Abstract: Several authors have studies liquefaction in different approaches like deterministic and probabilistic approach based in situ and laboratory test.

In this article we are going to evaluate the Real Probability of Liquefaction ($P_R^L$) in Golem area related with the Factor of Safety (FS) computed with the deterministic piezocone penetration test (CPTU)-based model by Juang et al (2008). The real probability of liquefaction is calculated for a given seismic hazard level expressed in terms of acceleration, with the probability of exceeding the same acceleration during the lifespan of engineering structures (usually 50 years). The effects of liquefaction are going to calculate with Liquefaction Potential Index $IPL$. The computed liquefaction probabilities indicate that during the life span of a structure, the phenomenon may be encountered and it should be taken into consideration during the design stage of the engineering projects.

Keywords: Liquefaction, cone penetration test, probability of liquefaction, liquefaction potential index.

1. Introduction

The simplified procedure for evaluating the liquefaction potential of a soil was created by Seed & Idriss (1971) [1]. Generally speaking, the framework of the analysis with the simplified procedure consists of evaluation of the seismic loading, in the form of the cyclic stress ratio (CSR), and of the soil resistance against liquefaction, in the form of the cyclic resistance ratio (CRR). The liquefaction potential or triggering is then measured with a Factor of Safety (FS=CRR/CSR), in which liquefaction is said to occur if $FS<1$. In practice, to evaluate CRR correlations with in situ tests data are different tests such as the standard penetration test (SPT), the cone penetration test (CPT), the Becker penetration test (BPT), and shear wave (Vs) measurements. The present study focuses on the use of piezocone penetration tests (CPTU), and thus only CPT-based models proposed by Juang et al (2010) [2] is presented here. For performance-based earthquake engineering (PBEE) design, it is often necessary to determine the probability of liquefaction of a soil at a site in a given exposure time or the Real Probability of Liquefaction ($PR^L$). To measure the effect of liquefaction is necessary to calculate Liquefaction Potential Index $IPL$. In this study we are going to present the methodology and the result of calculation of FS, ($PR^L$) and $IPL$ in the area of Golem where 10 CPTU test where done to conduce this study. The area under study is situated at Golem municipally of Kavaja County, at the central Albanian coast, in Tirana Prefecture. From the seismic point of view this area is located in the Periadiatic Depression, denoted as PL – zone, strongly affected by post–Pliocene compression movements, in direct convergence with Adria microplate. It is characterized by a high seismic activity and according to the seismic hazard map of Albania the second level of hazard represent an earthquake with 10% probability of exceedance in 50 years, or a PGA=0.273g [3]. According to the Albanian earthquakes catalogue, the expected earthquake surface wave magnitudes ($M_s$) of the considered area vary from 4.5 – 6.6 [4]. The highest magnitude is $Ms = 6.6$ (occurred in year 346 and in the region with coordinates: 41.30 °N; 19.30 °E). The soil behavior type index, $I_c$, as defined by Robertson and Wride (1998) [5] is used to identify the soils type and the boundaries of the $I_c$ are indicate in Table 1. The type of soils met in Golem area are silty sands and sandy silt, clay and silty clay. This type of soils are suspected to liquefy when they are in e seismic area and under the ground water table.
2. Evaluation of Liquefaction Potential, Based on CPTU Data

To evaluate liquefaction potential essentially we have to compares the cyclic resistance ration (CRR) at a given depth with the earthquake-induced cyclic stress ration (CSR) at the depth from a specified design earthquake. The evaluation procedure can be outlined using empirical equations deduced by different authors (Seed and Idriss, 1971; Olsen, 1997 [6]; Robertson and Wride, 1998; Chen and Juang, 2000 [6], 2006 [7], 2010; Lee et al., 2003 [8]; Yuan et al., 2003 [9]). The determination of CSR, usual is calculate based in methods proposed by Seed and Idriss (1971), in this paper is calculated based in model proposed by Idriss and Boulanger (2006) [10]. For the determination of CRR, different simplified methods have been proposed till today. In this paper the method proposed by Juang (2010) is applied.

2.1 The calculation of CSR after Idriss and Boulanger (2006)

The average uniform cyclic stress ratio (CSR) within a liquefiable layer is defined as

\[
\text{CSR}_{7.5} = 0.65 \left( \frac{\sigma_v}{\sigma_0} \right) \left( \frac{a_{\text{max}}}{g} \right) \left( \frac{1}{r_d} \right) \left( \text{MSF} \right)
\]

(1)

Where:

- \(a_{\text{max}}\) is the peak ground surface acceleration generated by the earthquake (g); \(g\) is acceleration of gravity;
- \(\sigma_v\) and \(\sigma_0\) are the vertical total stress and effective overburden stresses (kPa);
- \(r_d\) is the depth-dependent shear stress reduction factor; MSF is a magnitude scaling factor; and \(K_{\sigma}\) is the overburden correction factor for CSR.

The term \(r_d\) is computed as

\[
\ln r_d = \alpha + \beta M_w
\]

(2)

Where

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

(2.1)

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

(2.2)

\(Z\) is the depth of interest (m), and \(M_w\) is the moment magnitude.

The term MSF is computed as

\[
\text{MSF} = -0.058 + 6.9 \exp \left( \frac{-M_w}{11.8} \right)
\]

(3)
2.2 The calculation of CRR after Juang (2000, 2006, 2010)

Cyclic resistance ratio (CRR) in the CPTU-based model is a function of two derived CPTU parameters, the adjustment cone tip resistance \( q_{t, 1N} \) and the soil behaviour type index \( f \). In the present study the parameter \( c_{BJ} \) is defined by Robertson (2009 a) [11] and \( I_{BJ} \) is defined by Been and Jefferies (1992) [12]. CRR after Juang 2010 has the following equation:

\[
CRR = 0.05 + \exp[A + B] \left( \frac{q_{t, 1N}}{100} \right)^c
\]

Where

\[
A = I_{c, BJ} \left( q_{t, 1N} \sqrt{100} \right) - 14.7 \quad (4.1)
\]

\[
B = 0.909 I_{c, BJ}^3 - 7.47 I_{c, BJ} + 19.28 \quad (4.2)
\]

\[
C = 0.059 + 0.015I_{c, 2} \quad (4.3)
\]

\( I_{c, BJ} \) defined by Been and Jefferies (1992) is

\[
I_{c, BJ} = \sqrt{\frac{3 - \log \left[ \frac{Q_{10} - 1 - B}{10} + 1 \right]_2 + \left[ 1.5 + 1.3 \log F \right]^2}{10 + r}}
\]

\( q_{t, 1N} \) defined by Robertson (1998) is

\[
q_{t, 1N} = \left[ q - \sigma_0 \right]^{1/0} \frac{P}{\sigma_v} \left[ \sigma_v \right]^{3/0} \quad (4.5)
\]

\[
n = 0.381I_c + 0.05 \frac{\sigma_v}{P - 0.5} \quad (4.6)
\]

\[
I_{c, BW} = \sqrt{3.47 - \log \left[ \frac{Q_{10}}{10} \right]_2 + \left[ 1.22 + \log \frac{F}{10} \right]_2}
\]

Where

\( q_c \) is cone resistance; \( q_t \) is corrected resistance \( = q_c + (1 - a)u_2 \); \( a \) is the area ratio of the cone used (in this paper, \( a=0.85 \)); \( u_2 \) is penetration pore pressure (kPa); \( B_q \) is pore pressure parameter \( = (u_2 - u_0) / (q_t - \sigma_v) \); \( u_0 \) is hydrostatic water pressure (kPa); \( Q_t \) is normalized cone resistance \( = (q_t - \sigma_v) / \sigma_v \); \( R_f \) is friction ratio \( = (f / q_c) \cdot 100\% \); \( F \) is is normalised friction ratio \( = [f / (q_t - \sigma_v)] \cdot 100\% \); \( Pa \) is atmospheric pressure (\( \approx 100 \) kPa).

2.3 Calculation of Factor of Safety (FS)

After determine CSR and CRR we can calculate FS which after Lee et al., 2003 is:

\[
FS = \frac{CRR}{CSR_{7.5}}
\]

A soil is predicted to liquefy if \( FS \leq 1.2 \) (Sonmez, 2003) [13].

3. Probability Calculation

The real probability of liquefaction, \( P_R [L] \), is the probability of liquefaction during the lifespan of an engineering structure for different levels of safety, corresponding to different seismic hazard levels. For a given hazard seismic level, \( P_R [L] \) is calculated by combining the conditional probability of liquefaction for the corresponding acceleration, \( P_R [L | PGA = a] \), with the probability of occurrence of the scenario causing the exceedance of this acceleration \( P_R [PGA > a] \).
\[ P(L | P = a | PGA = a > a) \]  

(6)

The conditional probability of liquefaction, \( P_R [L | PGA = a] \), is calculated using the correlation proposed by Juang et al (2006), which represents a direct correlation between the FS obtained in a deterministic way as proposed by Juang and the probability of liquefaction

\[ P_R [L | PGA = a] = \frac{1}{1 + (FS/A)^B} \]

with \( A = 0.74 \) and \( B = 5.45 \)  

(7)

The seismic hazard curve, is used to determine the probability of exceeding the acceleration \( P_R [PGA > a] \) during the lifespan of the structure (50 years).

After Chen and Juang (2000) the likelihood of liquefaction can be interpreted using the calculated \( P_R [L] \) values in Table 2.

<table>
<thead>
<tr>
<th>Probability</th>
<th>Description (likelihood of liquefaction)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.85 ≤ ( P_R [L] ) &lt; 1.00</td>
<td>Almost certain that will liquefy</td>
</tr>
<tr>
<td>0.65 ≤ ( P_R [L] ) &lt; 0.85</td>
<td>Very likely</td>
</tr>
<tr>
<td>0.35 ≤ ( P_R [L] ) &lt; 0.65</td>
<td>Liquefaction/non-liquefaction is equally likely</td>
</tr>
<tr>
<td>0.15 ≤ ( P_R [L] ) &lt; 0.35</td>
<td>Unlikely</td>
</tr>
<tr>
<td>0.00 ≤ ( P_R [L] ) &lt; 0.15</td>
<td>Almost certain that it will not liquefy</td>
</tr>
</tbody>
</table>

(8)

It can be seen in Table II that liquefactions will occur only if \( P_R [L] \) greater than 35% is. The calculation of \( P_R [L] \) permits to observe if a layer is susceptible to liquefy during a specific earthquake.

4. Liquefaction Potential Index (\( IP_L \))

The liquefaction potential index (\( IP_L \)) is used to evaluate the ground failure risk. It severity categories were proposed originally by Iwasaki et al. (1982) [14] and modified by Sonmez in 2003. The calculation is used the FS (Equation 5) and a depth weighting function \( W(z) \) (Equation 8.2). In this way the contribution of the soil liquefaction at different depths to the failure of the ground is estimated. Sonmez (2003) proposed the change of the threshold value of FS between non-liquefiable and liquefiable layers from 1.0 (Iwasaki et al., 1982) to 1.2 and suggested following equations:

\[ IP_L = \int_{0}^{20} F(z) \cdot W(z) \cdot dz \quad \text{where } FS = CRR/CSR \]

(8)

\[ F(z) = \begin{cases} 1 - FS & \text{for } FS < 0 \\ 2 \cdot 10^6 \exp(-18.427 \cdot FS) & \text{for } 0.95 < FS < 1.2 \\ 0 & \text{for } FS \geq 1.2 \end{cases} \]

(8.1)
Fig. 2: Liquefaction evaluation for Golem area, BH.2

Fig. 2: Liquefaction evaluation for Golem area, BH.8
\[ W(z) = 10 - 0.5 \cdot z, \quad \text{for} \ 0 \leq z \leq 20m \]
\[ W(z) = 0, \quad \text{for} \ z \geq 20m \]

\[ z = \text{depth (m)} \]

To interpret the obtained values of IP\(_L\) (Equation 8), a classification proposed by Iwasaki (1982) and modified by Sonmez (2003) is used. In Table 3 liquefaction potential categories are presented.

<table>
<thead>
<tr>
<th>Liquefaction Potential Index IP(_L)</th>
<th>Liquefaction potential category</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Non-liquefiable (based on FS \leq 1.2 )</td>
</tr>
<tr>
<td>0 &lt; IP(_L) \leq 2</td>
<td>Low</td>
</tr>
<tr>
<td>2 &lt; IP(_L) \leq 5</td>
<td>Moderate</td>
</tr>
<tr>
<td>5 &lt; IP(_L) \leq 15</td>
<td>High</td>
</tr>
<tr>
<td>IP(_L) &gt; 15</td>
<td>Very high</td>
</tr>
</tbody>
</table>

5. Results

The detailed lithological profiles based on the soil Behavior Type Index \(I_c\) for two represented borehole for Golem area are shown in Fig. 1 and Fig.2. The dashed line green, in Fig.1 and Fig.2, corresponded to a value of \(I_c\) = 2.8. This value represents the uppermost limit below which the liquefaction assessment is necessary, Yuanget al., 2003. In Fig.1 where \(I_c < 2.8\) are two intervals to calculate about the liquefaction potential. The first interval is 0.72-7.97 m depth with \(1.31 < I_c < 2.05\) (Sandy to silty sand) and the second interval 7.97-12.78 m depth with \(2.05 < I_c < 2.60\) (Silty sand & sandy silty). In Fig.2 almost in all penetration depth \(I_c < 2.8\). There are two interval where \(1.31 < I_c < 2.05\) (Sandy to silty sand). The first one is 0.0-6.46m depth, and the second one is 7.14-10 m depth. The interval where \(2.05 < I_c < 2.60\) (Silty sand & sandy silty) is 6.46-7.14 m depth.

Factor of Safety, FS<1.2 (the dashed line red) is for two interval. In Fig.1 in the interval 8.5-12.7 m depth and the second interval is 3.8-8.2 m depth in Fig.2.

The conditional probability of liquefaction \(P_R[L\mid PGA = a]\) show that in generally the susceptible layers is liquefiable. According to the value in Table 2, in Fig.1 for the interval 8.5-12.7 where \(P[L\mid PGA = a] > 100\) the liquefaction is “almost certain that will liquefy”. In the Fig.2 \(P[L\mid PGA = a] > 100\) \(P[L\mid PGA = a] > 100\) the liquefaction is “almost certain that will liquefy” for the interval of depth 3.8-8 m depth.

Liquefaction potential index that evaluate the ground failure risk has a value >15, that means “very high risk” in the same interval for both Fig1, and Fig.2 where FS<1.2.

6. Conclusions

The Factor of Safety, the conditional probability value obtained for the given seismic hazard level indicate that in generally the liquefaction is almost certain to happen. The liquefaction probabilities attempt such values requiring to consideration of the phenomenon during the design of engineering projects especially because this is an important area of truism of the sea coast line in Albanian. Since this type of soils are met along the coast line of Adriatic Sea a study like this is very important in a large scale.

7. References


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