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Evaluation of Settlement in Soil Layers due to Liquefaction in Alluviums in South Eastern of Tehran

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ABSTRACT: Liquefaction in sand and silty soil layers due to earthquake is one of the important phenomena in earthquake geotechnical engineering. During the earthquake pore water pressure increases in soil layer until equal to total stress. Then, effective stress decrease to zero. In this condition soil layer loses its shear strength. The results of this phenomenon in soil layer are instability, wide settlement in ground level and underground. In last two decades, several empirical methods have been proposed for determining settlement in soil layers due to liquefaction based on experimental and field tests. Main idea in this research is evaluation of probable settlement of soil layers in up and below ground water level due to liquefaction in south eastern of Tehran. In this research, firstly liquefaction potential of soils in study area based on Standard penetration test (SPT) results was evaluated. Then, liquefaction potential index (LPI) for determining severity of liquefaction assessed. In final, value of settlement in soil layers (up and below ground water level) was calculated. Results of this study showed that with considering ground water table and SPT blow count in study area can be found there is low to moderate risk of liquefaction due to earthquake in future. Also, Analysis of data showed that volumetric strain value in saturate soil layer with increasing relative density goes down to significantly. Also, with increasing maximum shear strain due to earthquake in soil layer volumetric strain growth. Although, percentage of fines content in soils affect to rate of volumetric strain and in volumetric strain analysis in soil layers above ground water table showed that first rate of ε_v relative to saturate condition is more. Second, in all of relative density values volumetric strain with increasing fines content in soil layer go up to gradually.

Key words: Liquefaction, Settlement, Liquefaction potential index, Standard penetration test, South eastern of Tehran alluvium.

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INTRODUCTION

When loose sand is subjected to seismic shaking, it tends to volume reduction and settlement. The density of the under layers is revealed in the settlement of the ground surface that causes the destruction of the structures located on such surface. In dry sand layer, settlement in severe shakings occurs under a constant and effective stress condition and very rapid tension. In this regard, the sand deposit settlement is completed before the end of an earthquake, but if the sandy soil layer is saturated and drainage is limited the condition is prepared of fixed volume situation and the major effect of the seismic shocks is generation of exceed pore water pressure. Therefore, the deposit settlement of saturated sand requires a longer time, varying from a few minutes to a few days, depending on the permeability and compressibility of the soil and the length of the drainage path. However, sand settlement under the effect of the seismic loading is difficult to estimate and the possibility of error, between 25% and 50%, is even higher than the common values of the error in the estimation of the settlement of the stability (Ishihara and Yoshimine, 1992). The main purpose of the present study is to evaluate the probable rate of settlement in the soil layers at south and south eastern of Tehran alluvium and correlation with liquefaction potential index (LPI), discussed in the following paragraphs.

Liquefaction and settlement

If saturated sand sediment goes under seismic vibration it tends to compress and undergoes volume reduction in the absence of drainage possibility due to an increase in pore water pressure. If the pore water pressure in the sand deposit increases due to a continuous vibration, its quantity may sometimes be equal to the total

stress. Based on the concept of the effective stress, it can be written: $\sigma' = \sigma - u$ (1)

In the above equation the effective stress is σ′ and the total stress is σ and the pore water pressure is u, and, if σ is equal to u, σ ′ is equal to zero. Under this condition, the sand can be fluid, lacking in shear resistance. Such a condition is called liquefaction. Liquefaction of saturated sand during an earthquake is a damaging factor for buildings, earthen dams, and retaining walls and etc. The magnitude of an earthquake and its duration, the void ratio, the relative density, the fines content and soil types, the ratio of over consolidation and the range of shear stress inflicted on the mass of the soil are factors that influence the occurrence of liquefaction. In recent years, various field methods have been employed to assess this phenomenon. The standard penetrations method (SPT) (Seed and Idriss, 1971; Seed et al., 1983; Idriss and Boulanger, 2006 and 2010; Noutash et al., 2012; Ghasemian et al., 2017), Cone penetration method (CPT) (Robertson and Wride, 1998) and geo-seismic tests by measuring the velocity of the shear wave can be mentioned among field methods (Andrus and Stokoe, 1997; Andrus et al., 2004; Dabiri et al., 2011). The tendency of sand to become compressed while under earthquake vibration has been studied and analysed. The soil layer compression appears as settlement on the ground surface. The settlement due to an earthquake causes fatigue in the structures located on the shallow foundations and the destruction of the facilities serving the structures located on the piles, besides damaging the vital piles commonly buried in lower depths. The dried sand compaction occurs rapidly; typically, the settlement of a sand mass is completed after an earthquake, but the settlement of saturated sand requires a longer time. The settlement occurs when the pore pressure caused by earthquake is dissipated. The required time for settlement depends on the permeability and density of the soil and the length of the drainage course, with the time varying from a few minutes to several days. It is difficult to determine the settlement caused by an earthquake. The errors between 25% and 50% are common in the static settlement prediction and these errors increase in the case of more complicated loading of the earthquake (Askari et al., 2010). The rate of the settlement in the sand layers, based on the field test in the two states of the dried layers (Silver and Seed, 1971; Pyke et al., 1975) and the saturated layers (Lee and Albaisa, 1975; Tatsouka et al., 1982; Tokimatsu and Seed, 1987; Shamato et al., 1998; Wu, 2004; Cetin and Unutmaz, 2004; Chen et al., 2016; Oshnaviye and Dabiri, 2017) are evaluated. In continue, the general conditions and the layering of the soil in the study area and summary of the standard penetration used for assessing the liquefaction potential and then the evaluation method of the probable settlement in both the dried and saturated states in the area under study is mentioned and then, the results of the study are explained.

Geology and general conditions in study area

Tehran plain mainly consists of Quaternary formation. This formation is often a result of erosion and re-deposition of former sediments. It is extended to the south as a young fan and generally consists of unsorted fluvial and river deposits. Both, the effects of climate processes and tectonic young activities caused a miscellaneous alluvium of type, thickness and grain size to be formed. For the first time, Rieben (1955) and then Pedrami (1981) divided the Tehran plain into five units as shown in Figure 1. These units include units A and Bn in the north, unit Bs in the south, unit C in the north, west and centre and unit D in the centre and south of Tehran plain. Border of these units at the north of Tehran was further defined by Abbasi and Shabanian (1999). The general characteristics of the different units of Tehran plain are shown in Table 1. The data on the type of the materials, shown in Table 1, has been provided on the basis of the information obtained from about 700 drilled boreholes throughout the area (Jafari et al. 2001).

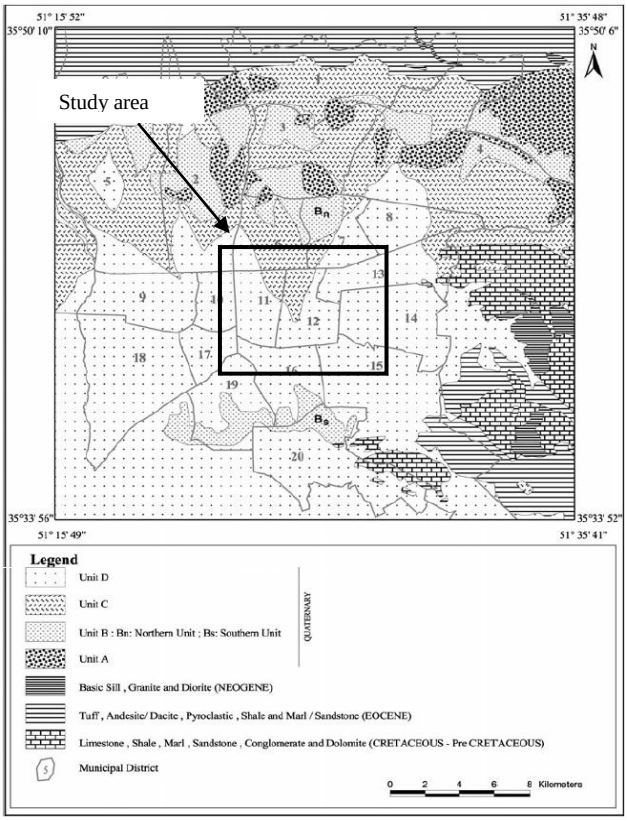

Figure 1. Geology map of Study Area (Jafari et al. 2001)

Table 1. Characteristics of different units in Tehran Plain (Jafari et al. 2001)

Unit	Period	Formation	Constituting materials
A	Plio-Pleistocene	Hezardareh	Congelomerate with silt-sand-gravel and silt-clay mixtures
Bn	Quaternary	Hezardareh	Cobble, boulder, gravel and sand
Bs	Quaternary	Kahrizak	Silty sand
C	Quaternary	Kahrizak	Gravel, sand, silt and clay
D	Quaternary	Kahrizak	Silt and clay

Assessment of liquefaction

In order to evaluate the liquefaction potential of soils using two field methods, geotechnical information of 67 boreholes in the south and southeast of Tehran including 11 to 16 municipality areas were collected (Figure 1). As mentioned before, the types of soil and geotechnical properties can affect the liquefaction potential. In this study, the gravely sand, silty sand and silty soils were studied. Ground water level is one of the main parameters in in soil liquefaction potential evaluation of soils. Variation of water level in boreholes have been proposed in Figure 2. The peak ground acceleration (PGA) is necessary for the analysis of boreholes to evaluate liquefaction potential of soils. According to Figure 3, PGA values were selected in each boreholes position (Shafiee et al., 2011).

In the assessment of the liquefaction potential of the soils in the study area, the simplified method by Idriss and Boulanger (2006) is used together with the results of the standard penetration test, so that, first, the quantiy of cyclic stress ratio (CSR) is estimated expressing the rate of the severity of the earquake load in a Mw=7.5 that is estimated using the equation bellow:

$$CSR_{7.5} = 0.65 \frac{a_{max}}{g} . \frac{\sigma_V}{\sigma'_V} . r_d . \frac{1}{MSF} \quad (2)$$

In the above equation, a_{max} is the peak ground acceleration, g is acceleration of gravity, σ_V total stress in the depth in the question, σ'_V effective stress in the same depth, r_d coefficient of shear stress reduction using the form Figure 4 is estimated and MSF (Magnitude Scale Factor) is earthquake magnitude scale factor that is calculated based on Andrus and Stokoe in 1997 using equation 3. In this equation Mw is earthquake magnitude:

$$MSF = \left(\frac{M_W}{7.5} \right)^{-3.3} \quad (3)$$

In order to determine to cyclic resistance ratio (CRR) of the soils simplified and modified method by Seed et al. (1985) are used. For this, first the results obtained from the standard penetration test are presented, based on the following equation by the application of the presented parameters by Skempton (1986) that are modified in Table 2.

$$(N_1)_{60} = N_{SPT} \times C_N \times C_E \times C_B \times C_R \times C_S \quad (4)$$

In this equation, 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 (2006, 2010), the overburden tension correction factor (C_N) is determined using the following equation:

$$C_N = \left(\frac{P_a}{\sigma'_V} \right)^{\alpha} \leq 1.7, P_a = 100kPa \quad (5)$$

$$\alpha = 0.784 - 0.0768 \sqrt{(N_1)_{60}} \quad (6)$$

In the above equation, Pa = 100kPa, is the atmospheric pressure and σ'_V is the effective stress at the depth in question, and $(N_1)_{60}$ is corrected the number penetration resistance test standard. After the modification of the number of the standard penetration test, its equal quantity is determined $(N_1)_{60CS}$ for clean sand, and then cyclic resistance ratio (CRR) is assessed by the application of the following equations (Figure 5):

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60} \quad (7)$$

$$\Delta(N_1)_{60} = 1.63 + \exp\left(1 + \frac{9.7}{FC + 0.1}\right) - \left(\frac{15.7}{FC + 0.1}\right)^2 \quad (8)$$

$$CRR = \exp\left(\frac{(N_1)_{60CS}}{14.1}\right) + \left(\frac{(N_1)_{60CS}}{126}\right)^2 - \left(\frac{(N_1)_{60CS}}{23.6}\right)^3 + \left(\frac{(N_1)_{60CS}}{25.4}\right)^4 - 2.8) \quad (9)$$

In equations mentioned above FC is equal fines content in soil layer.

In the calculation of the CRR, if the amount of effective vertical stress at the depth under the study is more than 100 kPa, the CRR value is modified by using the following equation: $CRR_j = K_\sigma . CRR$ (10)

In this equation, the CRR_j is corrected cyclic resistance ratio. Furthermore, the Kσ parameter is a coefficient based on the effective vertical stress that is calculated by the following (Hynes and Olsen, 1998):

$$K_\sigma = \left(\frac{\sigma'_V}{100} \right)^{f-1} \quad (11)$$

Where K_σ is the overburden correction factor, σ'_V is the effective overburden 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 6). Safety factor (Fs) against liquefaction in soil layers is calculated using the following equation:

$$F_S = \left(\frac{CRR_j}{CSR} \right) \quad (12)$$

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

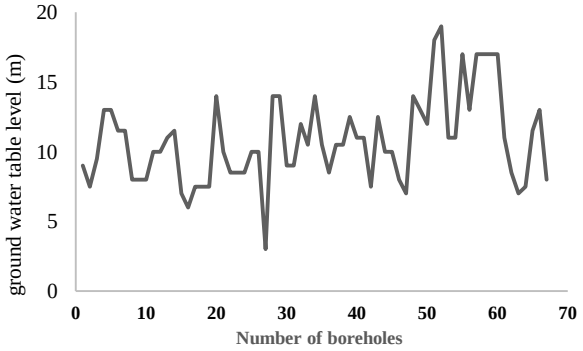

Figure 2. Variation of ground water table level in study area

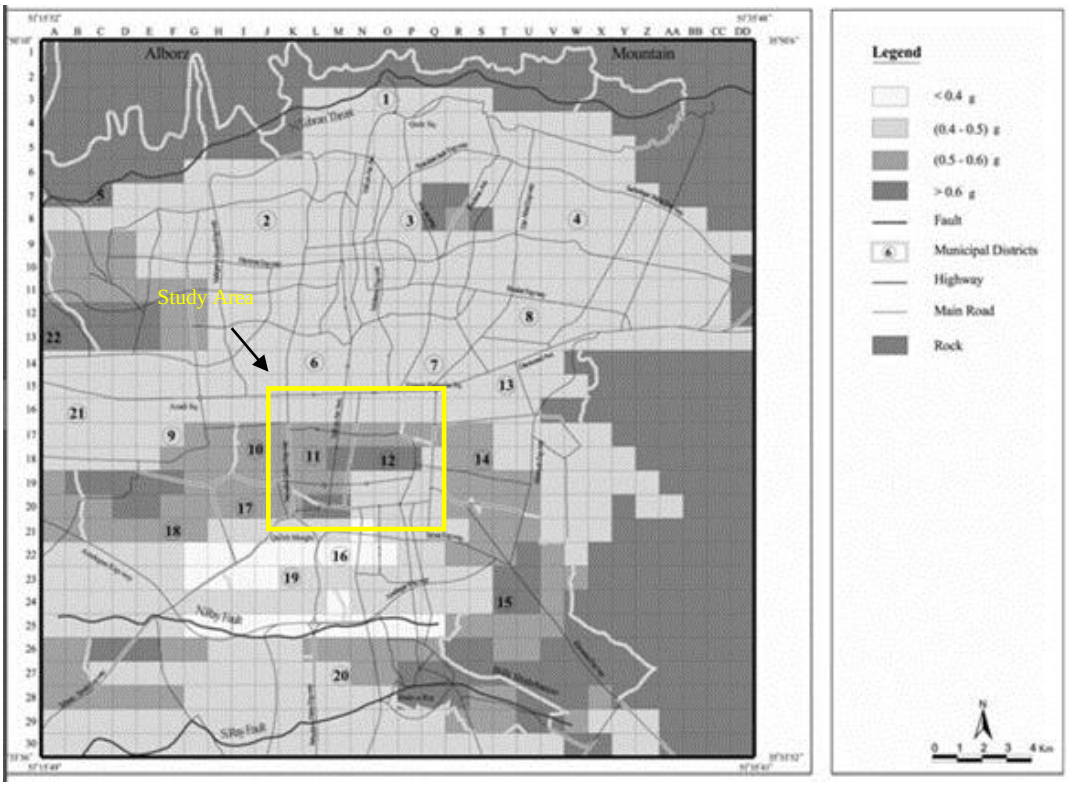

Figure 3. The study area and PGA distribution throughout Tehran for an earthquake corresponding to 475 year return period (Shafiee et al., 2011)

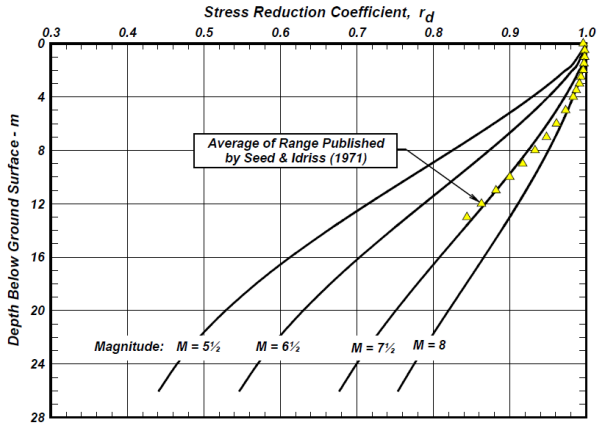

Figure 4. Variations of stress reduction coefficient with depth and earthquake magnitudes (Idriss, 1999)

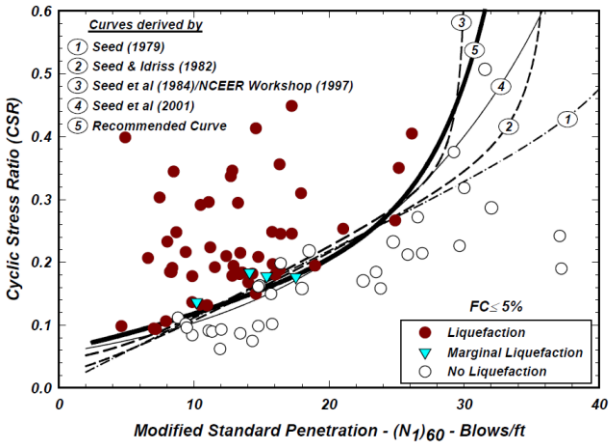

Figure 5. Liquefaction resistance curve for the earthquakes of 7.5 magnitudes (Idriss and Boulanger, 2010).

Table 2. Correction factor of SPT (Skempton, 1986)

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

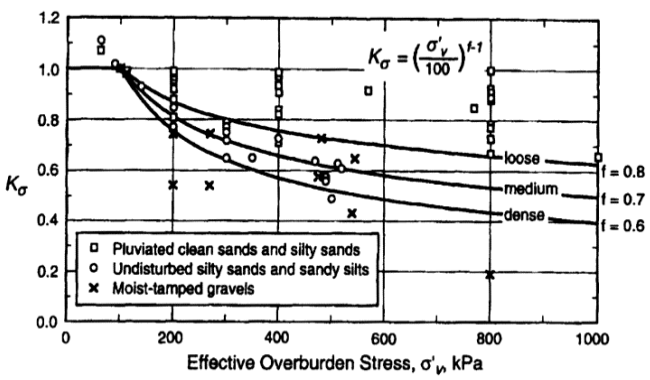

Figure 6. Variations of K_σ values versus effective overburden stress (Hynes and Olsen, 1998)

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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 that of Iwasaki et al. (1978, 1982), presented in the following equation:

$$LPI = \int_0^{20} W(Z).F(Z)dz \quad (13)$$

$$F(Z) = 1 - Fs \quad \text{For Fs} < 1 \quad (13a)$$

$$F(Z) = 0 \quad \text{For Fs} \geq 1 \quad (13b)$$

$$W(Z) = 10 - 0.5Z \quad \text{For Z} < 20 \text{ m} \quad (13c)$$

$$W(Z) = 0 \quad \text{For Z} > 20 \text{ m} \quad (13d)$$

In the above equation, Z is the depth of midpoint in question. 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).

Table 3. Liquefaction potential index (LPI) and its describes (Iwasaki et al., 1978, 1982)

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)

Evaluation of settlements due to liquefaction

In the present study, calculation of the settlement amount or the volumetric strain in soil after liquefaction was done in two parts. The first part was analysed in the soil layers above the groundwater level and the second part in the soil layers under the water table (it means in the saturated mode) as described in the following:

Tokimatsu and Seed (1987) method was used in order to determine the soil above the water table level in of the boreholes, whose chart of information is seen in Figure 7. In order to determine the volumetric strain (ε_v) in the upper layers of groundwater level in borehole logs (67 boreholes) in south and south eastern of Tehran city has taken the following:

1- The relative density (D_r) on a soil layer by using the formula provided by Idriss and Boulanger (2010) was determined according for a number of standard penetration resistance tests.

$$D_r = \sqrt{\frac{(N_1)_{60}}{46}}.100 \quad (15)$$

2- Using the formula provided by Tokimatsu and Seed (1987) cyclic shear strain in question layer were determined as follows:

$$\gamma_{cyc} = 0.65.\frac{a_{max}}{g}.\frac{\sigma_v}{G_{max}}.rd \quad (16)$$

In the above equation, rd is the stress reduction factor, a_{max} the peak ground acceleration, σ_v represents the total stress at the depth in question, and G_{max} is the maximum shear modulus calculated by below equation that proposed Tokimatsu and Seed (1987) based on kN/m^2:

$$G_{max} = 4400.((N_1)60)^{0.33}.(\sigma_V')^{0.5}) \quad (17)$$

3- After the calculation of the volumetric strain (ε_v) using the Figure 7 in each layer of the soil borehole logs settlement in each layer is determined using the following equation:

$$\Delta H = (\frac{\varepsilon_v}{100}) \times h \quad (18)$$

In the above equation, h is the thickness of layer in question. Finally, for each borehole log, the total of settlement of the each soil layers above the groundwater level is accumulated in meter.

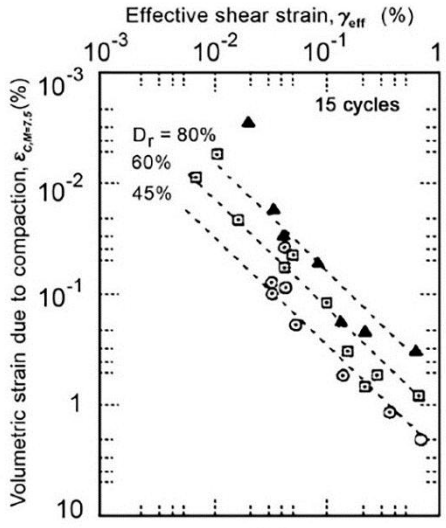

Figure 7. Correlation between volumetric strain and shear strain for Mw=7.5 based on relative density (Tokimatsu and Seed, 1987)

Also, Tokimatsu and Seed (1987) method was used for determining soil settlement and volumetric strain at the below groundwater level in 67 boreholes of the study area following procedure:

1-the cyclic stress ratio (CSR) due to earthquake is estimated with using equation (2) for each soil layers of boreholes in the underground water level.

2- Then with using Equation 7, clean sand equivalent to the number $(N_1)_{60CS}$ standard penetration resistance test for soil layers below the water table was determined.

3- The volumetric strain rate (ε_v) is calculated by determining both the CSR and $(N_1)_{60CS}$ for each soil layer under the study with using the diagrams shown in Figure 8.

4- Finally, after calculating the volumetric strain (ε_v) in each layer of the soil, settlement values was determined in meters for each layer of boreholes with using equation 18 and then total amount of settlement of layers below the water table is calculated.

Finally, after accumulating the amount of settlements of soil layers in the upper and lower of the groundwater level in each borehole study area the total value of settlement calculated for them.

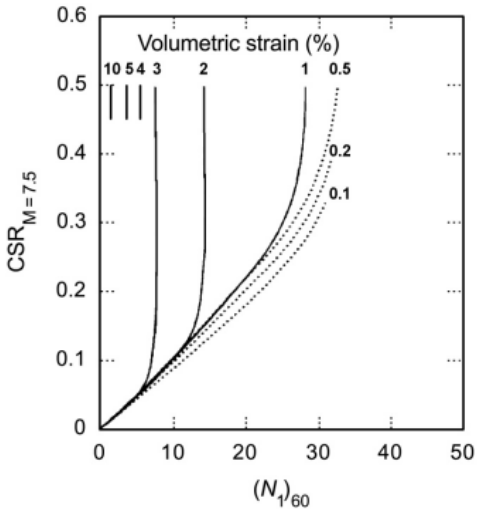

Figure 8. Evaluation of volumetric strain in saturated sand based on CSR and $(N_1)_{60}$ values, Mw=7.5 (Tokimatsu and Seed, 1987)

RESULTS

Results of this study can be explained in below:

1- In 67 boreholes was collected in study area generally 538 soil layers assessed. In terms of type 252 sandy, 220 silty and 66 gravelly layers were evaluated. According to Figure 9 distribution of N_{SPT} values can be seen. In general N_{SPT} amounts are between 10 and 70. Also, in accordance with Figure 10 can be observed that safety factor values against liquefaction about in 30 and 40 percent of saturated soil layers are less than 1.

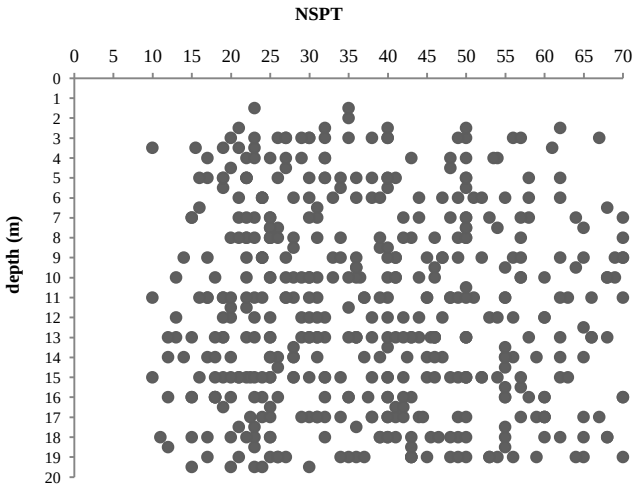

Figure 9. Variations of N_{SPT} versus depth in study area

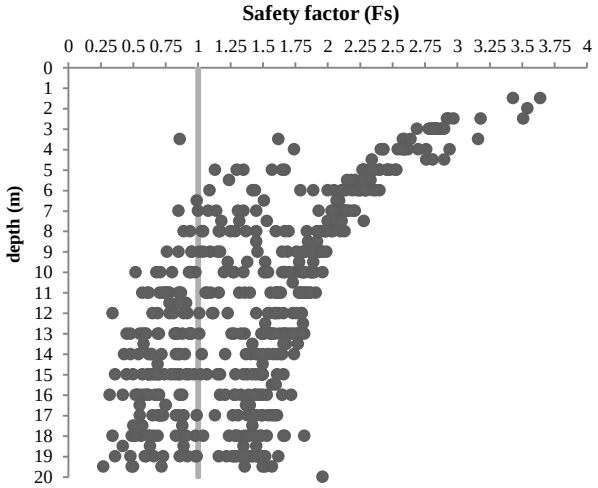

Figure 10. Variations of safety factors in soil layers versus depth based on Idriss-Boulanger method in study area.

2- Variations of liquefaction potential index (LPI) based on Iwasaki et al. (1978 and 1982) in study area is proposed in Figure 11. As can be observed according to Table 3 criterion about 80 percent of boreholes have LPI between 0 and 5 which state that they have low to moderate liquefaction risk. In the same way, almost 20 percent of boreholes have moderate to high liquefaction risk. In other words, study area with consideration ground water level generally has liquefaction risk.

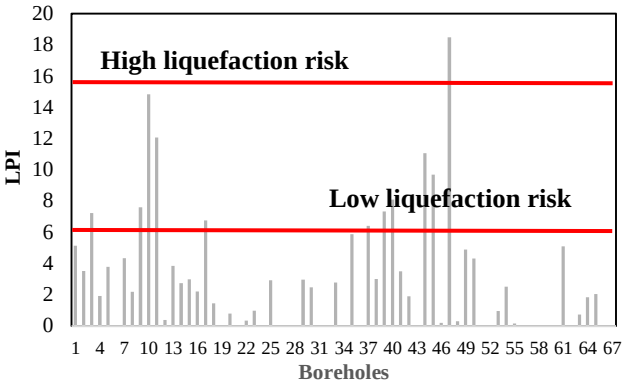

Figure 11. Values of LPI in boreholes in study area

Table 4. Values of LPI in boreholes in south and south east of Tehran

Liquefaction potential index (LPI)	LPI=0	0<LPI<5	5<LPI<15	LPI>15
Number of boreholes	22	32	13	1
Percent (%)	32%	47%	19%	2%

3- Probable total settlement due to liquefaction in soil layers (collection of up and below ground water level) at study area can be seen in Figure 12a. Maximum settlement can occur due liquefaction in soil layer is more than 0.25m. Also, rate of total settlements in soil layers compared with liquefaction potential index. It was showed there is good agreement between both of parameters

(Figure 12a, b). According to Table 5 boundary values of probable settlement accordance with LPI is proposed.

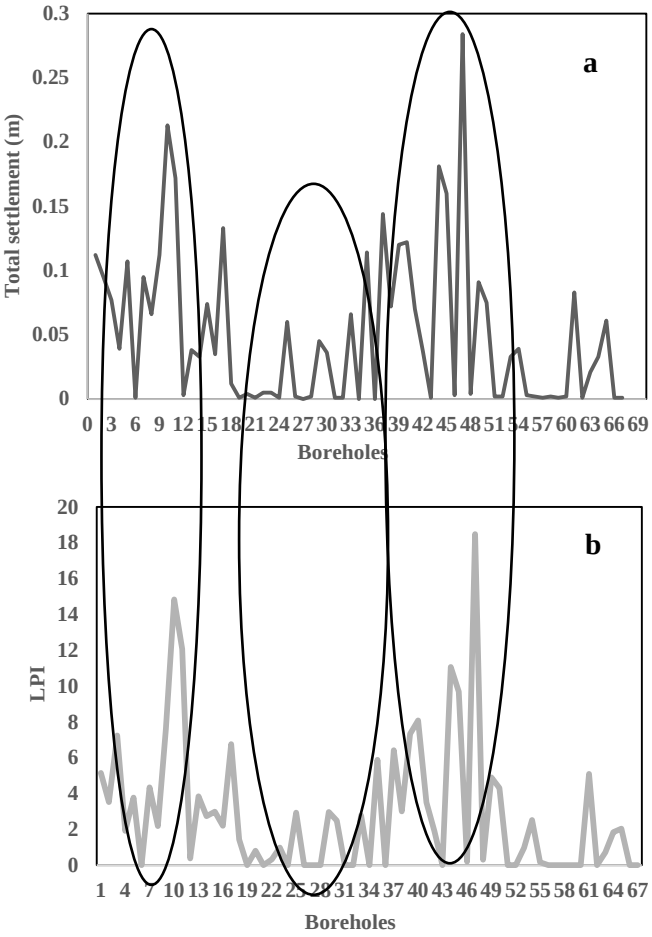

Figure 12. Comparison of total settlements with LPI in boreholes at study area

Table 5. Comparison of boundary values of total settlements with LPI in boreholes

Liquefaction potential index (LPI)	LPI=0	0< LPI <5	5< LPI <15	LPI >15
Total settlement in soil layer (m)	0.001<ΔH<0.002	0.002<ΔH<0.107	0.107<ΔH<0.213	ΔH>0.213
Number of boreholes	22	32	13	1
Percent (%)	32	47	19	2

4- In saturate soil layers effect of relative density on volumetric strain can be observed in Figure 13. Accordingly, with increasing relative density volumetric strain significantly decreases. Likewise, with growth fines content in soil layer volumetric strain went down to considerably.

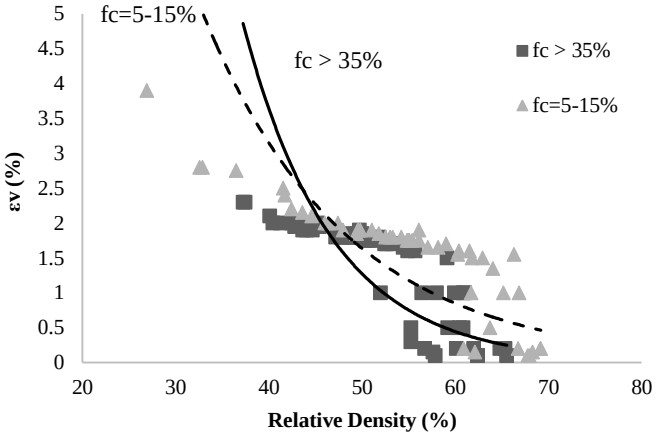

Figure 13. Effect of relative density on volumetric strain in saturate soil layer.

5- Effect of maximum shear strain (γ_{max}) due to earthquake in volumetric strain of saturate soil layers can be found in Figure 14. Generally, with increasing shear strain in soil volumetric strain go up. Although, rate of growth ε_v in fines soils (fc > 35%) relative to granular soil layers (with fc=5 to15%) is slow.

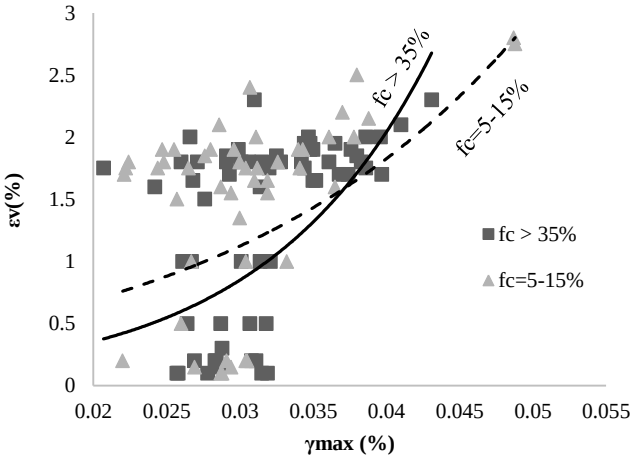

Figure 14. Effect of maximum shear strain on volumetric strain in saturate soil layer.

6- Variations of volumetric strain versus maximum shear strain in soil layers located above ground water table can be observed in Figure 15 (a, b). With considering of relative density it is observed that in loose to moderate dense soils volumetric strain with increasing γ_{max} go up. Similarly, this condition can be seen in very dense soils. Despite the fact that in relative density between 40 and 70% behaviour of soil in low maximum shear strain (γ_{max} < 0.015%) similar to each other. Also in contrast with saturate soil layers, in dry soil layers with increasing fines content (fc > 35%) volumetric strain is more than granular soils (fc=5-15%).

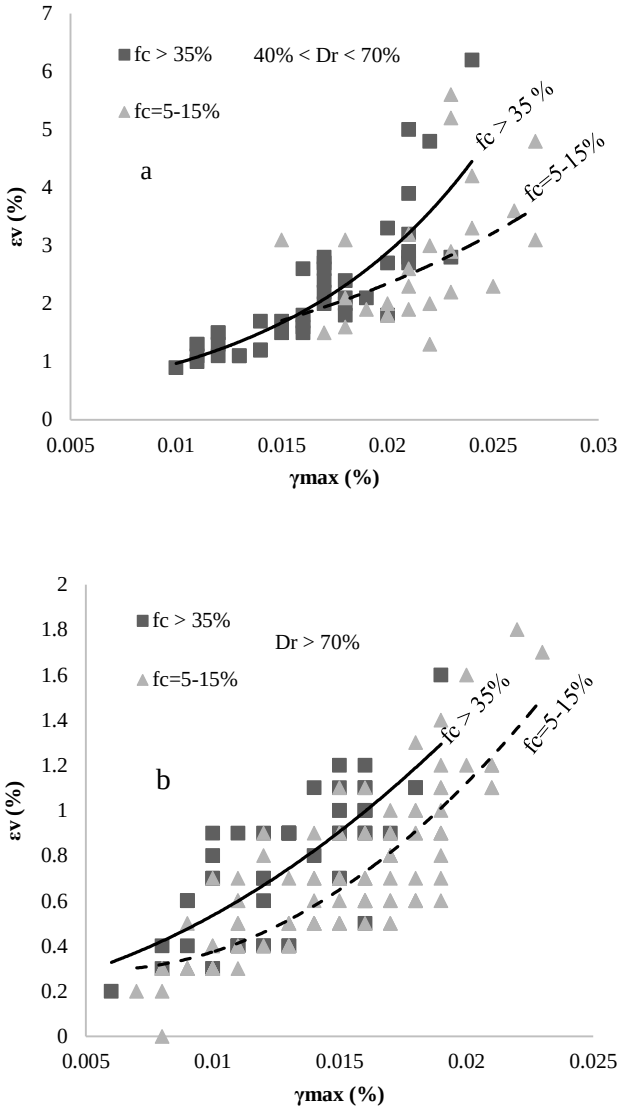

Figure 15. Variations of volumetric strain versus maximum shear strain in dry soil layers based relative density a: 40% < Dr < 70%, b: Dr >70%.

CONCLUSION AND DISCUSSION

As mentioned earlier, main aim of this study evaluation of probable settlement in soil layers due to liquefaction in south and south eastern of Tehran city. Liquefaction potential evaluation of soils have been performed according to Idriss and Boulanger (2006, 2010) method. Then, Liquefaction potential index (LPI) assessed based on Iwasaki et al. (1978, 1982) procedure. Finally, settlements of soil layers in above and below of ground water table have been evaluated. Almost 67 boreholes collected in study area. Results of this study can be explained follow:

1- Similar to results of Noutash Khalil et al. (2012) study, with considering ground water table and SPT blow count in study area can be found there is low to moderate risk of liquefaction due to earthquake in future.

2- Results of settlement in soil layer due to liquefaction showed that ground deformation is probable and about maximum settlement equal 0.25m can occur. As well as, values of LPI analysis approve this condition. With considering of boreholes position distribution risk of settlement in south part of study area (i.e. 11, 12, 16 and 19) is high.

3- Analysis of data showed that volumetric strain value in saturate soil layer with increasing relative density goes down to significantly. Also, with increasing maximum shear strain due to earthquake in soil layer volumetric strain growth. Although, percent of fines content in soils affect to rate of volumetric strain.

4- Volumetric strain analysis in soil layers above ground water table showed that first rate of ε_v relative to saturate condition is more. Second, in all of relative density values volumetric strain with increasing fines content in soil layer go up to gradually. Also, similarly with growth of maximum shear strain value of ε_v increases. Means that is fines soil located between granular particles can cause increasing in void ratio and then affect in volumetric strain.

In sum up, settlement due to liquefaction in soil layers is one of the important phenomena in geotechnical earthquake engineering. As has been noted, maximum rate of settlement in soil layers in study area is equal 0.25m which should be considered. Accordingly, serious damages can be inflicted to buildings, underground structures and life lines. Similarity of these results has been observed by Oshnaviye and Dabiri (2017) and Ghasemian et al. (2017) in their researches. Therefore, it is suggested in future researches with using empirical and numerical (or soft computing) methods based on field and experimental tests results a detailed assessment conducted and influence of various parameters on settlement of soil layers be determined until suitable methods for soil improvement proposed.

DECLARATIONS

Author's contributions

All authors contributed equally to this work.

Competing interests

The authors declare that they have no competing interests

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