Comparison of Liquefaction Potential Evaluation based on SPT and Energy methods in Tabriz metro line 2

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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. The use of energy is a logical step in the evaluation of liquefaction assessment methods. Main ideas this method is included: firstly effect of different parameters in liquefaction potential of soils be determined. Secondly, reliability of results as an affecting parameters in engineering decisions obtained. In this research results of two liquefaction potential methods based on standard penetration test (SPT) and energy were compared. Case study area is Tabriz Metro Line 2 with 22km length and 54 boreholes was collected. With considering type of soils and ground-water table level liquefaction potential evaluated. Then, liquefaction potential index (LPI) assessed. Results of this study showed that there is no suitable accordance between two processes. Moreover, energy method proposed higher liquefaction potential hazard than SPT. Major Factor affecting in un matching can be explained such as distance of Tabriz Metro Line 2 to Tabriz north fault, epicenter position and assumption of correction factors in SPT method.

Keywords: Liquefaction; Tabriz Metro Line 2; Liquefaction hazard; Energy method; Standard Penetration Test (SPT).

1- Introduction

Liquefaction is one of the most important events in earthquake geotechnical engineering leading to destructive damages. By raising pore water pressure in saturate loose granular soil layers (e.g., fine gravel or sand) and clay (in a special situation) and by reducing volume as a result of earthquake or seismic loading, effective confining stress decreases. In this condition, the shear strength of soil sharply declined and equals zero. This phenomenon leads to lateral spreading, settlement, sand boiling, and water leakage from voids in the ground. Several elements influence the occurrence of liquefaction including magnitude of earthquake, void ratio, relative density, and fines content percentage. New methods for assessing liquefaction are based on absorbed strain energy due to earthquake in soil. Several researches have been performed for evaluating liquefaction potential of soils with using energy theory such as Davis and Brill (1982), Brill and Davis (1985), Law *et al*. (1990), Running (1996), Trifunac (1995), Kayen and Mitchell (1997) and Green (2001). Also, some researches proposed based on numerical models statistical theories for energy method such as Trifunac and Todorovska (2004), Chen at al. (2005), Baziar and Jafarian (2007), Jafarian *et al*. (2011), Alavi and Gandomi (2012), Jafarian *et al*. (2014) and Kokusho and Mimori (2015). In this research, Davis and Brill (1982) method used for evaluating liquefaction potential of soil layers based on energy. Then, results were compared with Idriss and Boulanger (2010) process based on standard penetration test (SPT) blow counts. In final, liquefaction potential index (LPI) for both of method calculated with using Iwasaki *et al*. (1978, 1982) method. Study area Tabriz Metro Line 2 was selected. For study area different researches about geotechnical properties and liquefaction potential of soil layers have been carried out can be cited to studies of Ghasemian *et al*. (2017), Oshnaviye and Dabiri (2017), Mohammadi *et al*. (2015), Barzegari *et al*. (2014) and Ghobadi *et al*. (2010). In continue, first geology and general conditions of study have been described. Second, manners used for this researches have been expressed. Then, results of analysis compared.

2- Geology and General Condition in Study Area

The city of Tabriz is surrounded by the Eynali (Oon-Ebne-Ali) mountain range in the east-west, and not-so-high consolidated alluvial deposits and conglomerates in the south. The general slope of the plain is towards the west and, as a result, the direction of the general drainage of the surface and underground water is also westward. The surface of the plain is generally covered by alluvial deposits. The average height of the city of Tabriz is 1340 metres above sea level. The difference between the highest and the lowest points of Line 2 of the Metro route is 285 metres (Fig. 1).

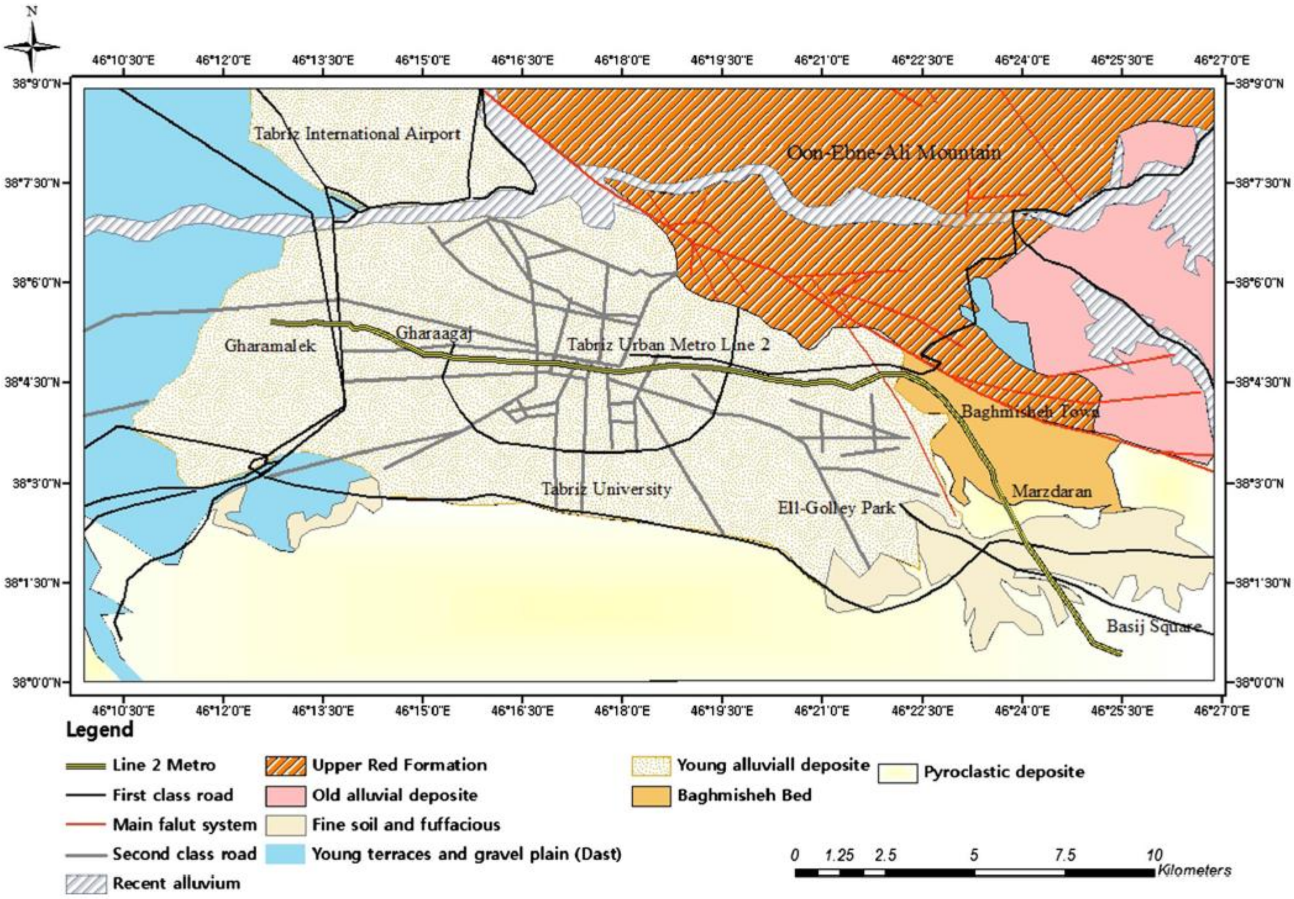

Figure 1) Geology map of Tabriz and Tabriz Metro Line 2 (Ghobadi et al., 2016).

2.1- Soil stratification in study area Azerbaijan, with respect to stratigraphy, has a long period of expansion and the surroundings of the Tabriz plain also have extensive Cambrian outcrops, but the stones and the alluvium in the area of Tabriz do not date back to such a time period with their formation components being related to the Cenozoic and Quaternary periods. The Cainozoic component in the Tabriz plain started from the Miocene Age and lasted up to the Quaternary era. The alluvial of the fourth period including, soft to hard conglomerates, is located on this sediment.

Table 1. Geological sequances and formations of Tabriz (Ghobadi et al., 2016)

Quaternary	Alluvium
Pliocene	Fish beds (marl, lapilli, diatomite)
Miocene	Baghmishe formation (marl with shale and lignite) Upper red formation (marl, sandstone, claystone with layer of gypsum)

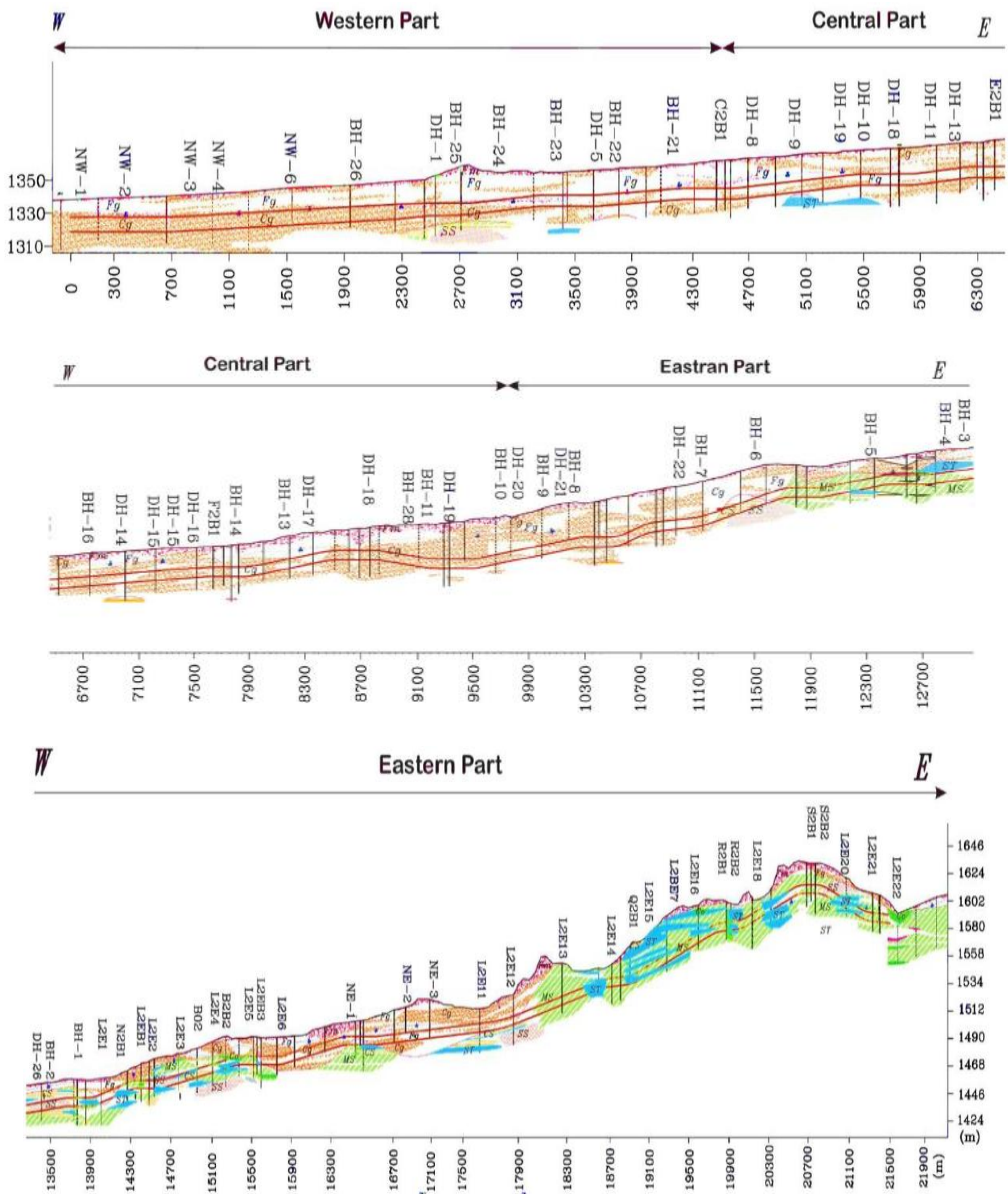

Figure 2) Geology section along Tabriz Metro Line 2 (Mohammadi et al., 2015).

The Line 2 of the Tabriz city subway, from its starting point in the west to the Baghmishe town, is covered with alluvial sediment, which , moving west, develops layers of marl and clay stones and siltstone or outcrop comprising a thin covering near the surface of the land. Under the alucia depostes of the Abbasi street towards the east, there are marn and sandstone conglomerate layers at a dept of less than 10 metres. The geological sequances and formations of Tabriz is shown in Table 1. Also, geology section along Tabriz Metro Line 2 is proposed in Figure 2.

2.2- Structural geology and earthquake

The city of Tabriz is located in the west The Alborz zone and follows the tectonic regimes ruling it. The forming of the Tabriz plain sediment in it and the formation of tectonic structures that often emerge as fractures or faults follow this system. The Tabriz plain is surrounded in the north by the mountains of Eynali and on the south by the volcanic altitudes of Sahand and its pyroclastic sediments. The reverse function of the north Tabriz Fault with the slope to the north had caused the collapse of its southern part. As a result, parallel to the northern part fractures with normal displacement, the southern plains have been created, resulting in a gardenlike collapse of the east-west continuation. The current formation on which Tabriz is located is the result of such a collapse. As a result of this collapse, the rest of the Miocene and pyroclastic sediment of the east and the south of the city are observable in lower height balances. Furthermore, the erosive function due to the entrance of the big rivers caused the deposit of alluvial material with high thickness in the plain. Regarding the headwaters of the river from the south and the east of the Tabriz plain and its elongation in an east-west state by moving towards the west, particle reduction is expected. According to the fault system activity and the occurred earthquakes in the region and observation of fractures in younger sediments, the area is tectonically active. The Alpine-Himalayan belt is one of the world's most important seismic belts, in which Iran is located. Azerbaijan is also located in this belt and had experienced destructive earthquakes in the past. There are many large and small faults in the region that may cause destructive tremors (Ghobadi *et al*., 2016).

An assessment of the peak ground acceleration (PGA) of study area should be performed to analyze the bore holes and identify the liquefaction potential of soil layers. The length of Tabriz North fault from Bostan abad to Sofian cities is at least 90 km but it seems to continue towards the south-east and the north-west. Therefore, 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 (Fig. 3).

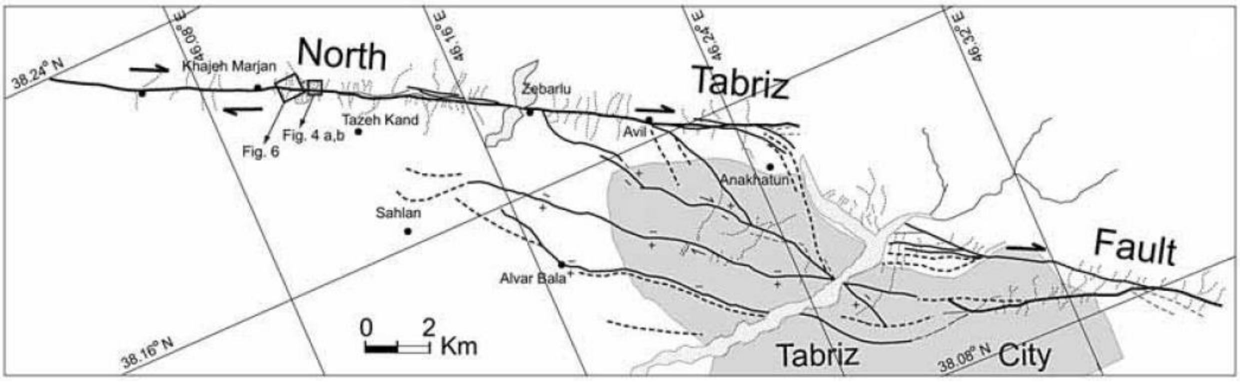

Figure 3) Map of active faults in North West of Iran (Hessami et al., 2003)

3- Liquefaction Potential Analysis

In this study, for evaluating liquefaction potential in saturate soil layers, 54 boreholes along Tabriz Metro Line 2 were collected. The Line 2 of the Tabriz city Metro, having an approximate length of 22 km, starts from the vicinity of the railway in the western part of the city and passes through Qaramalek and Qara-aqaj to the Bazar area in the central parts. The line passes the Daneshsara square, goes under the Mehranroud River, proceeding to the Abbasi Street and Shahid Fahmide Square. It continues from the Shahaid Fahmide Square towards the Baghmishe town and by changing its path, goes to the south east and finishing finally in front of the international Exhibition in Tabriz. This route is on even ground from the beginning to Baghmishe, but encounters ups and downs as it proceeds towards a hilly topography in the east. In the eastern part, the difference between the highest and lowest points along the route is about 140 metres. The position of the route is shown in Figure 4 (a, b).

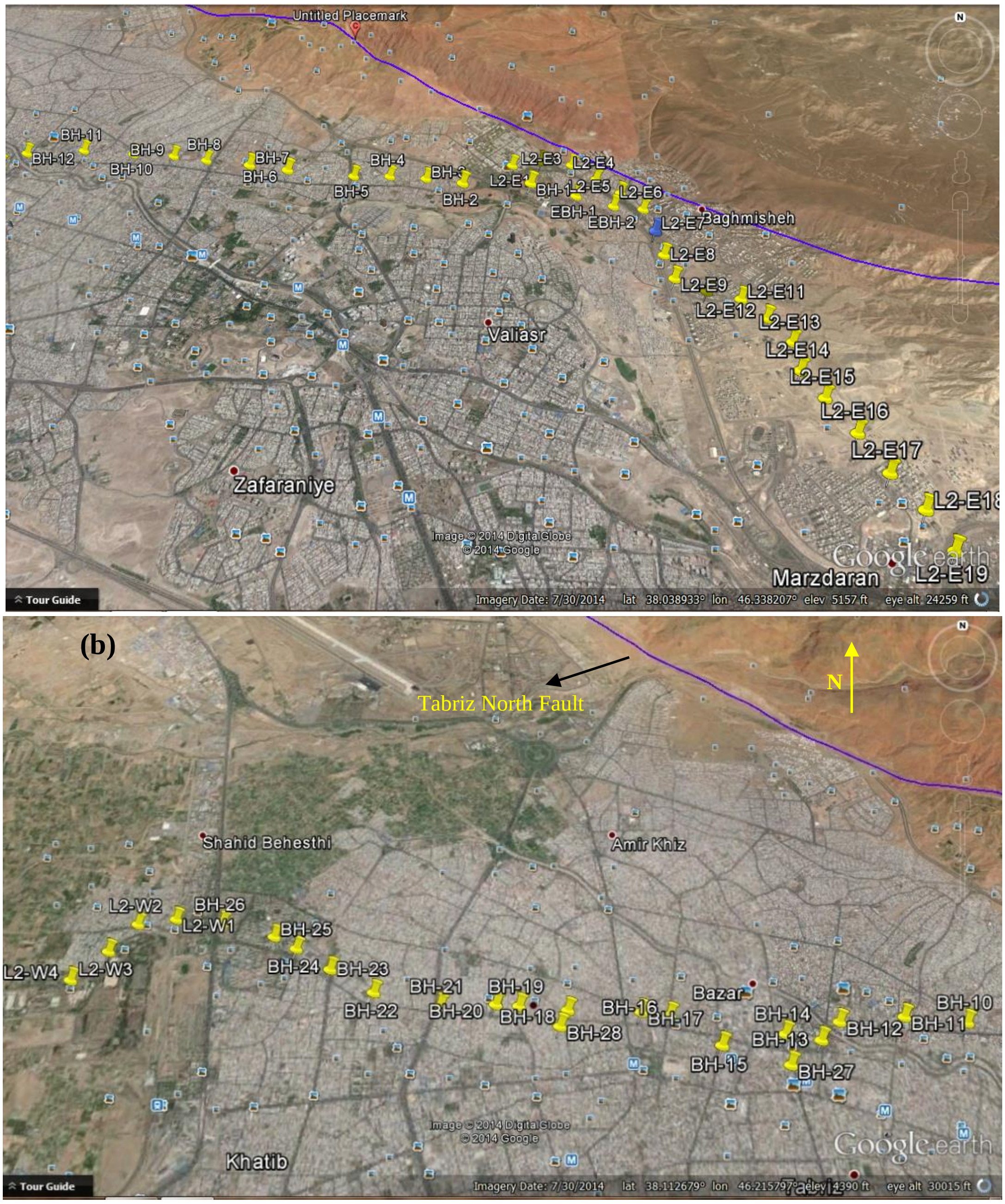

Figure 4a and b): Boreholes positions along Tabriz Metro Line 2.

The level of the ground water can be deemed as one of the main factors in assessing liquefaction potential of soil. Along the route of Line 2 of Tabriz Metro, the level of ground water changes. In one of the drilled boreholes, the water in the Artesian condition had overflew the surface of the borehole, while in other boreholes, waters, was not found above a considerable depth. The results indicated that ground water level changes were not drastic after being static, and the higher level of the ground water could be ascribed to spring season (Rahvar, 2008). Overall, the depth of the ground water was found to vary from 2 to 30 metres. The balance of the ground water decreased from east to west, showing that the water flow was from east to west corresponding to the slope of Tabriz plain. Ground water depth variations in the city of Tabriz are presented in Figure 5.

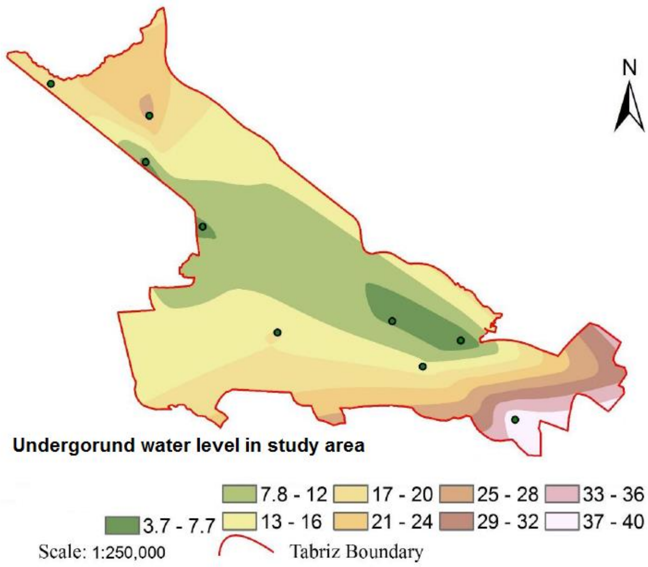

Figure 5: Variation of ground water depth in Tabriz city (Amiranlou et al., 2017)

Additionally, the level of ground water in boreholes along Tabriz Metro Line 2 is illustrated in Figure 6.

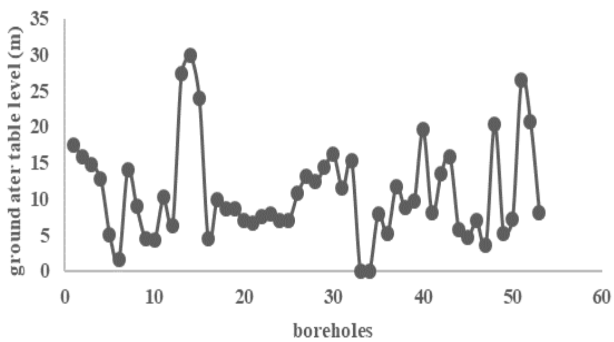

Figure 6) Variation of ground water depth in boreholes along Tabriz Metro Line 2.

3.1- 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 (2010) is carried out. In this method, first, the value of cyclic stress ratio (CSR) is estimated expressing the rate of the severity of the earquake load in a Mw=7.5. That is evaluated using the equation bellow:

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

Where 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 7 is estimated and MSF (Magnitude Scale Factor) is earthquake magnitude scale factor that is calculated based on Andrus and Stoke researches in 1997 using equation 2. Mw is earthquake magnitude:

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

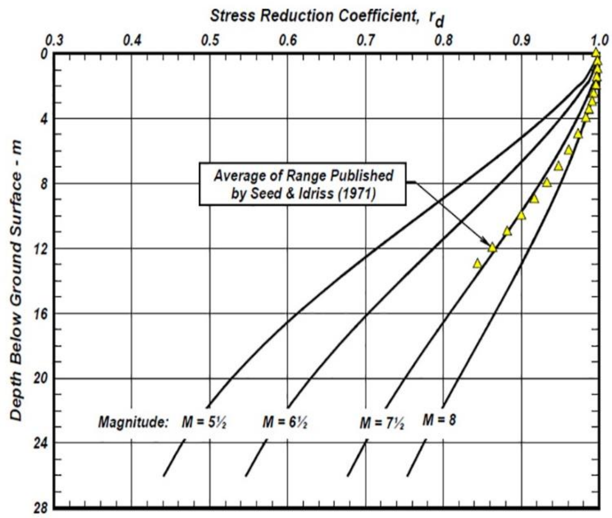

Figure 7) Variations of stress reduction coefficient with depth and earthquake magnitudes (Idriss, 1999).

Second, in order to determine to cyclic resistance ratio (CRR) of the soils simplified and modified method proposed by Seed *et al*. (1983) are used. In this step, the results obtained from the standard penetration test are modified based on the following equation proposed by Skempton (1986). Value of parameters can be observed in Table 2.

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

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

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

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

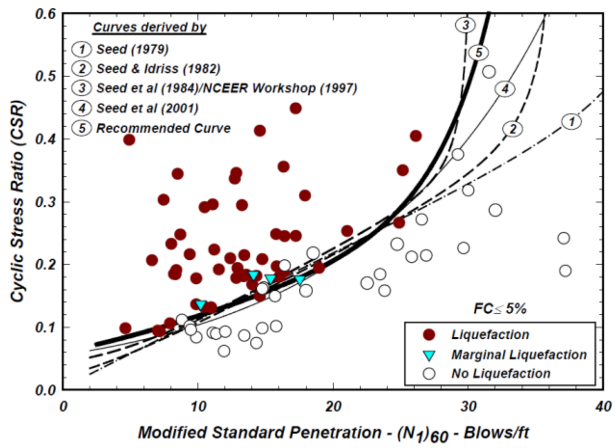

Figure 8) Liquefaction resistance curve for the earthquakes of 7.5 magnitudes (Idriss and Boulanger, 2010).

Where, 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 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):

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

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

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

Where, FC is equal fines content in soil layer.

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

3.2- Liquefaction Analysis based on Energy method

In contrast with other methods that stress or strain was used for determining cyclic resistance ratio (CRR) or cyclic stress ratio (CSR), in energy procedure a logical process for evaluating liquefaction potential of soils layers exist according with two reasons: firstly, seismologists proposed relationships for estimating released energy of earthquake. Secondly, existence of suitable correlations between dissipated energy and pore pressure (Law *et al.*, 1990).Therefore, energy method for evaluating liquefaction in soil layers have advantages are described as follows:

1- Energy is scalar quantity. Therefore, there is no necessary for determining history of stress and strain due to earthquake in soil layers.

2- Energy method is including stress, strain and geotechnical properties of soil layers.

Davis and Brill (1982) for determining imported energy (demand) or trigger factor at the same time with estimating energy of earthquake based on Gutenberg-Richter relationship three assumptions have been applied. Firstly, energy value is proportion with ($1/r^2$); (r) factor is distance between site and epicenter of earthquake. This model of attenuation in energy dissipation is not including damping materials and just geometric form of wave front is considered. Secondly, there is a linear relationship between exceed pore water pressure and dissipated energy. Thirdly, dissipated energy due to material is proportion with ($1/(\sigma'_{v0})^{0.5}$). Triggering factor is calculated with using relationship mentioned below:

$$Demand = \left[\frac{r^2 \cdot \sigma'^{\,1.5}_{vo}}{10^{1.5Mw}}\right]^{-1} \quad (9)$$

Where, r is distance between site and epicenter (m), Mw is magnitude of earthquake, σ'_{vo} is effective vertical stress in study area (kPa). According to Figure 7, demand can be determined by corrected standard penetration test results (Fig. 9).

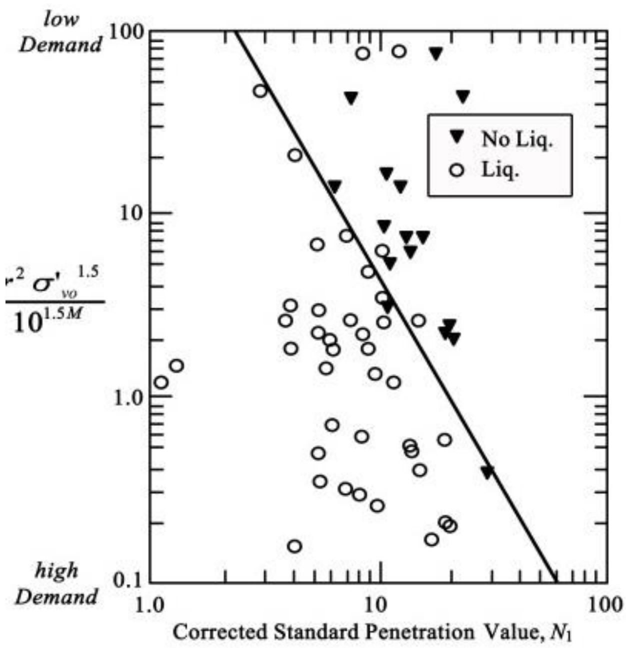

Figure 9) Correlation between demand and (N1)60, (Davis and Brill, 1982).

According to Davis and Brill (1982) procedure, capacity value of soil layers can be evaluated based on equation No.10:

$$Capacity = \left[\frac{450}{N_1^2}\right]^{-1} \quad (10)$$

In liquefaction potential evaluation based on energy method, critical state when will be happened that energy due to earthquake reached to site is more than threshold value that is representative of resistance of soil layers. In this research, Google earth satellite images are used for measuring of distance between study area (Tabriz Metro line 2) and epicenter. It should be noted that for this research epicenter was assumed in center of Tabriz north fault. Distance of boreholes along Tabriz Metro line 2 from assumed epicenter on fault (Central part of Tabriz north fault) calculated. In continue, demand and capacity in soil layers was determined with using equations No. 9 and 10.

3.3- 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:

$$CRR_j = K_\sigma \times CRR \quad (11)$$

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

$$K_\sigma = \left(\frac{\sigma'_V}{100}\right)^{f-1} \quad (12)$$

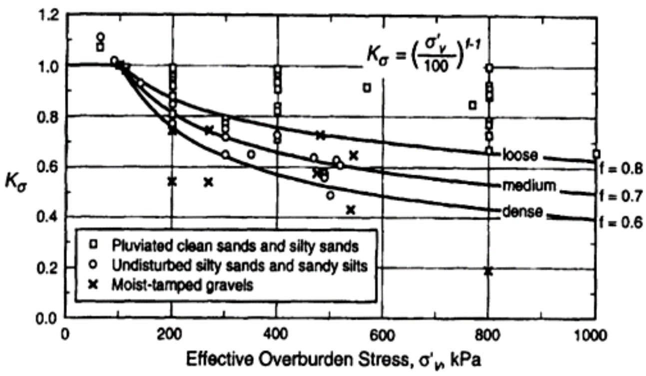

Figure 10) Variations of Kσ values versus effective overburden stress (Hynes and Olsen, 1998).

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 (Fig. 10).

3.4- Factor of safety (Fs)

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

$$F_S = \frac{CRR_j}{CSR} \quad (13)$$

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

3.5- 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*. (1978 and 1982) presented in the following equation:

$$LPI = \int_0^{20} W(Z) \times F(Z).dz \quad (14)$$

$$F(Z) = 1 - F_S \text{ For Fs < 1} \quad (14a)$$

$$F(Z) = 0 \text{ For Fs} \geq 1 \quad (14b)$$

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

$$W(Z) = 0 \text{ For Z > 20 m} \quad (14d)$$

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

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.

4- Results

The results of this study can be expressed as follow:

1- In study area, 54 boreholes collected and according to unified classification soil types are 33 gravel, 175 sand, 210 silt and 104 clay. SPT values in study area are between 4 and 70. Distribution of SPT values can be seen in Figure 11. Variation of safety factor against liquefaction in soil layers based on two method have been presented in Figure 12. With considering results can be found that in SPT method almost 30 to 40 % of soil layers have safety factor less than 1. In contrast, according to energy procedure about 80% of soil layers have liquefaction potential.

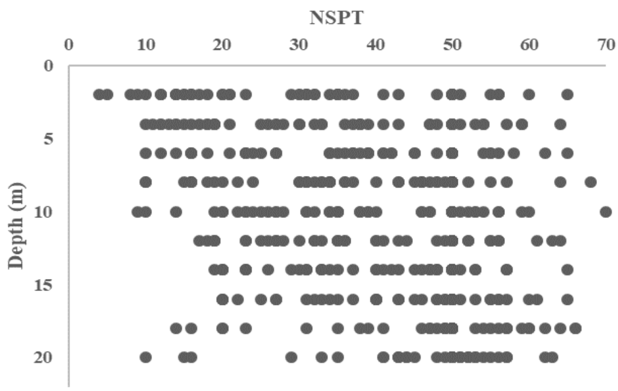

Figure 11) Variation of SPT values in study area.

2- Number of soil layers according to both method have liquefaction potential with considering soil type proposed in Table 4. As seen, generally in energy method more soil layers have liquefaction potential than SPT procedure. Also, in both method sandy layers have the most liquefaction hazards.

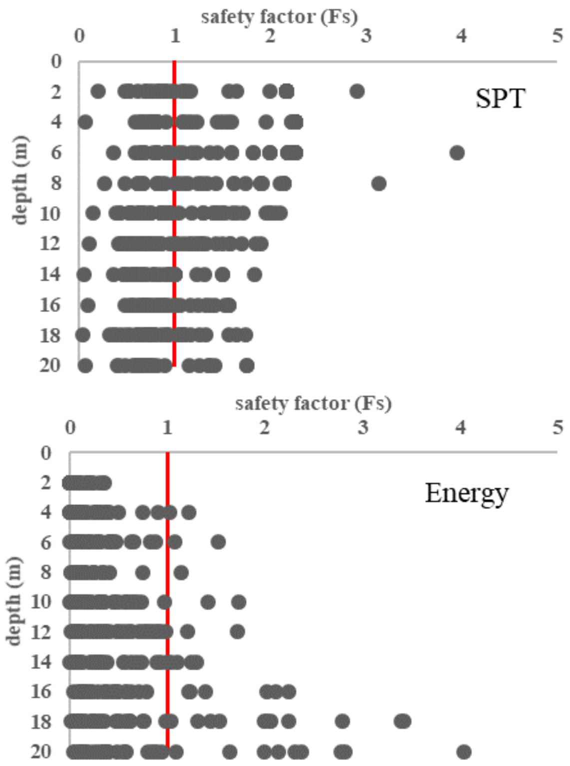

Figure 12) Variation of safety factor of soil layers against liquefaction based on two method.

3- In order that better comparison, rate of conformity and no matching between results of both method in the same soil layer with considering value of safety factor evaluated and presented in Table 5. As seen, generally both method 48% are match in express liquefaction hazard or no liquefaction. In contrast, both procedure have 52% no similarity to explain liquefaction potential of soil layers or no.

4- Liquefaction potential index (LPI) in study area according to results of both method are proposed in Figures 13 and 14. As seen in Figure 13, distribution of LPI values in boreholes is not balanced, because estimated liquefaction potential index based on energy method is more than SPT procedure. Also, Figure 14 shows liquefaction hazard according to energy method have high value and so conservative.

Table 4) Number of layers have liquefaction hazards based on both method with considering soil types.

Number of layers					**Soil type**
No liquefaction in Energy method	**No liquefaction in SPT method**	**liquefaction in Energy method**	**liquefaction in SPT method**	**Total**	
4	20	29	13	33	gravel
35	82	140	93	175	sand
80	131	130	79	210	silt

Table 5. Conformity of results of both method in the same soil layer.

Descriptions					Soil type
Rate of un conformity	Rate of conformity	No Liquefaction in both method	Liquefaction in both method	Number of layer	
%55	%45	2	13	33	gravel
%45	%55	9	88	175	sand
%57	%43	16	75	210	silt
%52	%48	27	176	418	total

5- According to Iwasaki *et al*. criterion, number of boreholes with considering LPI values based on SPT and energy method presented in Table 6. As seen, in SPT method almost 70% of boreholes have high and very high hazards in liquefaction potential in soil layer. In contrast, in energy method about 89% boreholes have very high hazard for liquefaction.

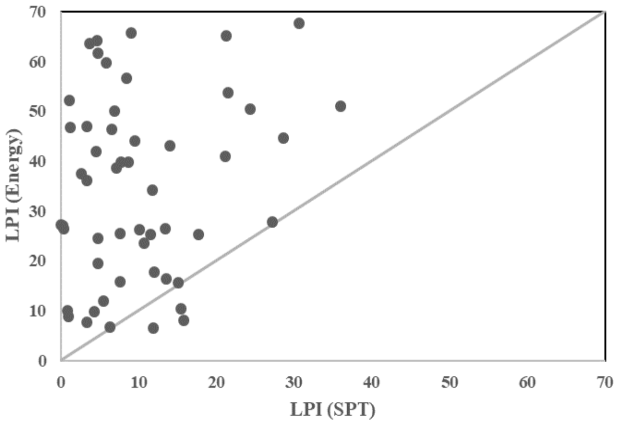

Figure 13) Distribution of LPI according to energy and SPT method.

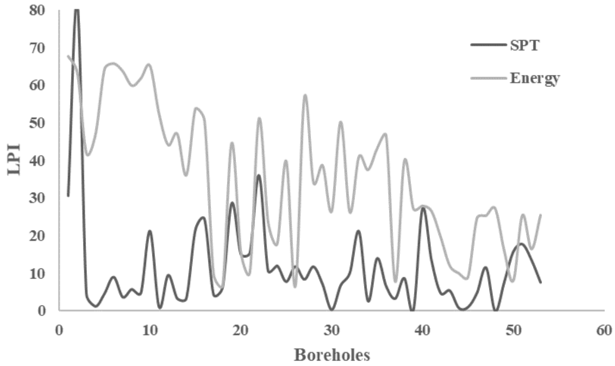

Figure 14) Comparison of LPI values in boreholes according to energy and SPT method.

According to results mentioned in Table 6, comparative between two methods along Tabriz Metro line 2 in boreholes position have been performed. As seen in Figure 15, in east part of study area through vicinity to fault and in west part because of high water level based on SPT results analysis liquefaction potential hazard have high value. Also, as seen in Figure 16 in energy method because of mentioned items in previous part and vicinity to fault, almost in more of boreholes liquefaction potential hazard have been observed.

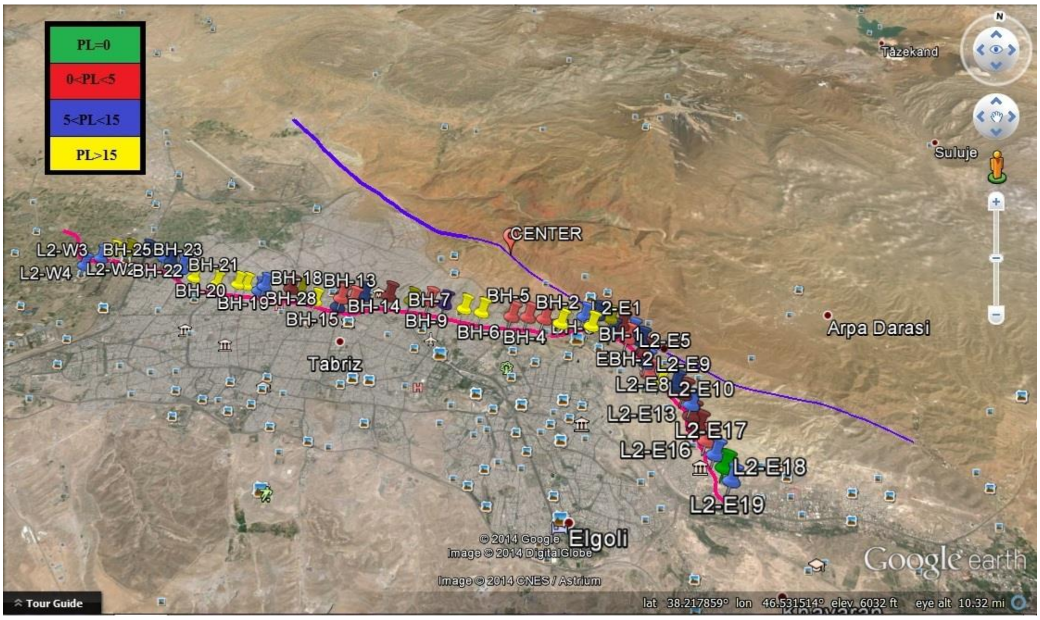

Figure 15) Liquefaction hazard along Tabriz Metro line 2 based on SPT method.

Table 6) Classification of boreholes according to LPI values for both of method.

Number of boreholes according to LPI values (SPT method)				
LPI>15	5<LPI<15	0<LPI<5	LPI=0	Liquefaction Potential Index (LPI)
14	22	18	٠	Number
25	41	33	٠	percent
Number of boreholes according to LPI values (energy method)				
LPI>15	5<LPI<15	0<LPI<5	LPI=0	Liquefaction Potential Index (LPI)
48	6	٠	٠	Number
89	11	٠	٠	percent

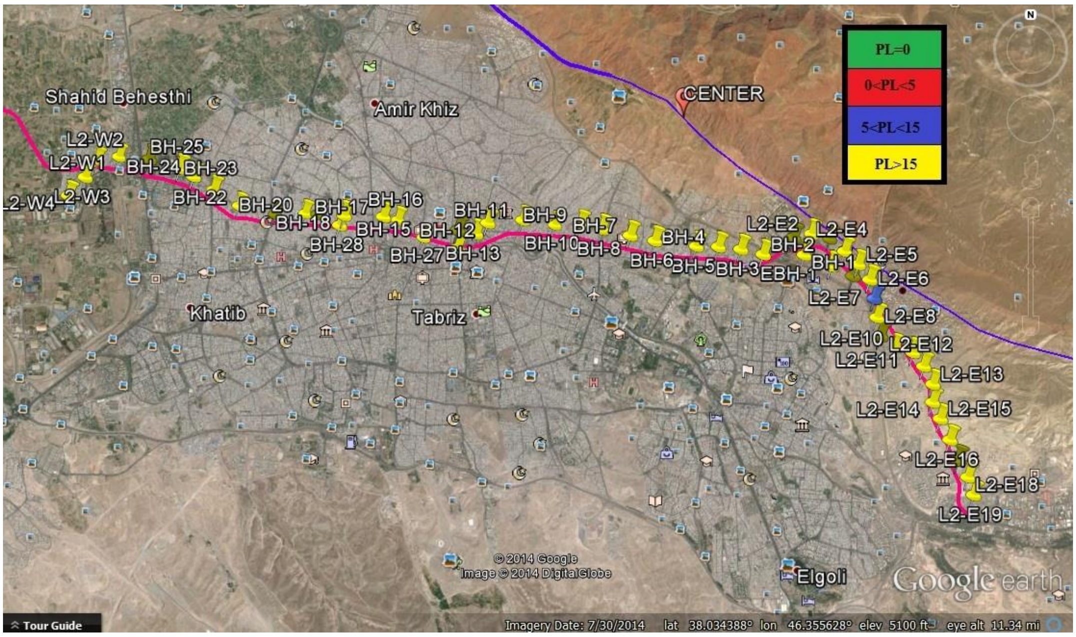

Figure 16) Liquefaction hazard along Tabriz Metro line 2 based on energy method.

5- Conlucions

In this research, two liquefaction potential evaluation methods were studied. Idriss and Boulanger (2010) process based on SPT results with energy method according to Davis and Brill (1982) procedure compared to each other. Then, liquefaction potential index (LPI) according to Iwasaki *et al*. (1982, 1987) procedure determined. Study area is Tabriz Metro Line 2 was selected. Results of this study is described as follows:

1- There is no good matching between results of two methods. Because, fundamentals theory of both method is difference. Energy procedure is based on release of earthquake energy and distance between epicenter of earthquake and site. Meanwhile, in SPT method liquefaction potential of soil layers evaluate as a point. Also, cyclic stress ratio (CSR) determine with using so much approximate parameters .

2- It is obvious that for correcting SPT blow count some coefficients are used. With considering drilling machine and energy efficiency and accuracy of test performance can create uncertainties on results. One of effective factor is energy correction coefficient. Therefore, it is proposed that factors related to energy coefficient determined exactly at least for two boreholes drilled in study area with using sensors.

3- Predict of liquefaction hazards in soil layers of study area (i.e. Tabriz Metro Line 2) showed that according to energy method this potential is more than SPT procedure. Because of proximity of Tabriz Metro Line 2 to Tabriz North fault as an important factor in energy method and effective in actuator for happening liquefaction in soil layers. Outcome of this research is similar to researches performed by Oshnaviye and Dabiri in 2017, Ghasemian *et al*. in 2016 and Mohammadi *et al*. in 2015. These studies observed that in some parts of Tabriz Metro Line 2 are exist. But, energy method proposed new predictions of liquefaction hazards.

With considering of results, it can be explained that Tabriz city and specially Tabriz Metro Line 2 have liquefaction potential because of type of soil layers, ground water table depth and peak ground acceleration due to probable earthquake. Therefore, it is necessary using new method and procedures for prediction liquefaction hazards such as neural network analysis, algorithm genetics and reliability analysis for propose improvement process and stabilization of soil layers against damages due to liquefaction hazards.

References

Alavi, A. H., Gandomi, A. H. 2014. Energy-based numerical models for assessment of soil Liquefaction. Geoscience Frontiers: 4, 1–15.

Amiranlou, H., Pourkermani, M., Dabiri, R., Qoreshi, M., Buzari, S. 2017. Computing maximum seismic acceleration of North Tabriz Fault using earthquake simulation based on finite fault source. Geoscience: 27, 193–199.

Andrus, R. D., Stokoe, K. H. 1997. Liquefaction Resistance Based on Shear Wave Velocity, NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Technical Report NCEER-97-0022, T.L. Youd and I. M. Idriss, Eds, Held 4-5 January 1996, Salt lake City, UT, NCEER, Buffalo, NY,88-128.

Babazadeh, R., Ghobadi, M. H., Khanlari, Gh. R., Asghari, E. 2010. Zonation of Liquefaction Susceptibility in Soils along Tabriz Metro Line 2 with using GIS. Journal of Iranian Engineering Geology Society: 3, 15–30.

Barzegari, Gh., Uromiehy, A., Zhao, J. 2014. EPB tunneling challenges in bouldery ground: new experience on Tabriz Metro Line 1, Iran. Bulletin of Engineering Geology and the Environment: 73, 429-440

Baziar, M. H., Jafarian, Y. 2007. Assessment of liquefaction triggering using strain energy concept and ANN model: Capacity Energy. Soil Dynamics and Earthquake Engineering: 27, 1056–1072.

Berrill, J. B., Davis R. O. 1985. Energy dissipation and seismic liquefaction of sands: revised model. Soils and Foundation: 25, 106–18.

Chen, Y. R., Hsieha, S. C., Chen, J. W., Shih, C. C. 2005. Energy-based probabilistic evaluation of soil liquefaction, Soil Dynamics and Earthquake Engineering: 25, 55–68.

Davis, R. O., Berrill, J. B. 1982. Energy dissipation and seismic liquefaction in sands. Earthquake Engineering and Structural Dynamics: 10, 59–68.

Final Report of Preliminary Geotechnical Investigation, Tabriz Urban Railway Line 2. (2008), P. O. Rahvar Consulting Engineering’s.

Ghasemian, M., Dabiri, R., Mahari, R. 2017. Settlements Hazard of Soil Due to Liquefaction along Tabriz Metro Line 2. Research Bulletin of Seismology and Earthquake Engineering (In Persian- In press).

Ghobadi, M. H., Firuzi, M., Asghari, E. 2016. Relationships between Geological Formations and Ground Water Chemistry and Their effects on Concrete Lining of Tunnels (case study: Tabriz Metro Line 2). Environmental Earth Science: 75, 2–14.

Green, R. A. 2001. Energy-based evaluation and remediation of liquefiable soils. Ph.D. dissertation, Virginia Polytechnic Institute and State Univ., Blacksburg, VA.

Hessami, Kh., Pantosti, D., Tabassi, H., Shabanian, E., Abbassi, M. R., Feghhi, Kh., Solaymani, Sh. 2003. Paleoearthquakes and slip rates of the North Tabriz Fault, NW Iran: preliminary results. Annals of Geophysics: 46, 903–917.

Hynes, M. E., Olsen, R. S. 1998. Influence of Confining Stress on Liquefaction Resistance, Proceeding, International Workshop on the Physics and Mechanics of Soil Liquefaction, held 10-11 September 1998, Baltimore, M.D., A.A. Balkema, Rotterdam, Netherlands.

Idriss, I. M. 1999. An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential. In: Proceedings, TRB workshop on new approaches to liquefaction, publication no. FHWARD-Federal Highway Administration, 99–165.

Idriss, I. M., Boulanger, R. W. 2010. SPT-based liquefaction triggering procedures. Report no. UCD/CGM-10/02, Center for Geotechnical Modeling, University of California, Davis.

Iran design building against earthquake code 2800- Road. 2014. housing and urban development research center, and version 4.

Iwasaki, T., Tokida, K., Tatsuko, F. Yasuda, S. 1978. A practical method for assessing soil liquefaction potential based on case studies at various sites in Japan, Proceedings of 2nd International Conference on Microzonation, San Francisco, 885–896.

Iwasaki, T., Tokida, K., Tatsuoka, F., Watanabe, S., Yasuda, S., Sato, H. 1982. Microzonation for soil liquefaction potential using simplified methods, Proceedings of 2nd International Conference on Microzonation, Seattle, 1319–1330.

Jafarian, Y., Sadeghi Abdollahi, A., Vakili, R., Baziar, M. H., Noorzad, A. 2011. On the efficiency and predictability of strain energy for the evaluation of liquefaction potential: A numerical study. Computers and Geotechnics: 38, 800–808.

Jafarian, Y., Vakili, R., Sadeghi Abdollahi, A. Baziar, M. H. 2014. Simplified Soil Liquefaction Assessment Based on Cumulative Kinetic Energy Density: Attenuation Law and Probabilistic Analysis. International Journal of Geomechanics: 14, 267–281.

Kayen, R. E., Mitchell, J. 1997. Assessment of liquefaction potential during earthquakes by Arias intensity, Journal of Geotechnical and Geoenvironmental Engineering: 12, 1162–1174.

Kokusho, T., Mimori, Y. 2015. Liquefaction potential evaluations by energy-based method and stress-based method for various ground motions. Soil Dynamics and Earthquake Engineering: 7, 30–146.

Law, K. T., Cao. Y. L., He, G. N. 1990. An energy approach for assessing seismic liquefaction potential. Canadian Geotechnical Journal: 27, 320–9.

Mohammadi, S. D., Firuzi, M., Asghari Kaljahi E. 2015. Geological and geotechnical risk in the use of EPB-TBM, case study: Tabriz Metro, Iran. Bulletin of Engineering Geology: 75, 1571–1583.

Oshnaviyeh, D, Dabiri, R. 2017. Comparison of Standard Penetration Test (SPT) and Shear Wave Velocity (Vs) method in determining risk of liquefaction potential along Tabriz Metro Line 2. Journal of Engineering Geology, Kharazmi University (in Persian, in press).

Running, D. L. 1996. An energy-based model for soil liquefaction. Ph.D. Thesis, Washington State Univ., Pullman, WA.

Seed H. B., Idriss I. M., Arango I. 1983. Evaluation of Liquefaction Potential Using Field Performance Data, Journal of Geotechnical Engineering: 109, 458-482.

Skempton, A. K. 1986. Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Aging and over consolidation. Journal of Geotechnique: 36, 425-447.

Trifunac, M. D., Todorovska M. I. 2004. Maximum distance and minimum energy to initiate liquefaction in water saturated sands. Soil Dynamics and Earthquake Engineering: 24, 89–101.