Assessment of Liquefaction Potential of Cohesionless Soil by Semi-Empirical: SPT- Based Procedure

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Abstract - Liquefaction is one of the critical problems in the field of Geotechnical engineering. It is the phenomena when there is loss of shear strength in saturated and cohesion-less soils because of increased pore water pressures and hence reduced effective stresses due to dynamic loading. 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. The strength of semi-empirical procedure is the use of both experimental findings together with the theoretical considerations for establishing the framework of the analysis procedure.

Keywords -Liquefaction, assessment, SPT, CSR, CRR, MSF

I. INTRODUCTION
The term “liquefaction” has been first used by Terzaghi and Peck (1948) [1] to describe the significant loss of strength of very loose sands causing flow failures due to slight disturbance. Similarly, Mogami and Kubo (1953) [2] used the same term to define shear strength loss due to seismically-induced cyclic loading. However, its importance has not been fully understood until 1964 Niigata earthquake, during which the significant causes of structural damage were reported. The liquefaction characteristic of a soil depends on several factors, such as ground acceleration, grain size distribution, soil density, thickness of the deposits and especially the position of the ground-water table. Liquefaction and ground failures are commonly associated with large earthquakes. 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. The strength of semi-empirical procedure is the use of both experimental findings together with the theoretical considerations for establishing the framework of the analysis procedure.

II. MECHANISM OF LIQUEFACTION
Robertson and Wride (1997) [3] reported that as an engineering term, “liquefaction” has been used to define two mainly related but different soil responses during earthquakes: flow liquefaction and cyclic softening. Since both mechanisms can lead to quite similar consequences, it is difficult to distinguish. However, the mechanisms are rather different, as discussed below:

2.1 Flow liquefaction: It is the phenomenon in which the static equilibrium is destroyed by static or dynamic loads in a soil deposit with low residual strength (strength of liquefied soil). It occurs when the static shear stress in the soil exceeds the shear strength of liquefied soil. This will cause large deformation in soils. Flow liquefaction can be triggered by either monotonic or cyclic loading.

Fig.1 Flow Liquefaction

Earthquakes, blasting and pile driving are all examples of dynamic loads that could trigger flow liquefaction. Once triggered the strength of the soil susceptible to flow liquefaction is no longer sufficient to withstand the static stresses that were acting on the soil before the disturbances.

2.2 Cyclic Softening: Cyclic softening is another phenomenon, triggered by cyclic loading, occurring in soil deposits with static shear stresses lower than the soil strength. Two main engineering terms can be used to define the cyclic softening phenomenon, which applies to both strain softening and strain hardening materials.
2.2.1 Cyclic Mobility: It is the liquefaction phenomenon triggered due to cyclic loading occurring in the soil deposit. Cyclic mobility develops incrementally because of static and dynamic stresses that exist during an earthquake. It can occur in almost any sand provided that the cyclic loading is sufficiently large in size and duration, but no shear stress reversals occurs, and clayey soils can experience cyclic mobility, but deformations are usually controlled by rate effects (creep).

Cyclic mobility mechanism is illustrated as shown in Figure 2. Figure on the left shows the variation of shear stress during cyclic loading and the figure on the right is the development of the shear strain during this loading. As this figure implies, no zero effective stress develop during cyclic loading.

2.2.2. Cyclic Liquefaction: It occurs when a initial static shear stress is exceeded by cyclic shear stresses to produce a stress reversal. It can occur in almost all sands provided that the cyclic loading is sufficiently large in size and duration. Clayey soils can experience cyclic liquefaction but deformations are generally small due to cohesive strength at zero effective stress. Cyclic liquefaction mechanism is illustrated as shown in Figure 3. The figure on the left shows the variation of stress state during cyclic loading, whereas the figure on the right illustrates modulus degradation. As the figures imply, zero effective stress state develops and thus results in zero shear strength for non cohesive soils. Strains (deformations) during cyclic loading may reach to higher values as presented in the right figure.

III. POTENTIALLY LIQUEFIABLE SOILS

For the assessment of liquefaction triggering potential, different criteria’s are developed by researchers and are discussed below:

Chinese criteria: Chinese criteria proposed by Seed and Idriss (1982) [4] is widely used for many years and is summarized below (Table 1)

<table>
<thead>
<tr>
<th>Potentially Liquefiable Soils</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Fines Content (&lt; 0.005 mm)</td>
<td>≤ 15%</td>
</tr>
<tr>
<td>Liquid Limit (LL)</td>
<td>≤ 35%</td>
</tr>
<tr>
<td>Water Content (w_c)</td>
<td>≥ (0.9xLL)%</td>
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Table 1: Chinese Criteria proposed by Seed and Idriss (1982)


Recent advances revealed that non-plastic fine grained soils can also liquefy and plasticity index is a major controlling factor in the cyclic response of fine grained soils. The criteria by Andrews and Martin (2000) [5] is summarized below (Table 2)

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<td>Clay Content (≥ 0.002 mm)</td>
<td>Liquid Limit ≥ 32%</td>
<td>Non-Liquefiable</td>
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Table 2: Modified Chinese Criteria by Andrews and Martin (2000)

Bray et al. (2001) has concluded that the Chinese criteria may be misleading in the concept of percent “clay-size”. According to their findings, percent of clay minerals and their activities are more important than the percent of “clay-size”. They give the example of fine quartz particles which may be smaller than 2 – 5 mm, but they me largely non-plastic and may be susceptible to liquefaction, behaving as a cohesionless material under cycling loading. Recommendations of Bray et al. (2001) [6] are presented in Figure 4.

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Fig.2 Cyclic Mobility

Fig.3 Cyclic Liquefaction

Fig.4 Potentially Liquefiable Soils (Bray et al., 2001)
Seed et al. (2003) [7] recommended a new criterion inspired from case histories and cyclic testing of “undisturbed” fine grained soils compiled after 1999 Kocaeli Turkey and Chi Chi-Taiwan earthquakes as shown in Figure 5. This criterion classify saturated soils with a plastic index (PI) less than 12 and liquid limit (LL) less than 37 as potentially liquefiable, provided that the soil natural moisture content is greater than 80% of the liquid limit.

Boulanger and Idriss (2004): Recommended the new criteria based on cyclic laboratory test results and an extensive engineering judgment. The deformation behavior of fine-grained soils are grouped as “Sand-Like” and “Clay-Like”, where soils within the sand-like behavior region are judged to be susceptible to liquefaction and have substantially lower values of Cyclic Resistance Ratio (CRR) [8]

**IV. FRAMEWORK USED FOR ASSESSMENT OF LIQUEFACTION POTENTIAL IN SOIL USING SEMI-EMPIRICAL METHODS:**

An overview for the framework used for evaluating liquefaction potential of Cohesionless Soil during earthquake is presented here:

**Cyclic Shear Stress Ratio (CSR):** Seed and Idriss (1971) [9] proposed cyclic stress ratio, CSR, which is defined as the average cyclic shear stress, $\tau_{av}$, developed on the horizontal plane of a soil layer due to vertically propagating shear waves normalized by the initial vertical effective stress, $\sigma'_v$, to incorporate the increase in shear strength due to increase in effective stress. The cyclic shear stress ratios (CSR) induced by earthquake ground motions, at a depth $z$ below the ground surface, using the following equation

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

where
- $a_{max} = $ maximum horizontal acceleration at the ground surface
- $\sigma_{vo} = $ total vertical stress
- $\sigma'_{vo} = $ effective vertical stress at depth
- $z = $ depth (m)
- $r_d = $ stress reduction coefficient that accounts for the flexibility of the soil column

To convert irregular forms of seismic shear stress time histories to a simpler equivalent series of uniform stress cycles, an averaging scheme is proposed by Seed and Idriss. The average shear stress is taken as 65% of the maximum shear, $\tau_{max}$.

$$\tau_{av} \approx 0.65 \cdot \gamma_n \cdot h \cdot a_{max} g \cdot r_d$$

where
- $\gamma_n = $ natural unit weight of soil
- $h = $ Soil depth (m)

**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 $M = 7\frac{1}{2}$, i.e.,
Accordingly, the values of (CSR)_{M,7.5} are given by:

\[
(CSR)_{M,7.5} = \frac{CSR}{MSF} = 0.65 \left(\frac{\sigma_{vo_{max}}}{\sigma_{v0}}\right) \frac{r_d}{MSF} \quad \text{Eqn. 2}
\]

where, MSF- Magnitude Scaling Factor

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 characteristics require that the effects of soil density and effective confining stress on penetration resistances be separated. Boulanger and Idriss (2004) [8]. Hence Seed et al. (1975a) [10] included the normalization of penetration resistances in sand to an equivalent of one atmosphere (1 Pa =1 tsf =101 kPa) as part of the semi-empirical procedure. This normalization currently takes the form equation

\[
(N_1)_{60} = C_N(N)_{60} \quad \text{Eqn. 3}
\]

\[
q_{c1} = C_N q_c \quad \text{Eqn. 4}
\]

where,

\[
(N)_{60} - \text{SPT ‘N’ value after correction to an equivalent 60\% hammer efficiency}
\]

\[
q_c - \text{Cone tip resistance}
\]

\[
C_N - \text{Overburden Correction Factor for penetration resistance}
\]

Stress reduction coefficient, \(r_d\):
The stress reduction coefficient \(r_d\) was introduced by Seed and Idriss (1971) [9] 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 \(r_d\) 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) [11] extended the work of Golesorkhi (1989) [12] and performed several hundred parametric site response analyses and concluded that for the conditions of most practical interest, the parameter \(r_d\) could be adequately expressed as a function of depth and earthquake magnitude (M). The following relation was derived using those results.

\[
\ln r_d = \alpha(z) + \beta(z)M \quad \text{Eqn. 5(a)}
\]

\[
\alpha(z) = -1.012 - 1.126 \sin \left(\frac{z}{11.73} + 5.133\right) \quad \text{Eqn. 5(b)}
\]

\[
\beta(z) = 0.106 + 0.118 \sin \left(\frac{z}{11.28} + 5.142\right) \quad \text{Eqn. 5(c)}
\]

where,

\[
z = \text{Depth (m) – up to 34m}
\]

For \(z > 34\) m following relation is used,

\[
r_d = 0.12 \exp(0.22M) \quad \text{Eqn. 5(d)}
\]

The plot of \(r_d\) vs Depth for different magnitudes of earthquake using equation 5 is presented below:

Magazine scaling factor, MSF:
The magnitude scaling factor, MSF, is used to adjust the induced CSR during earthquake magnitude \(M\) to an equivalent CSR for an earthquake magnitude, \(M = 7.5\)

\[
MSF = \frac{CSR_M}{CSR_{M,7.5}} \quad \text{Eqn. 6(a)}
\]

Idriss (1999) [11] 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) [13]. The MSF relation produced by this re-evaluation is given by:

\[
MSF = 6.9 \exp \left(-\frac{7.5}{4}\right) - 0.058 \quad \text{Eqn. 6(b)}
\]

where

\[
M = \text{Magnitude of the earthquake}
\]

The MSF should be less than equal to 1.8

Overburden correction factor, \(K_o\):
Boulanger and Idriss (2004) [14] found that overburden stress effects on the Cyclic Resistance Ratio (CRR). The recommended K curves are expressed as follows:

\[
K_o = 1 - C_o \ln \left(\frac{\sigma_{vo}}{\sigma_{vo}}\right) \leq 1.0 \quad \text{Eqn. 7(a)}
\]

The coefficient \(C_o\) is expressed in terms of \((N_1)_{60}\) or \(q_{c1N}\)

\[
C_o = \frac{1}{18.9-2.55 \sqrt{(N_1)_{60}}} \quad \text{Eqn. 7(b)}
\]
\[ C_\sigma = \frac{1}{37.3 - 0.27(q_{c1n})^{0.264}} \]  
\text{Eqn. 7(c)}

where,

\((N_1)_{60}\) and \(q_{c1n}\) are limited to maximum value of 37 and 211 respectively (i.e., keeping \(C_\sigma\) less than equal to 0.3)

**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. Thus, following the semi-empirical approach, the CSR and \((N_1)_{60}\) values can be recalculated using the revised \(r_d\), MSF, and \(K\) relations.

**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) [14]. It is given as follows:

\[
(CSR)_{M=7.5} = 0.65 \left( \frac{\sigma_{vo,max}}{\sigma_{vo}} \right) \frac{r_d}{MSF} K_d \]  
\text{Eqn. 8}

**Evaluation of CRR**

For the CRR value, at first the SPT penetration resistance was adjusted by Boulanger and Idriss (2004) [14] to an equivalent clean sand value:

\[(N_1)_{60cs} = (N_1)_{60} + \Delta(N_1)_{60} \]  
\text{Eqn. 9(a)}

\[
\Delta(N_1)_{60} = \exp \left( 1.63 + \frac{9.7}{FC} - \left( \frac{15.7}{FC} \right)^2 \right) \]  
\text{Eqn. 9(b)}

where,

\(FC = \) fine contents

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:

\[
CRR = \exp \left( \frac{(N_1)_{60cs}}{14.1} + \left( \frac{(N_1)_{60cs}}{126} \right)^2 - \left( \frac{(N_1)_{60cs}}{23.6} \right)^3 \right.
\]

\[
+ \left( \frac{(N_1)_{60cs}}{25.4} \right)^4 - 2.8 \right) \]  
\text{Eqn. 9(c)}

**Liquefaction Triggering Possibility**

A comparison of the level of CSR as the load (demand) term and cyclic resistance ratio, CRR as the capacity term helps to conclude about liquefaction triggering possibility.

Cetin et al. (2004) [15] introduced new chart solutions similar to Seed et al. (1984a)’s deterministic curves for soil liquefaction triggering by using higher-order probabilistic tools. Moreover, these new correlations were based on a significantly extended database and improved knowledge on standard penetration test, site specific earthquake ground motions and in-situ cyclic stress ratios. These charts can be seen in Figure 10 and Figure 11. For comparison purposes Seed et al (1984a)’s deterministic boundary is also shown on the figures. Curves in Figure 9 through Figure 11 represent the resistance of soils to liquefaction referred by cyclic resistance ratio, (CRR). Knowing the demand term, CSR (estimated by simplified procedure) and the capacity term, CRR (from the figures or empirical correlations) it is easy to determine if the soil body in concern will liquefy or not, i.e. if CSR > CRR, the soil is concluded to liquefy and vice versa.

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**Fig.9** Liquefaction boundary curves recommended by Seed et al (1984a)
Fig. 10  Recommended probabilistic SPT-based liquefaction triggering correlation for $M_w=7.5$ and $\sigma_v=1.0$ atm. (Cetin et al. 2004)

Fig. 11  Deterministic SPT-based liquefaction triggering correlation for $M_w=7.5$ and $\sigma_v=1.0$ atm. with adjustments for fines content. (Cetin et al. 2004)

IV. CONCLUSIONS

The semi–empirical field based procedures used to evaluate the liquefaction potential of cohesionless soil are widely accepted and used due to the fact that it is based on actual case histories. Due to the high sensitivity of residual shear strength to small variations of void ratio and difficulties in simulating field stress and loading conditions, laboratory-based techniques are not widely used in engineering analysis.

REFERENCES


