from the following equation

$$\text{MSF} = \left( \frac{M_w}{7.5} \right)^{-2.56} \quad (27)$$

For magnitude < 7.5 the MSF s proposed by Andrus and Stokoe are rather close in value to the MSFs proposed by Ambraseys. For magnitude > 7.5 Andrus and Stokoe MSFs are slightly smaller than the revised MSFs proposed by Idriss.

#### 4.4 Youd and Noble (1997) scaling factor

Youd and Noble (1997) used a probabilistic and logistic analysis to analyze case history data from sites was or were not reported following past earthquakes. They defined three sets of MSFs for different magnitude ranges and with different probabilities of liquefaction ( $P_L$ ) occurrence.

$$P_L < 20 \% \quad \text{MSF} = \left[ \frac{10^{3.81}}{M^{4.53}} \right] \quad \text{for } M_w < 7 \quad (28)$$

$$P_L < 32 \% \quad \text{MSF} = \left[ \frac{10^{3.74}}{M^{4.33}} \right] \quad \text{for } M_w < 7 \quad (29)$$

$$P_L < 50 \% \quad \text{MSF} = \left[ \frac{10^{4.21}}{M^{4.81}} \right] \quad \text{for } M_w < 7.75 \quad (30)$$

The NCEER (2001) workshop report provides a useful insight in to the choice of MSF. For magnitude < 7.5, the lower bound for the recommended range is the new MSF proposed by

Idriss. The suggested upper bound is the MSF proposed by Andrus and Stokoe (1997). The upper bound values are consistent with MSFs suggested by Ambraseys (1988) and Youd and Noble(1997a) for  $P_L < 20\%$ . For magnitude  $> 7.5$ , the new factors recommended by Idriss should be used. These new factors are smaller than the original Seed and Idriss (1982) factors. The relations by Ambraseys (1998) and Arango (1996) give significantly larger MSF values for earthquake magnitudes less than 7. Table 1 gives the MSF values given by various researchers.

Magnitude M	Seed and Idriss (1982)	Ambra- seys (1988)	Arango (1966)		Andrus and Stokoe (1997)	Youd and Noble (1997)		
			Distance Based	Energy Based		$P_L < 20\%$	$P_L < 32\%$	$P_L < 50\%$
5.5	1.43	2.86	3.00	2.20	2.80	2.86	3.42	4.44
6.0	1.32	2.20	2.00	1.65	2.10	1.93	2.35	2.92
6.5	1.19	1.69	1.60	1.40	1.60	1.34	1.66	1.99
7.0	1.08	1.30	1.25	1.10	1.25	1.00	1.20	1.39
7.5	1.00	1.00	1.00	1.00	1.00	---	---	1.00
8.0	0.94	0.67	0.75	0.85	0.80	---	---	0.73
8.5	0.89	0.44	---	---	0.65	---	---	0.56

Table 1. Magnitude Scaling Factors Suggested by Various Researchers (Youd and Noble,1997)

During the Niigata earthquake ( $M=7.5$ ) on June 16, 1964, widespread liquefaction was observed in the low-lying areas. The liquefaction was accompanied by foundation failure and failure of retaining structures. It is important to note, however, that the structures themselves suffered very little damage; they essentially settled and rotated as rigid bodies under the loss of bearing capacity. Several of the apartment buildings were later jacked back to a vertical position and underpinned with new foundations. Nearly 20 years after the earthquake, excavation beneath the building, as part of the construction of upgraded foundations for an increase in the height of the building, revealed that the piles had been extensively damaged during the Niigata earthquake. Large displacements of the buildings were driven by gravity and resulted in performance that would be considered as "failure" by almost any definition, even though the structural elements of the buildings were virtually undamaged. In the 1964 Niigata earthquake, a number of RC buildings settled and tilted due to a loss of bearing capacity.

During the 1990 Luzon earthquake, substantial building damage occurred in 2 to 4 story buildings resting on liquefied ground in Dagupan City. During the 1999 Kocaeli earthquake in Turkey, settlement or tilting of buildings due to liquefaction was widespread in Adapazari. Although liquefaction in this case was less severe than in Niigata probably because the soil contained a lot of fines, many 5 to 6 story buildings suffered heavy structural damage due to uneven settlement of at most 2 m. Riverbanks, levees, and dikes suffered liquefaction induced settlement and sliding failures during the 1993 Kushiro-Oki earthquake, the 2003 Tokachi-Oki earthquake and many other earthquakes. During the 2003

Tokachi-Oki earthquake, the right banks of the Tokachi River suffered severe damage due to liquefaction-induced deep sliding and crest settlement. Another failure mode is the uplift of subsurface structures such as pipelines, buried tanks, pools, etc. Concrete manholes and pipes of sewage systems are raised because of liquefaction in surrounding soils during the 1993 Kushiro-Oki earthquake, the 2003 Tokachi-Oki earthquake, and the 2004 Niigata ken-Chuetsu earthquake. Although surrounding ground appeared stable during earthquakes, sands backfilling buried structures liquefied and buoyant pressure in the liquefied fill raised them over 1.5 m. The detailed liquefaction assessment requires several inputs regarding the site specific geological, geophysical, geotechnical, seismotectonic, ground motion parameters and their effects on the structures considering their design aspects, local soil conditions which would enhance the earthquake effects like soil amplification, liquefaction of soils etc.

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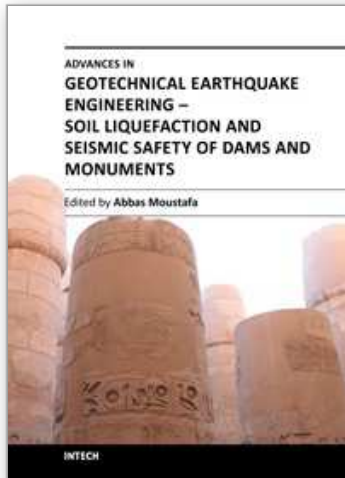
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**Advances in Geotechnical Earthquake Engineering - Soil Liquefaction and Seismic Safety of Dams and Monuments**

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This book sheds lights on recent advances in Geotechnical Earthquake Engineering with special emphasis on soil liquefaction, soil-structure interaction, seismic safety of dams and underground monuments, mitigation strategies against landslide and fire whirlwind resulting from earthquakes and vibration of a layered rotating plant and Bryan's effect. The book contains sixteen chapters covering several interesting research topics written by researchers and experts from several countries. The research reported in this book is useful to graduate students and researchers working in the fields of structural and earthquake engineering. The book will also be of considerable help to civil engineers working on construction and repair of engineering structures, such as buildings, roads, dams and monuments.

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