ABSTRACT: One of the most devastating effects of earthquakes is the seismic soil liquefaction. Many research works have been done in this area and at present different methods are available to evaluate the assessment of liquefaction potential. In this paper an overview of different methods to evaluate liquefaction potential are presented. Further, in this paper emphasis has been given for the evaluation of liquefaction initiation based on the field tests such as Standard Penetration Test (SPT), Cone Penetration Test (CPT) and Shear wave velocity ($V_s$) test data. The probabilistic liquefaction evaluation method, which accounts for the uncertainty in the earthquake loading, is also discussed and presented in detail. Case studies of liquefaction potential evaluation were done for the city of Bangalore, where geotechnical data from SPT and Shear wave velocity were available, and for south India (using NEHRP site classes) based on the probabilistic performance based approach. The probabilistic liquefaction analysis of Bangalore shows that most of the study area is safe against liquefaction. From the analyses, the spatial variation of SPT and CPT values required to prevent liquefaction for the entire south India are also presented.

1. INTRODUCTION
The term liquefaction can be defined, in a broad manner, as the loss of strength of saturated sands due to the sudden increase in pore water pressure due to dynamic loading. The devastating effects of liquefaction were observed during the Niigata and Alaska earthquakes in 1964. Many researchers have worked on identifying the liquefaction susceptible areas and also on the different methods to mitigate and prevent liquefaction. One of the first and the most widely used method to quantify the liquefaction resistance of soils is the simplified procedure developed by Seed & Idriss (1971). Over the past four decades, the liquefaction potential evaluation has undergone lots of changes and modifications. The NCEER workshop (Youd et al. 2001) recommends the use of $CRR$ to denote the soil resistance against liquefaction and this notation is being followed in this paper. This paper presents an overview of different liquefaction potential evaluation methods which are currently being used.

2. LIQUEFACTION SUSCEPTIBILITY BASED ON SOIL PROPERTIES
The susceptibility of different types soil against liquefaction can be identified based on different soil properties. The important soil properties which will affect the liquefaction potential are briefly discussed in the following sections.

2.1 Age of the Soil
The age of the soil deposits is an important factor in assessing the liquefaction susceptibility of the soil deposit. The older soil deposits are less susceptible to liquefaction because of the stable arrangement of the particles and better inter particle bond.

2.2 Index Properties of Soil
Liquefaction is mainly observed in fine to medium grained cohesion less sands. However now it has been concluded that gravels may also undergo liquefaction. The results of the lab experiments have shown that majority of the clays are not susceptible to liquefaction. Based on the findings of Andrews & Martin (2000), the following types of clays are susceptible to liquefaction (Table 1).

2.3 Shape of Particle
Rounded particles will get densified more easily than the particles with angular shapes. The sudden dynamic earthquake load will increase the density of the soil with rounded particle and this will lead to the increase in the pore water pressure. Hence the soils with rounded particles are...
more susceptible to seismic soil liquefaction than the soils with angular particles.

Table 1: Liquefaction Susceptibility of Silty and Clayey Sands (Andrew & Martins 2000)

<table>
<thead>
<tr>
<th>Clay content</th>
<th>Liquid Limit &lt; 32</th>
<th>Liquid Limit ≥ 32</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 10 %</td>
<td>Susceptible</td>
<td>Further studies required (Considering plastic non-clay sized grains such as Mica)</td>
</tr>
<tr>
<td>&gt; 10 %</td>
<td>Further studies required (Considering plastic non-clay sized grains such as Mine and quarry tailings)</td>
<td>Not Susceptible</td>
</tr>
</tbody>
</table>

2.4 Relative Density (rd) of Soil

During an earthquake the loose soil tend to contract (densify) and this will increase the pore water pressure, which may lead to liquefaction of the soil. Where as the dense sand will dilate when it is subjected to a cyclic loading and this will reduce the pore water pressure and decrease the liquefaction susceptibility. Hence the soils with lower relative density (rd) are more susceptible to liquefaction than the soil with higher relative density.

2.5 Permeability of Soil

The ease with which the excess pore pressure gets dissipated depends on the permeability of the soil. Soils with higher permeability will be less susceptible to liquefaction when compared to similar soils with lower permeability. When a loose soil stratum is surrounded by a high permeable soil layer, then its liquefaction susceptibility will be less due to the high permeability of the surrounding layer.

3. EVALUATION OF LIQUEFACTION TRIGGERING POTENTIAL

The initiation or triggering of liquefaction can be evaluated using two methods: (i) based on the laboratory testing of undisturbed sample (ii) using the empirical correlation available with various in situ tests. Since the tests involving undisturbed samples are extremely difficult, most of the liquefaction triggering studies are based on in situ tests. The important in situ tests used in liquefaction evaluation are Standard Penetration Test (SPT), Cone Penetration Test (CPT) and based on the measurement of shear wave velocity in the soil overlying the bed rock (V_s). The main emphasis in this paper will be given to evaluation of liquefaction potential based on SPT and CPT values. A preliminary assessment of liquefaction potential can be had from the following indices (Rao & Neelima 2007).

- Mean grain size, D_{50} = 0.02 to 1.0 mm
- Fines content < 10%
- Uniformity coefficient (D_{60}/D_{10}) < 10
- Relative density < 75%
- Plasticity index, PI < 10

3.1 Laboratory Test Methods

The important laboratory tests used to study the initiation of soil liquefaction are cyclic triaxial test, cyclic simple shear test, cyclic torsional shear test and shake table test. Cyclic triaxial test is one of the common tests done to measure the dynamic properties of the soil at higher strain levels. In this test the earthquake loading is simulated by applying cyclic shear stress to the sample. However the direction of the principal stress axes remains vertical and horizontal and hence they do not simulate the true seismic loading conditions. The cyclic simple shear test is capable of simulating the earthquake loading conditions more accurately than cyclic triaxial test. In this test the deformation of the specimen will be similar to that of a soil element under the influence of S waves. The most effective test to determine the damping characteristics of soil over a wide range of strain levels is through cyclic torsional shear test. The construction of scaled models on a shake table and subjecting them to the expected ground motion levels will help in studying the liquefaction susceptibility of these models.

3.2 In situ Test Methods

The in situ test methods discussed in this section are based on SPT, CPT and shear wave velocity values.

3.2.1 Evaluation of Liquefaction Potential Based on SPT Values

(i) Deterministic Approach: The method suggested by Seed and Idriss (1971) in the “simplified method” to evaluate the CSR values is,

\[ CSR = 0.65 \frac{a_{max}}{g} \frac{\sigma_{vo}}{\sigma_{vo}} \frac{r_d}{MSF} \]  \hspace{1cm} (1)

Where \( a_{max} \) is the peak ground acceleration (at surface level), \( \sigma_{vo} \) and \( \sigma_{vo} \) are the total and effective over burden pressure, \( r_d \) is the depth reduction factor and \( MSF \) is the magnitude scaling factor. The above relation was developed for an earthquake of magnitude \( M_w = 7.5 \) and if the magnitude of earthquake is different from this, it is being taken care off by the \( MSF \).

One of the most widely used deterministic approaches for evaluation of CRR values is proposed by Seed et al. (1985). This was further modified by Youd et al. (2001) and the curves between CRR and the corrected SPT values are given in Figure 1. In this work it was noted that the CRR value will increase with fines content.
The term $r_d$, used in the evaluation of CSR, is to account for the flexibility of the soil. There are many methods available for evaluating the $r_d$ value. The initial studies in this direction were done by Seed and Idriss (1971) and later on lots of relations were developed to predict the depth reduction factor. In a recent work Cetin and Seed (2004) has developed a new relation to evaluate the depth reduction factor. In this relation, the value of $r_d$ is taken as a function of depth, earthquake magnitude, ground acceleration and the soil stiffness.

$$
\begin{align*}
\frac{\left( -9.147 - 4.173a_{\text{max}} \right)}{1 + 0.652M_W} + \frac{\left( -9.147 - 4.173a_{\text{max}} \right)}{10.567 + 0.089e^{0.089(-d-7.760d_{\text{max}}+78.567)}}
\end{align*}
$$

(2)

Where, $a_{\text{max}}$ and $M_W$ correspond to the peak ground acceleration and moment magnitude values and $d$ is the depth and $\sigma_{r_d}$ is the standard error in the model.

![Fig. 1: Deterministic Cyclic Resistance Curves Proposed by Youd et al. 2001](image)

(ii) **Probabilistic Approach:** One of the first probabilistic approaches proposed for evaluation of liquefaction potential based on SPT values was developed by Liao et al. (1988). A recent and comprehensive work in this area was by Cetin et al. (2004). The probability of liquefaction can be evaluated using the procedure suggested by Cetin et al. (2004).

$$
\begin{align*}
P_L = \Phi \left[ \frac{(N_1)_{60}(1 + \theta FC) - \theta_2 \ln CSR_{eq}}{\theta_3 \ln MW - \theta_4 (\ln (\sigma_{\text{q0}} / P_a) + \theta FC) + \theta \sigma_e} \right]
\end{align*}
$$

(3)

Where $P_L$ – Probability of liquefaction (as a fraction); $F$ – standard normal cumulative distribution function; $(N_1)_{60}$ – corrected $N$ value; $FC$ – fineness content in percentage; $CSR_{eq}$ – cyclic stress ratio without MSF; $M_W$ – moment magnitude of earthquake $\sigma_{\text{q0}}$ – effective vertical pressure at the given depth; $P_a$ – atmospheric pressure (in the same unit as $\sigma_{\text{q0}}$); $\theta_1 - \theta_6$ - regression coefficients; $\sigma_e$ - model uncertainty. A probabilistic curve with a probability of 15% can be taken as an approximate equivalent of the deterministic liquefaction evaluation procedure by Youd et al. (2001).

(iii) **Probabilistic Performance Based Approach:** The evaluation of earthquake loading in liquefaction potential evaluation requires the quantification of the uncertainties in earthquake loading. Both the above methods, probabilistic and deterministic, use a single ground acceleration and earthquake magnitude. The results obtained from the PSHA analysis shows that several magnitudes contribute towards the ground acceleration and their percentage of contribution also varies. This is clear from a seismic hazard curves given in Figure 2. From this figure it is clear that it won’t be fair to come to the conclusion that a particular ground acceleration was created by a particular magnitude, instead it is being contributed by different magnitudes. More over the frequency of occurrence of lower acceleration values will be more and that of higher acceleration values will be less. In the conventional liquefaction analysis such variations in frequency of occurrence is also not taken into account. Hence a new probabilistic performance based approach will help in considering the uncertainty in earthquake loading for the liquefaction triggering in a better way. A probabilistic performance based approach for liquefaction analysis based on SPT values was suggested by Kramer & Mayfield (2007).

$$
\begin{align*}
\lambda_{EDP^*} = \sum_{i=1}^{N_{IM}} P \left[ EDP > EDP^* \right] |M = i_{IM}| \Delta m_i
\end{align*}
$$

(4)

Where $EDP$ – Engineering damage parameter like factor of safety etc.; $EDP^*$—a selected value of $EDP$; $IM$ – intensity measure which is used to characterize the earthquake loading like peak ground acceleration, etc; $i_{IM}$—the discretized value of $IM$; $\lambda_{EDP^*}$—mean annual rate of exceedance of $EDP^*$.
\( \Delta \lambda_{im_j} \) — incremental mean annual rate of exceedance of intensity measure \( im \). The following equation can be derived by considering the EDP as factor of safety and the intensity measure of ground motion as a combination of PGA and magnitude.

\[
\Lambda_{FSL}^* = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P[FS_L < FS_L^* | a_i, m_j] \Delta \lambda_{a_i, m_j}
\]

(5)

Fig. 2: A Typical Deaggregated Seismic Hazard Curve

Where \( \Lambda_{FSL}^* \) — annual rate at which factor of safety will be less than \( FS_L^* \); \( FS_L \) — factor of safety against liquefaction; \( FS_L^* \) — targeted value of factor of safety against liquefaction; \( N_M \) — number of magnitude increments; \( N_a \) — number of peak acceleration increments; \( \Delta \lambda_{a_i, m_j} \) incremental annual frequency of exceedance for acceleration \( a_i \) and magnitude \( m_j \) (this value is obtained from the deaggregated seismic hazard curves with respect to magnitude). The conditional probability in the previous equation is:

\[
P[FS_L < FS_L^* | a_i, m_j] = \Phi \left[ \frac{(N_i)_{ho} (1 + \theta FC) - \theta_1 \ln(CSR_{eq,i}^*)/FS_L^*)}{\sigma_e} - \theta_3 \ln(m_j) - \theta_4 (\ln(\sigma_v / \sigma_e) / P_o) + \theta \right]
\]

(6)

Where \( \Phi \) — standard normal cumulative distribution; \( (N_i)_{ho} \) — penetration resistance of the soil from the standard penetration test (SPT) corrected for energy and overburden pressure; \( FC \) — fines content of the soil in percent; \( \sigma_v \) — effective overburden pressure; \( \sigma_e \) — atmospheric pressure in the same unit as \( \sigma_v \); \( \theta_1 \) and \( \sigma_e \) — model coefficients developed by regression.

\[
CSR_{eq,i} = 0.65 \frac{a_i}{g} \frac{\sigma_v}{\sigma_e} r_d
\]

(7)

\( CSR_{eq,i} \), the \( CSR \) value calculated without using the MSF for an acceleration \( a_i \), will be calculated for all the acceleration levels. In a similar manner the corrected SPT (for overburden pressure or overburden pressure and percentage of fines) values required to prevent liquefaction can also be evaluated for any given return period (Kramer and Mayfield 2007).

\[
\lambda_{N_{req}}^* = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P[N_{req} > N_{req}^* | a_i, m_j] \Delta \lambda_{a_i, m_j}
\]

(8)

Where

\[
P[N_{req} > N_{req}^* | a_i, m_j] = \Phi \left[ \frac{-\theta_3 \ln(m_j) - \theta_4 (\ln(\sigma_v / \sigma_e) / P_o) + \theta}{\sigma_e} \right]
\]

(9)

The value of \( N_{req}^* \) is the corrected \( N \) value (for both overburden pressure and percentage of fines) required to prevent the liquefaction with an annual frequency of exceedance of \( \lambda_{N_{req}}^* \).

3.2.2 Evaluation of Liquefaction Potential Based on CPT Values

In the previous section the discussion was centered around the correlations to evaluate the liquefaction potential based on SPT values. Due to the better accuracy and repeatability, several correlations are available for estimating the CRR values based on CPT values. The first to propose liquefaction triggering model based on CPT values was Robertson & Campanella (1985) and Seed & de Alba (1986). One of the most widely used correlations now a days is the one proposed by Robertson & Wride (1998). The two set of relations proposed by Robertson and Wride (1998) are:

\[
CRR = 0.833 \left[ \frac{(q_{CS1N})_{CS}}{1000} \right]^3 + 0.05 \quad \text{for} \quad (q_{CS1N})_{CS} < 50
\]

(10)

\[
CRR = 93 \left[ \frac{(q_{CS1N})_{CS}}{1000} \right]^3 + 0.08 \quad \text{for} \quad 50 \leq (q_{CS1N})_{CS} < 160
\]

(11)

Where \( (q_{CS1N})_{CS} \) is the normalized cone penetration resistance corrected for overburden pressure and percentage
fines. A logistic regression approach was proposed for the liquefaction potential evaluation by Lai et al. (2006). The general form of the equation suggested by Lai et al. (2006) is:

\[
PL = \frac{1}{1 + \exp\left(-\left[b_0 + b_1 \ln(CSR_{7.5}) + b_2 \ln(C1N)\right]\right)}
\]  

(12)

This work has proposed different values for the regression coefficients, based on the types of soils (sands, sand mixtures and silt mixtures), to evaluate the probability of liquefaction. In a more recent work, Moss et al. (2006) suggests a probabilistic approach to identify the liquefaction triggering based on CPT values. The probability of liquefaction can be obtained based on this method using the following relation.

\[
PL = \Phi\left[\frac{q_{1}^{0.045} + 0.110 R_f + 0.001 R_f}{-0.848 \ln(Mw) - 0.002(\ln(\sigma_{vo}) - 20.923)} + \frac{1}{1.632}\right] \ln(CSR)
\]

(13)

Where \(PL\) — probability of liquefaction; \(q_{1}\) is the normalized tip resistance; \(R_f\) — friction ratio (%); \(c\) — normalization exponent; \(CSR\) — cyclic stress ratio; \(Mw\) — moment magnitude of earthquake; \(\sigma_{vo}\) — effective overburden pressure. In this method the earthquake loading is characterized by the ground acceleration and the earthquake magnitude; the soil resistance is characterized by normalized tip resistance, friction ratio, normalization exponent and effective overburden pressure.

A probabilistic performance based relation can be derived from the above equation for evaluating the factor of safety against liquefaction based on CPT values.

\[
\Lambda_{FSL}^{*} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_a} P[FS_{L} < FS_{L}^{*} | \Delta \sigma_{q1}, m_j] \Delta \sigma_{q1}, m_j
\]

(14)

Where \(\Lambda_{FSL}^{*}\) is the annual rate at which factor of safety will be less than \(FS_{L}^{*}\); \(N_M\) is the number of magnitude increments; \(N_a\) is the number of peak acceleration increments; \(\Delta \sigma_{q1}, m_j\) is the incremental annual frequency of exceedance for acceleration \(a_i\) and magnitude \(m_j\). The conditional probability in the above equation can be estimated using the following expression.

\[
P[FS_{L} < FS_{L}^{*} | \Delta \sigma_{q1}, m_j] = \Phi\left[\frac{q_{1}^{0.045} + 0.110 R_f + 0.001 R_f}{-7.177 \ln(CSR_{eq}) - 0.848 \ln(m_j - 0.002(\ln(\sigma_{vo}) - 20.923)} + \frac{1}{1.632}\right]
\]

(15)

In a similar way the CPT values required to prevent liquefaction for a given return period can be evaluated using the following equations.

\[
\lambda_{q_{cl} > m_j} = \sum_{i=1}^{N_M} \sum_{j=1}^{N_a} P[q_{cl} > q_{cl,req} | \Delta \sigma_{q1}, m_j]
\]

(16)

Where the conditional probability in the previous equation can be evaluated by:

\[
P[q_{cl} > q_{cl,req} | \Delta \sigma_{q1}, m_j] = \Phi\left[\frac{q_{cl}^{*} - q_{cl,req} + 0.110 R_f}{-7.177 \ln(CSR_{eq}) - 0.848 \ln(m_j - 0.002(\ln(\sigma_{vo}) - 20.923)} + \frac{1}{1.632}\right]
\]

(17)

3.2.3 Evaluation of Liquefaction Potential Based on Shear Wave Velocity Values

One of the latest methods to evaluate the sub-soil properties is through the measurement of small strain shear wave velocity (\(V_s\)) values. In the last three decades lots of studies were conducted to evaluate the relationships between liquefaction susceptibility and \(V_s\). After considering the liquefaction case histories of 20 earthquakes and by analyzing about 193 liquefaction and non-liquefaction case histories, Andrews and Stokoe (1997) developed a relation to evaluate the cyclic resistance of soils based on \(V_s\). This relation was modified by Andrews and Stokoe (2000), for cemented and aged soils (> 10,000 years), by incorporating a correction factor and is given below.

\[
CRR = \left\{a_1 \frac{K_c V_S}{100} \right\}^2 + b_1 \frac{1}{V_s - K_c V_S} + b_2 \frac{1}{V_s} \right\} MSF
\]

(18)

Where \(V_s\) — shear wave velocity corrected for overburden pressure, \(K_c\) — correction factor for higher values of \(V_s\). The model uncertainty in the above relation was evaluated using reliability methods by Juang et al. (2005). A probabilistic neural network (PNN) was adopted by Gho (2002) to evaluate the liquefaction susceptibility based on \(V_s\).

4. PROBABILISTIC SEISMIC HAZARD ANALYSIS

(PSHA)

One of the most important steps in the evaluation of liquefaction susceptibility is the assessment of earthquake loading. There are two different methods for evaluating the earthquake loading—based on deterministic and probabilistic approaches. Since the uncertainties involved in the earthquake occurrence is well accounted in the probabilistic approach, this is widely being followed in hazard evaluation. The uncertainties involved in the magnitude of earthquake,
recurrence rate, hypocentral distance and the attenuation characteristics of the seismic waves were considered in the analysis. By using the PSHA method, the entire range of ground acceleration values and the entire earthquake magnitude range can be used in the liquefaction analysis. The different methods used to calculate the uncertainties involved in the seismic hazard analysis is similar to the one adopted by Vipin et al. (2009). In the liquefaction susceptibility analysis, the earthquake acceleration at ground surface needs to be evaluated. The site classification was done based on the recommendations of NEHRP (BSSC 2001). The surface level Peak Ground Acceleration (PGA) values were evaluated by using the amplification factors suggested by Raghukanth & Iyengar (2007).

5. CASE STUDY OF BANGALORE URBAN CENTRE

The first case study for the evaluation of liquefaction potential based on the probabilistic performance based approach was done for Bangalore. This evaluation was done based on the SPT data obtained from 500 boreholes. The borehole data at a particular location includes the variation of SPT values with depth, soil properties with depth like density of soil, percentage fines, etc and the depth of water table. The seismic hazard analysis of Bangalore was done based on probabilistic approach and the Peak Horizontal Acceleration (PHA) values were evaluated at the bed rock level. For the evaluation of liquefaction potential the Peak Ground Acceleration (PGA) values at surface level is required. Based on the results of the MASW study conducted for Bangalore, the study area was classified as site class D (Anbazhagan and Sitharam 2008). The PGA values were evaluated based on the attenuation relations suggested by Raghukanth and Iyengar (2007) for south India. At each borehole location the deaggregated PGA values with respect to magnitude were evaluated. Based on this, the factor of safety against liquefaction was evaluated at a depth of 3 m for a 10% probability of exceedance in 50 years (return period of 475 years), using the methods explained in the previous sections. The contour map showing the spatial variation of factor of safety against liquefaction is shown in Figure 3. It can be observed from Figure 3 that majority of Bangalore is having a factor of safety higher than 1.

The SPT values required to prevent liquefaction for a return period of 475 years at a depth of 3 m were also evaluated for Bangalore. The contour curves showing the spatial variation of SPT values required to prevent liquefaction are shown in Figure 4. The CPT values required to prevent liquefaction for a return period of 475 years (10% PE in 50 years) and at a depth of 3 m were also evaluated (Fig. 5). A table showing the comparison of SPT and CPT values required to prevent liquefaction at some of the selected locations in Bangalore are given in Table 2.

6. CASE STUDY FOR SOUTH INDIA BASED ON SITE CLASSES

In the case of a vast study area like south India it is extremely difficult to get the complete soil profile (borehole data) of all the places/locations. Hence in this work a new methodology is proposed to evaluate the liquefaction susceptibility. Using the Eq. 8 & 16 the corrected SPT and CPT values required to prevent liquefaction for a given return period were evaluated for the entire study area. For this calculation the surface level PGA values were evaluated for different site classes. In this paper the results are
presented for site class- D alone. The variation of SPT and CPT values required to prevent liquefaction for the entire study area is presented in Figure 6 & 7. This will help in knowing the liquefaction susceptibility of any location in south India. Based on the actual SPT or CPT values obtained from the site, the factor of safety against liquefaction can be evaluated. The SPT and CPT values required to prevent liquefaction are high near the Koyna region. The regions near Bangalore and Ongole are also having comparatively higher values.

Table 2: The Factor of Safety Against Liquefaction Based on SPT and CPT Values for Different Locations in Bangalore at a Depth of 3 m and for a Return Period of 475 Years.

<table>
<thead>
<tr>
<th>Location</th>
<th>Factor of safety against liquefaction</th>
<th>$(N_1)_{so}$ Required to prevent liquefaction</th>
<th>$q_{ct}$ Required to prevent liquefaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hebbal</td>
<td>0.88</td>
<td>13.77</td>
<td>11.06</td>
</tr>
<tr>
<td>Hudson Circle</td>
<td>0.74</td>
<td>13.55</td>
<td>10.74</td>
</tr>
<tr>
<td>Indira Nagar</td>
<td>0.73</td>
<td>12.81</td>
<td>10.44</td>
</tr>
<tr>
<td>Koramangala</td>
<td>2.18</td>
<td>13.65</td>
<td>10.65</td>
</tr>
</tbody>
</table>

7. CONCLUSIONS

An overview of different methods to evaluate the liquefaction potential is presented in this work. The conventional liquefaction analysis methods do not consider the uncertainty in the earthquake loading. Hence the adoption of a performance based probabilistic methods will help in considering the uncertainties due to earthquake loading in a better manner. The entire procedure for liquefaction analysis based on the probabilistic approach is explained in this work.

REFERENCES


Seismic Soil Liquefaction Based on in situ Test Data


