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ABSTRACT

Liquefaction is one of the most important and complex topics in geotechnical earthquake engineering. During this phenomenon, pore water pressure increases as long as it will be equal to confining stresses. Hence, the effective confining stress becomes zero and the soil will not have any shear resistance. As a result the soil mass is unstable and causes much destruction. In this research, according to information from boreholes of Tabriz Urban Train Line 2 all required parameters including total stresses, pore water pressures and effective stresses, the results according to soil type and water depth for all 53 preliminary boreholes were collected and evaluation of the liquefaction potential assessment based on energy standard penetration resistance test (SPT) has been compared. The depth of standard penetration resistance test (Nspt) in which the results were not available, the interpolation method were used for all layers. For evaluation of liquefaction potential based on (SPT) method the latest techniques offered by Idriss - Boulangr (2008) were used. In this paper, calculations are presented for an earthquake of 7.5 in the scale of Richters. Then the safety factor against liquefaction is computed by these methods for several boreholes at different depths, and liquefaction risk evaluation has been done by Iwasaki method. At last by comparison of 3 sample boreholes and considering difference between them it can be concluded that some areas of Tabriz metro line 2 is located in perfect liquefaction conditions.

INTRODUCTION

The increase in pore water pressure, results in reduction in shear strength of sandy soils or even it may completely vanish. This is called liquefaction phenomenon. Soils that lose their shear strength totally will act as a thick liquid and has a tendency to flow. Considering the existence of saturated sandy soils with noticeable thickness in different boreholes and also graphs of SPT there is possibility in occurrence of liquefaction phenomenon. In recent years several methods have been presented in order to evaluate the soil liquefaction potential. In this paper the latest method presented by Idriss and Boulanger [10] has been used that is the most common method for the evaluation of soil liquefaction potential.

RELATION BETWEEN STANDARD PENETRATION TEST (SPT) AND SOIL LIQUEFACTION

Standard Penetration Test (SPT) is one of the most usual site test in order to determine the resistance against liquefaction. Parameters that cause increase in the resistance against liquefaction are density, strain before the earthquake, over consolidation ratio, lateral earth pressure and also high SPT number. In 1985 studies have been taken by Seed et. al. for a clean Sand to measure the least ratio of cyclic strain which is expected for occurrence of liquefaction in clean sand with a given SPT. Having fine ingredients can influence SPT, therefore it must be calculated in the evaluation of the resistance against liquefaction. If the amount of fine sand is less than 5% ($FC \leq 5\%$) the resistance against liquefaction will not be influenced by fine sand but higher percents of fine sand prevents liquefaction because it needs higher CSR (cyclic shear stress ratio) to start liquefaction for a given number of $(N1)_{60}$. The increase in CSR and $(N1)_{60}$, cause the decrease in risk of liquefaction. In the following section these parameters and some others are studied.

IDRISS – BOULANGER METHOD

In this method in contrary to previous methods by Seed & Idriss [13] or Seed & et, al. [14], the liquefaction potential evaluation is based on trial and error $(N1)_{60}$. By using some formulas or tables precise results can be achieved from the liquefaction potential evaluation. Furthermore in this paper Idriss-Boulanger formula based on SPT method has been used.

$$(N1)_{60} = N_{spt} .C_N.C_E .C_S .C_R \qquad (1)$$

In this formula the coefficients are correction factors for SPT. Where C_N is an overburden correction factor, CE = ERm/60%, ERm is the measured value of the delivered energy as a percentage of the theoretical free-fall hammer energy, C_R is a rod correction factor to account for energy ratios being smaller with shorter rod lengths, C_B is a correction factor for nonstandard borehole diameters, C_S is a correction factor for using split spoons with room for liners but with the liners absent, and N_{spt} is the measured SPT blow counts. The factors C_B and C_S are set equal to unity if standard procedures are followed[11], other correction factors are shown in Table 1.
The amount of C_N based on Idriss - Boulangr method can be measured by Eq. (2).

$$C_N=(P_a/\sigma'_v)^m\leq 1.7,\ \ Pa = 100\ Kpa \qquad (2)$$

$$m = 0.748-0.0768\sqrt{(N_1)_{60}} \qquad (3)$$

The Simplified Procedure For Estimating Cyclic Shear Stress Ratios Induced By Earthquake Ground Motions

seismic demand energy usually is defined on a layer of a soil by CSR. For this purpose, Seed–Idriss [13] simplified procedure is used to estimate the cyclic shear stress ratios (CSR) induced by earthquake ground motions, at a depth z below the ground surface, using the following Eq**.** (4):

Table 1. Standard Penetration Test (SPT) correction factors

Correction Coef	Index	Equpment pro	Title
1	C_B	65-115mm	Diameter Of Boreholes
1.05		150mm	
1.15		200mm	
0.75	C_R	3-4m	Length of Rod
0.85		4-6m	
0.95		6-10m	
1		10-30 m	
>1		More than 30 m	
1	Cs	Standard sampling	Sampling Method
1.1-1.3		Non Coating	
0.5-1	C_E	Donut Hammer	Energy Ratio
0.7-1.2		Safetly Hammer	
0.8-1.3		Automatic Donut Hammer	

$$CSR= \tau_{av}/\sigma'_v =0.65(\sigma_v/\sigma'_v)(a_{max}/g)\ r_d \qquad (4)$$

where σ_v = vertical total stress at the depth under consideration, σ' = effective stress at the depth under consideration, a_{max}/g = maximum horizontal acceleration (as a fraction of gravity) at the ground surface, and r_d = shear stress reduction factor that accounts for the dynamic response of the soil profile.
The values of CSR calculated using Eq. (4) correspond to the equivalent uniform shear stress induced by the earthquake ground motions generated by an earthquake having a moment magnitude M. It has been customary to adjust the values of CSR calculated by Eq. (4) 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=7.5, i.e. Eq. [5] $(CSR)_{M=7.5}$. Accordingly, the values of $(CSR)_{M=7.5}$ are given by:

$$(CSR)_{M}=7.5= CSR/MSF=0.65(\sigma_v/\sigma'_v)(a_{max}/g)\ r_d/MS \qquad (5)$$

Shear Stress Reduction Factor (rd)

Shearing stress reduction factor (r_d) has been introduced by Seed and Idriss , as a parameter that accounts for the dynamic response of the soil profile. As it has been displayed on Figure1. They have given $r_\mathbf{d}$ for the wide range of earth movement and earthquake[3].

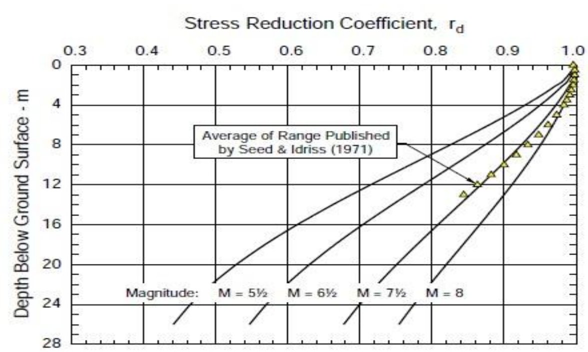

Fig.1. Variations of stress reduction coefficient with depth and earthquake magnitude

In extending the work of Golesorkhi [5,6], Idriss performed several hundred site data analysis and concluded that, for the purpose of developing liquefaction evaluation procedures, the parameter r_d could be expressed as depth and earthquake magnitude from formula (6a) which is accurate upto depth of 34 meters. The uncertainty in r_d increases with increasing depth such that Eq. (6a) should only be applied for depths less than about 20 ± m. Liquefaction evaluations at greater depths often involve special conditions for which more detailed analysis can be performed. For these reasons, it is recommended that CSR (or equivalent r_d values) at depths greater than about 20 m should be based on site response studies, providing, however, that a more accurate response calculation can be completed for the site (In this research the region soil liquefaction has been studied upto 20 meters depth.)

$$\begin{cases} r_d=\exp(\alpha(z)+\beta(z)M) \\ \alpha(z)=-1.012-1.126\sin(z/11.73+5.133) \\ \beta(z)=0.106+0.118\sin(z/11.28+5.142) \end{cases} \quad (6a)$$

If the depth of study is more than 34 meters, for figuring out shear stress reduction factor (r_d) equation (6b) can be used.

$$r_d=0.12\exp(0.22M) \quad (6b)$$

The relationship between the modified number of SPT ($(N1)_{60}$) and clean sand number $(N1)_{60cs}$is expressed by clean sand $\Delta(N1)_{60}$ .This parameter is based on the percentage of fine soil (FC), that has been expressed via Eq.(7) or Figure2. [10,9]. extracted.

$$\Delta(N1)=\exp[1.63+(9.7/(FC+0.01)) - (15.7/(FC+0.01))^2] \quad (7)$$

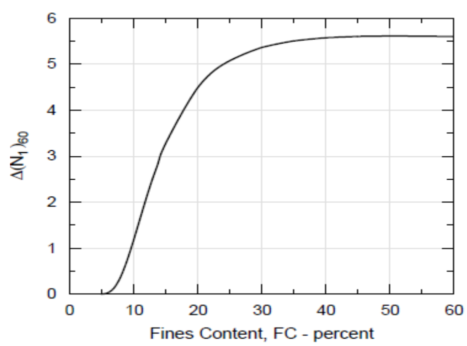

Fig.2. Variation of Δ(N1)60 with fines content.

According to the amount of $(N1)_{60}$ and $\Delta(N1)_{60}$, the amount of $(N1)_{60cs}$ is calculated via Eq.(8).

$$(N1)_{60CS} = (N1)_{60} + \Delta(N1)_{60} \quad (8)$$

The Cyclic Resistance Ratio (CRR)

Calculation of soil potential to liquefaction phenomenon is expressed by CRR. In Idriss-Boulanger formula, cyclic resistance ratio of soil (CRR) is calculated based on $(N1)_{60cs}$.[11]. In Eq.9 the amount of CRR is calculated for earthquake with magnitude of 7.5 .

$$CRR_{7.5,1\,atm} = \exp\left[\frac{(N_1)_{60cs}}{14.1} + \left(\frac{(N_1)_{60cs}}{126}\right)^2 - \left(\frac{(N_1)_{60cs}}{23.6}\right)^3 + \left(\frac{(N_1)_{60cs}}{25.4}\right)^4 - 2.8\right] \quad (9)$$

Whereas Idriss-Boulanger method is based on $(N1)_{60}$ & $(N1)_{60CS}$ and in most calculations these parameters interfere, therefore the diagram which is shown on Figure 3. has been obtained by Idriss-Boulanger to calculate the amount of CRR regarding the percent of fine soil based on $(N1)_{60CS}$ which is calculated by Eq. (8).

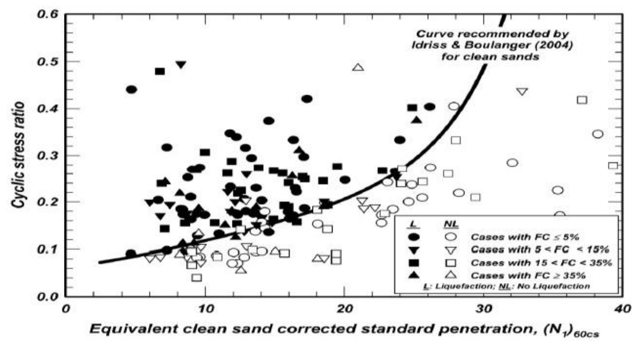

Fig.3 .SPT case history database used previously by Idriss and Boulanger

According to the status of increase in strain or liquefaction potential evaluation based on earthquakes other than 7.5 magnitude, Eq. (10) for CRR is used for correction. Considering that in this research the earthquake magnitude is 7.5, there is no need for MSF.
Since the semi-empirical liquefaction correlations are based primarily on data for level ground conditions and effective overburden stresses in the range of 100 ± kPa, Seed recommended that the CRR be corrected for these effects using the following expression:

$$CRR_{(M,K\sigma)} = CRR_{(M=7.5,1\,atm)}.MSF.K\sigma \quad (10)$$

Overburden Correction Factor, kσ

In which K_σ is the overburden correction factor and K_σ is the static shear stress correction factor. Revised K_σ relations are described in more detail by Boulanger [2] and by Idriss and Boulanger [7, 8], and so they are not reviewed herein.
When in a layer $\sigma'/Pa<1$, there is no need to correct for the soil under study. But if the aforesaid condition is not appointed then the result achieved from formula (10), must be corrected by k_σ according to formula (11a). By the way in formula (11a) factor C_σ is calculated by formula (11b).
This correction against last proposed corrected formulas in previous researches by Hynes and Olsen [4], Seed and Harder [15] is not based on relative density (D_R), but according to $(N1)_{60}$ it can be calculated by this formula :

$$K_\sigma=1-C_\sigma \ln((\sigma')_v/P_a) \quad (11a)$$

$$C_\sigma=1/(18.9-2.55\sqrt{(N_1)_{60}} \quad (11b)$$

Magnitude Scaling Factor, MSF

The magnitude scaling factor (MSF) is used to account for duration effects on the triggering of liquefaction. The MSF relationship was derived by combining 1- laboratory based relationships between the CRR and the number of equivalent uniform loading cycles, and 2-correlations of the number of equivalent uniform loading cycles with earthquake magnitude. The MSF factor is applied to the calculated value of CSR for each case history to convert to a common value of M (conventionally taken as M = 7.5). The MSF for sands was reevaluated by Idriss (1999), who recommended the following relationship or graph of Figure 4. [11]

$$MSF=6.9 \exp(-M/4)-0.058 \leq 1.8 \quad (12)$$

In this research all calculations are based on an earthquake with magnitude of 7.5 for Tabriz city. therefore the MSF factor is not included in the above calculations.

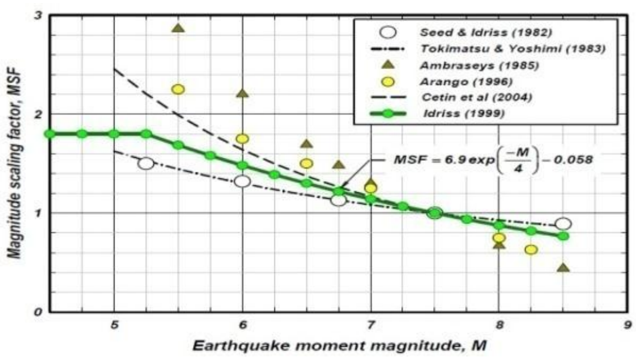

Fig.4. Magnitude scaling factor (MSF) relationship

The amount of safety factor in both methods are equal to formula (13), as follows. If FS< 1.0 then liquefaction occurrence in the considered depth is probable and if FS> 1.0 it will not be liquefied.

$$F.S.=CRR/CSR \quad (13)$$

LIQUEFACTION POTENTIAL INDEX

The Liquefaction Potential Index I_L , has been extended by Iwasaki[12] for predicting the risk of liquefaction potential. This index is interpreted by Iwasaki that if I_L= 0, the risk of liquefaction is very low, $0<I_L \leq 5$ risk of liquefaction is low, $5<I_L \leq 15$ risk of liquefaction is high and $I_L > 15$ risk of liquefaction is too high. So it is necessary to use some methods for decreasing the risk. The amount of liquefaction in the studied range can be achieved by formula (14).The amount of I_L is between 0 to 100 .

$$I_L = \int_0^{20} F.W(z).dz \quad (14)$$

On the above formula F is defined as an index. If FS ≤ 1.0, then F = 1- FS and if FS> 1.0 then F =0. In this formula W(z) is a weight function based on the depth for estimating the ratio of soil liquefaction that is being used in different depths. Z is the depth of the layer in which the liquefaction potential is being evaluated .

COMPARISON OF BOREHOLES

In this paper three boreholes as samples from 53 boreholes have been investigated and the results compared with each other and has been resulted in 3 figures that are shown in three different conditions. Further more by comparison of these differences between liquefaction and non-liquefaction situations are indicated. Boreholes specifications are shown in Table 2. As can be seen, perfect liquefaction, semi liquefaction and non-liquefaction situations are shown in figures 5, 6, and 7 respectively. [1]
The top graphs seismic force required to initiate liquefaction by (Load) and soil resistance to liquefaction phenomenon with (Resistance) is shown which are in order of the concepts CSR and CRR, according to the graphs, in which the soil resistance to liquefaction under seismic force it is the point where safety factor (F.S) was less than one and increases the risk of liquefaction (I_L).

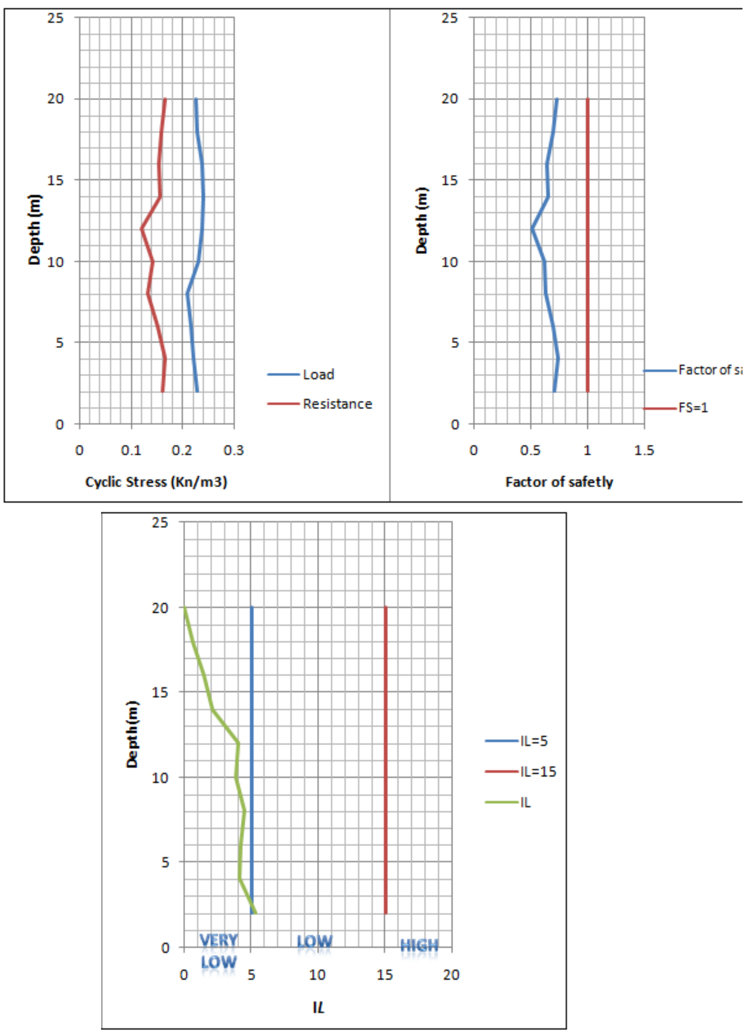

Fig.5 .Perfect liquefaction Condition (Boring No:C2B1); A: Liquefaction Load and Resistance Condition., B: Safety Factor, C: Liquefaction Risk

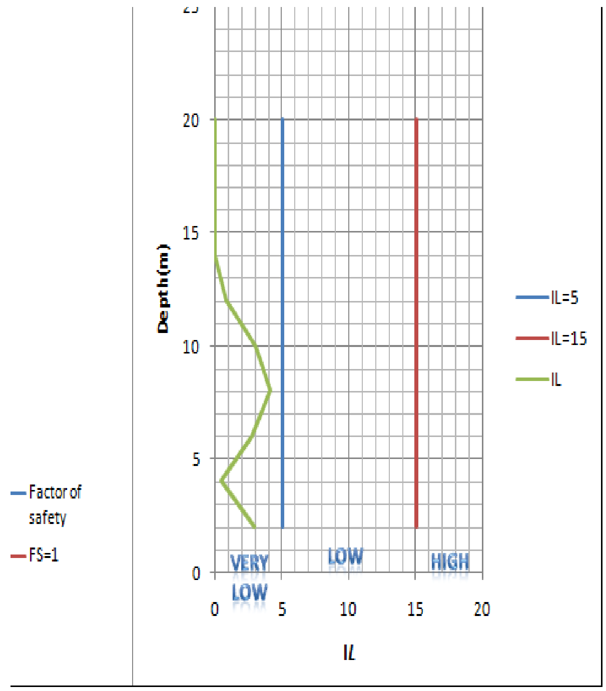

Fig.6 .Semi liquefaction condition, (Boring No:BH-19); A: Liquefaction Load and Resistance condnitio B: Safety Facto., C: Liquefaction Risk

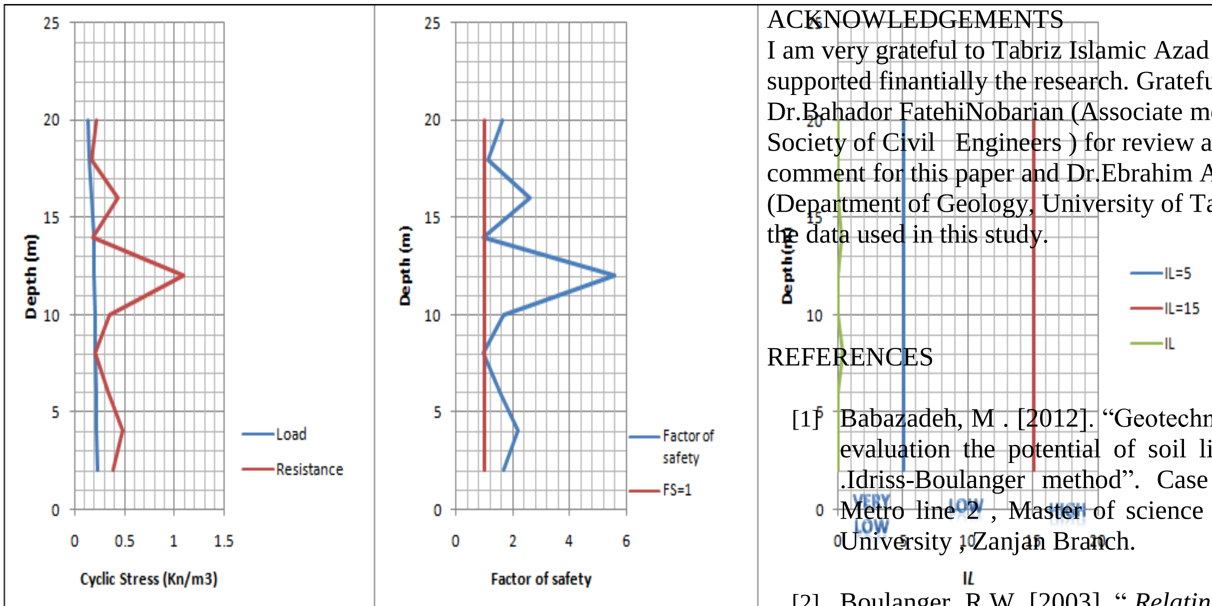

Fig.7.Non - liquefaction Cond., (Boring No:BH-3); A: Liquefaction Load and Resistance Condition , B: Safety Factor, C: Liquefaction Risk

CONCLUSIONS

This research has been carried out to study the liquefaction potential along the path of the Tabriz metro line 2 based on the results of Standard Penetration Test (SPT) using the latest method in liquefaction potential evaluation by Idriss and Boulanger.

In this paper three different condition between all boreholes have been investigated, and have resulted in here graphs (5-7) graphs Figures(5-7). In some areas in this subway there are risk of liquefaction. By comparison of the results achieved from current method it can be concluded that in some areas of Tabriz metro line2 there is in liquefaction condition, and even with a high risk of liquefaction potential.

Considering the climate similarities between Iran and U.S.A and considering that the evaluation of Idriss and Boulanger is on the base of too many geotechnical data in the U.S.A, by preparing different diagrams from these data , this conclusion can be achieved that the Idriss -Boulanger evaluation method for liquefaction potential is the best choice and as it is based on (N1)60 and trial & error process , the results can be near to the reality. Study of liquefaction potential of 53 boreholes from all areas in Tabriz city with the datum acceleration 0.35g for earthquake of magnitude 7.5 has been carried out. Practically in the layers that NSPT is bigger than 30, liquefaction has not been observed. According to Figures (5-7) in which resistance factor (CRR) is less than (CSR) or equal to it, liquefaction potential is high. In this case F.S is less than 1.0 . Considering the liquefaction risk analysis using Iwasaki & et. al method, the liquefaction risk is high in lower depths and near to ground along Tabriz metro line 2.

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