Liquefaction Potential Hazard in South East of Khoy

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ABSTRACT

One of the important problems in earthquake geotechnical engineering is liquefaction phenomenon that happens in loose saturated granular soils. This phenomenon can cause great damages to underground structures and buildings and lifelines. Liquefaction resistance of soils can be evaluated by experimental and field tests. In this research, results of liquefaction potential evaluation based on standard penetration test (SPT) were proposed. Case study area is Southeast of Khoy city at West Azerbaijan province in Iran. In this study 18 boreholes was collected. With considering type of soils and ground-water table level liquefaction potential evaluated. Then, liquefaction potential index (LPI) assessed. Obtained results showed that almost more of 50% alluvium sediments deposits is included sand and silty sand. Also, liquefaction potential hazard with considering ground water table level is high.

Keywords:

1. Introduction

Liquefaction in soil due to earthquake is one of the important happens and cause of sever damages on structures and lifelines. The pore water pressure during earthquake in loose saturated granular soils (in special condition clayey soil) increases and in continue soil tend to reduce volume and confine stress decreases [1]. Finally, shear strength in soil is about equal to zero and in this state liquefaction has happened [1]. This phenomenon occurs such as extended ground settlements, sand boiling and water seepage on ground. Several factors can be affected on occurrence of liquefaction such as earthquake magnitude and duration, void ratio, relative density, fines content, plasticity index and etc. [1]. Liquefaction resistance of soils can be evaluated with using laboratory tests such as cyclic simple shear test, cyclic triaxial tests and cyclic torsional test or field tests e.g. standard penetration test (SPT) [2], cone penetration test (CPT) [3] and shear wave velocity (Vs)

[4]. Main goal of this study, evaluation of soils liquefaction potential in the sediment deposits of Ghazan Chay Dam near to Khoy city by SPT. In continue, liquefaction potential index (LPI) assessed. In this research, Idriss and Boulanger [5] procedure is used in SPT method, Liquefaction potential index (LPI) evaluated by Iwasaki et al. [6-7] and Sonmez [8] procedure.

2. Geology and general condition in study area

Study area located in Southeast of the Khoy city at West Azerbaijan province in Iran. In this region about 18 boreholes were collected and according to geotechnical properties of taken specimens, generally soil layers type with using unified method more of 50% are SM, SC and ML. Variations of groundwater level in study area almost groundwater level changes between 0.3 and 6.3 meters. One of the important factor in liquefaction potential evaluation in soil layers is peak ground acceleration due to earthquake happen. According to the Iranian Code of Practice for Seismic Resistant Design of Buildings the PGA equal to 0.35g (475 years is the return period and a useful life 50 years) and Mw equal 7.5 are considered.

3. Liquefaction analysis based on SPT method

Assessment of the liquefaction potential of the soils in the study area based on the simplified method proposed by Idriss and Bolanger [2] is carried out. In this method, first, the value of cyclic stress ratio (CSR) is estimated expressing the rate of the severity of the earhquake load in a Mw=7.5. That is evaluated using the equation bellow:

$$CSR_{7.5} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_V}{\sigma'_V} \cdot r_d \cdot \frac{1}{MSF} \tag{1}$$

Where a_{max} is the peak ground acceleration, g is acceleration of gravity, σV total stress in the depth in the question, σ´V effective stress in the same depth, rd coefficient of shear stress reduction using the form Figure 1 is estimated and MSF (Magnitude Scale Factor) is earthquake magnitude scale factor that is calculated based on Andrus and Stoke researches [4] using equation 2. Mw is earthquake magnitude:

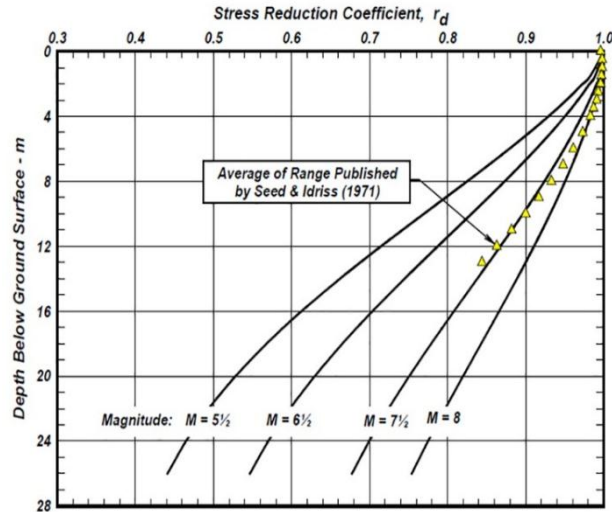$$MSF = \left(\frac{M_W}{7.5} \right)^{-3.3} \tag{2}$$

Figure 1. Variations of stress reduction coefficient with depth and earthquake magnitudes [12].

Second, in order to determine to cyclic resistance ratio (CRR) of the soils simplified and modified method proposed by Seed et al. [12] are used. In this step, the results obtained from the standard penetration test are modified based on the following equation proposed by Skempton [13]. Value of parameters can be observed in Table 1.

$$(N_1)_{60} = N_{SPT} \times C_N \times C_E \times C_B \times C_R \times C_S \qquad (3)$$

Where, N_{SPT}, the number of standard penetration resistance test, C_N coefficient of the over burden stress, C_E the coefficient of the hammer energy, C_S the coefficient of the sampling method, C_B the coefficient of the bore hole diameter, C_R the coefficient of the rod length and $(N_1)_{60}$ is the modified number of the standard penetration test. After that, according to the presented proposal by Idriss and Boulanger [5], the overburden tension correction factor (C_N) is determined using the following equation:

$$C_N = (\frac{Pa}{\sigma'_V})^\alpha \leq 1.7 \qquad (4)$$

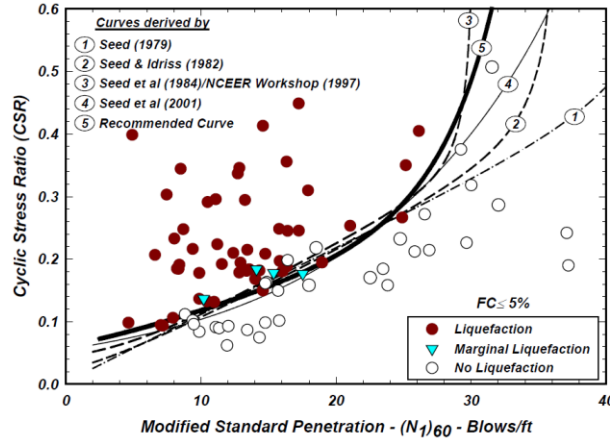$$\alpha = 0.784 - 0.0768 \times \sqrt{(N_1)_{60}} \qquad (5)$$

Figure 2. Liquefaction resistance curve for the earthquakes of 7.5 magnitudes [5].

Where, Pa = 100kPa, is the atmospheric pressure and $\sigma'v$ is the effective stress at the depth in question, and $(N_1)_{60}$ is corrected the number of standard penetration test. After the modification of the number of the standard penetration test, its equivalent in clean sand ($(N_1)_{60CS}$) is determined. Then, cyclic resistance ratio (CRR) is assessed by the application of the following equations (Figure 8):

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60} \tag{6}$$

$$\Delta(N_1)_{60} = 1.63 + \exp\left(1 - \frac{9.7}{FC+0.1}\right) - \left(\frac{15.7}{FC+0.1}\right) \tag{7}$$

$$CRR = \exp\left(\left(\frac{(N_1)_{60CS}}{14.1}\right) + \left(\frac{(N_1)_{60CS}}{126}\right)^2 - \left(\frac{(N_1)_{60CS}}{23.6}\right)^3 + \left(\frac{(N_1)_{60CS}}{25.4}\right)^4 - 2.8\right) \tag{8}$$

Where, FC is equal fines content in soil layer.

Table 1. Correction factor of SPT [13].

Overburden Pressure		C_N	$(P_a / \sigma'_v)^{0.5}$ $C_N \leq 1.7$
Energy ratio	Donut Hammer Safety Hammer Automatic-Trip Donut-Type Hammer	C_E	0.5 to 1.0 0.7 to 1.2 0.8 to 1.3
Borehole diameter	65 mm to 115 mm 150 mm 200 mm	C_B	1.0 1.05 1.15
Rod length	3 m to 4 m 4 m to 6 m 6 m to 10 m 10 m to 30 m > 30 m	C_R	0.75 0.85 0.95 1.0 〈1.0
Sampling method	Standard sampler Sampler without liners	C_S	1.0 1.1 to 1.3

3.1 Corrected cyclic resistance ratio (CRR$_J$)

In both method, the calculation of the CRR, if the amount of effective vertical stress at the depth in question is more than 100 kPa, the CRR value is modified by using the following equation:

$$CRR_j = K_\sigma \times CRR \tag{11}$$

In this equation, the CRRj is corrected cyclic resistance ratio. Furthermore, the Kσ parameter is a coefficient based on the effective vertical stress is calculated by the following [14]:
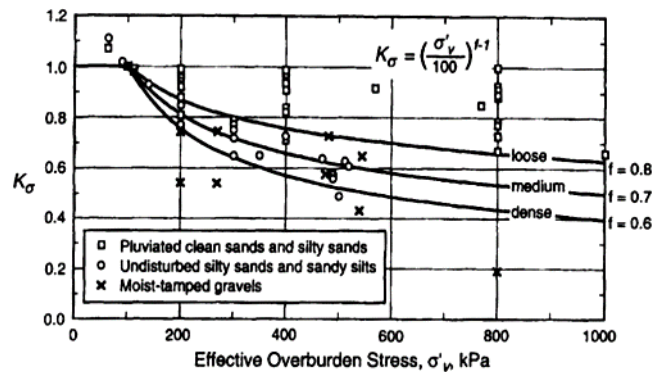
$$K_\sigma = (\frac{\sigma'_v}{100})^{f-1} \tag{12}$$

Figure 3. Variations of Kσ values versus effective overburden stress [14].

Where Kσ is the overburden correction factor, σ′v is the effective vertical stress and f is an exponent that is a function of site conditions including relative density, stress history, aging and over consolidation ratio. For the relative densities between 40% and 60%, f= 0.7-0.8 and for the relative densities between 60% and 80%, f= 0.6-0.7 (Figure 3).

3.2 Factor of safety (Fs)

Safety factor (Fs) against liquefaction in soil layers is calculated using the following equation:

$$F_S = \frac{CRR_j}{CSR} \tag{13}$$

Liquefaction occurs when the amount is Fs ≤ 1; when it is Fs >1 there is no probability of the occurrence of liquefaction.

4. Liquefaction potential index (LPI)

The researchers presented several methods for the assessment of the rate of liquefaction and the level of occurrence. One of the common methods is proposed by Iwasaki et al. [6-7] presented in the following equation:

$$LPI = \int_0^{20} W(Z) \times F(Z). dz \tag{14}$$
$$F(Z) = 1 - F_S \ \text{ For Fs} < 1 \tag{14a}$$
$$F(Z) = 0 \ \text{ For Fs} \geq 1 \tag{14b}$$

$$W(Z) = 10 - 0.5Z \quad \text{For } Z < 20 \text{ m} \tag{14c}$$
$$W(Z) = 0 \quad \text{For } Z > 20 \text{ m} \tag{14d}$$

Where, Z is the depth of midpoint in question layer. The Liquefaction intensity is stated between zeros and 100. The liquefaction risk can be obtained using Table 3 based on the liquefaction potential index (LPI) value.

Table 2. Liquefaction potential index (LPI) and its describes [6-7].

LPI- Value	Liquefaction risk and investigation/Countermeasures needed
LPI=0	Liquefaction risk is very low. Detailed investigation is not generally needed. (very low)
0<LPI≤ 5	Liquefaction risk is low. Further detailed investigation is needed especially for the important structures. (low)
5<LPI≤ 15	Liquefaction risk is high. Further detailed investigation is needed for structures. A countermeasure of liquefaction is generally needed. (high)
LPI> 15	Liquefaction risk is very high. Detailed investigation and countermeasures are needed. (very high)

The liquefaction severity categories proposed by Iwasaki et al. [6-7] consist of four classes called ''very low,'' ''low,'' ''high'' and ''very high'' depending on the value of the LPI (Table 3). The areas showing different degree of susceptibility classes and non-susceptible areas may be classified on susceptibility maps such as land slide prone. However, non-susceptible areas could not be distinguished based on the categories proposed by Iwasaki et al.. Furthermore, although ''high'' and ''low'' liquefaction potential categories are defined, the category ''moderate'' is lacking in the categories listed in Table 3. The limitations of the LPI and severity categories (Table 3) were discussed in detail by Sonmez [8]. To overcome these limitations, Sonmez modified F(Z) term appearing the equation of LPI by considering the threshold value of 1.2 between the non-liquefiable and marginally liquefied categories as follow:

$$F(Z) = 0 \qquad \text{For} \qquad \text{Fs} \geq 1.2 \tag{15a}$$
$$F(Z) = 2 \times 10^6 \times e^{-18.42 Fs} \qquad \text{For} \quad 0.95 < \text{Fs} < 1.2 \tag{15b}$$
$$F(Z) = 1 - F_S \qquad \qquad \text{For Fs} < 0.95 \tag{15c}$$

Sonmez introduced two new categories into the classification proposed by Iwasaki et al. [6-7] as ''non-liquefiable'' and ''moderate'' (Table 4). The boundary values of LPI for the categories of ''high'' and ''very high'' by Iwasaki et al. preserved by Sonmez. When FL>1.2 throughout the soil column from surface to a depth of 20 m, LPI of the soil column becomes zero and the column is classified as ''non-liquefiable'' by Sonmez . However, Sonmez pointed out that the threshold value of Fs between non-liquefiable and marginally liquefied conditions (Fs=1.2) is open to discussion, and the threshold value for the non-liquefiable category suggested in his study can be changed depending on the data in future studies. Seed et al. [12] mentioned that the values of Fs against liquefaction ranging between 1.25 and 1.5 are acceptable.

Table 3. Liquefaction potential index (LPI) and its describes [8].

LPI- Value	Liquefaction risk and investigation/Countermeasures needed
LPI=0	Non- Liquefiable (based on Fs $\geq$ 1.2)
0<LPI$\leq$ 2	Low
2<LPI$\leq$ 5	Moderate
5<LPI$\leq$ 15	High
LPI> 15	Very High

Also, Sonmez in 2003 proposed a new category for liquefaction potential intensity. In this method, liquefaction intensity (Ls) calculates with applying probability of liquefaction happen in soil layers with using equations in below:

$$L_s = \int_0^{20} P_L(Z).F(Z).dz \tag{16}$$

$$P_L(Z) = \frac{1}{1+(\frac{Fs}{0.96})^{4.5}} \tag{17}$$

Where, L_s is liquefaction potential intensity and $P_L(Z)$ is probable of liquefaction phenomenon in soil layer. In continue with combination eq.16 and eq.17 with Table 3 a new category according to Ls values suggested:

Table 4. Liquefaction potential index (Ls) category [8].

Ls-values	Describe
85 $\leq$ Ls < 100	Very high risk
65 $\leq$ Ls < 85	High risk
35 $\leq$ Ls < 65	Moderate risk
15 $\leq$ Ls < 35	Low risk
0 $\leq$ Ls < 15	Vry low risk
Ls=0	No liquefaction

5. Results

Results of present study can be explained as follows:

1- As mentioned in previous parts, in study area 18 boreholes collected and according to geotechnical properties of soil layers and groundwater table level liquefaction potential evaluations were performed. In boreholes type of soil layers based on unified classification method were included 85 silty sand (SM), 38 silty clay (CL-ML), 15 silt (ML) and 27 clay (CL). Variation of safety factors in soil layers versus liquefaction hazards can be observed in Figure 4. It is demonstrated about 70 percentage of soil layers in under groundwater table have liquefaction potential and risk value is relatively high.
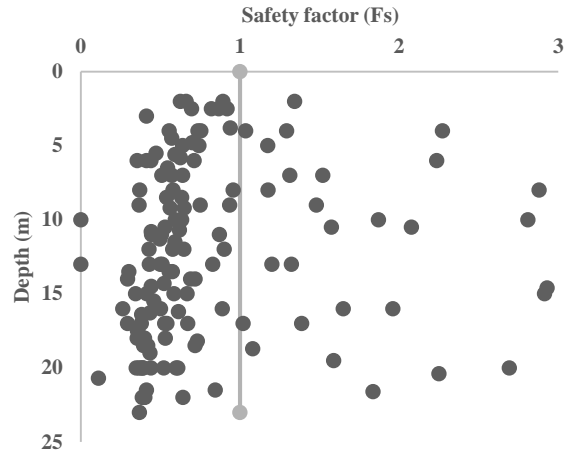
Figure 4. Variations of safety factors (Fs) in soil layers at study area.

2- Liquefaction potential index (LPI) values according to Iwasaki [6-7] method in soil layers of study area were estimated. Results can be seen in Figure 5. It is showed that 80 percentage of boreholes located in very high risk in terms of liquefaction. This outcome is in totally adopted with safety factors values.
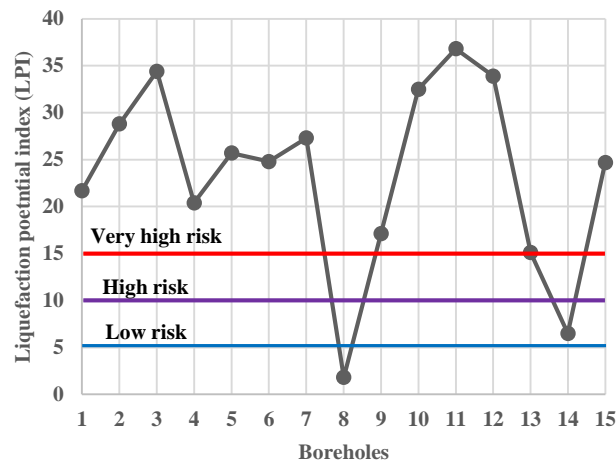

Figure 5. Variations of liquefaction potential index (LPI) in soil layers at study area.

3- Probability of liquefaction risk (Ls) in study area with using Sonmez [15] method evaluated too. Result proposed in Figure 6. As seen, almost 27 percentage have probability of liquefaction with high level risk. In the other hand, probable of liquefaction phenomenon in 8 boreholes located in moderate risk is 53 percentage. In final, 20% of boreholes have low risk in terms of probabilities.
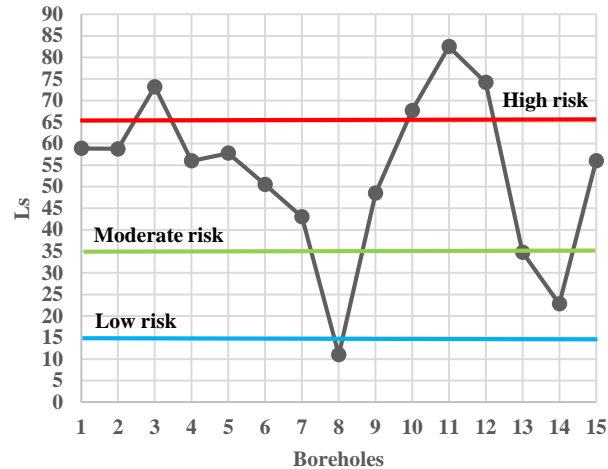
Figure 6. Variations of liquefaction potential probability (Ls) in soil layers at study area.

6. Conclusion and discussion

In this research, liquefaction potential of soil layers in Ghazan Chay Dam in Southeast of Khoy city in West Azerbaijan province was evaluated. In this research Idriss and Boulanger (2010) method based on SPT results used. Then, liquefaction potential index (LPI) according to Iwasaki et al. (1982, 1987) procedure and Sonmez (2003) method determined and obtained outcomes were compared. Consequences of this study is described as follows:

1- Values of safety factors versus liquefaction in soil layers showed about 70 percentage of soil layers in under groundwater table have liquefaction potential risk.

2- Liquefaction potential index (LPI) amounts showed that in total, axes of Ghazan Chay Dam and upstream have liquefaction risk with high level.

3- According to obtained results, it can be explained that there no good agreement and adaption between Iwasaki et al. (1978, 1982) and Sonmez methods in description of risk liquefaction level.

With considering of the items mentioned above, it is suggested with using soft computing method and artificial neural network a new criteria for proposing suitable levels in liquefaction risk descriptions.

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