The Evaluation of Soil Liquefaction Potential Using Shear Wave Velocity Based on Empirical Relationships

Mehrad Khalil Noutash m.noutash@yahoo.com
Msc of Soil and Foundation Engineering
Zanjan Bransh, Islamic Azad University, Zanjan, Iran

Rouzbeh Dabiri rouzbehdabiri@yahoo.com
Assistant Prof., Dep. of Civil Engineering, Tabriz Branch
Islamic Azad University, Tabriz, Iran (Corresponding Author)

Masoud Hajialilue Bonab mhbonab@gmail.com
Associate Prof., Dep. of Civil Engineering, Tabriz University
Tabriz, Iran

Abstract

The liquefaction resistance of soils can be evaluated using laboratory tests such as cyclic simple shear, cyclic triaxial, cyclic torsional shear, and field methods such as Standard Penetration Test (SPT), Cone Penetration Test (CPT), and Shear Wave Velocity (Vs). The present study is aimed at comparing the results of two field methods used to evaluate liquefaction resistance of soil, i.e. SPT based on simplified procedure proposed by Seed and Idriss (1985) and shear wave velocity (Vs) on the basis of Andrus et al. (2004) process using empirical relationships between them. Iwasaki's (1982) method is used to measure the liquefaction potential index for both of them. The study area is a part of south and southeast of Tehran. It is observed that there is not a perfect agreement between the results of two methods based on five empirical relationships assuming cemented and non-cemented conditions for (OF) soil. Liquefaction potential index (PL) value in SPT test was found to be more than Vs method.

Keywords: Liquefaction, Liquefaction Potential Index (PL), Shear Wave Velocity (Vs), South of Tehran, Standard Penetration Test (SPT).

1. INTRODUCTION

The simplified procedure is widely used to predict liquefaction resistance of soils world. It was originally developed by Seed and Idriss [1] using Standard Penetration Test (SPT) blow counts correlated with a parameter representing the seismic loading on the soil, called cyclic stress ratio (CSR). This procedure has undergone several revisions and updated [2, 3, 4]. In addition, procedures have been developed based on the Cone Penetration Test (CPT), Becker Penetration Test (BPT) and small-strain Shear Wave Velocity (Vs) measurements. The use of Vs to determine the liquefaction resistance is suitable, because both Vs and liquefaction resistance are influenced by such factors as; confining stress, soil type/plasticity and relative density [5, 6, 7] and in situ Vs can be measured by several seismic tests including cross hole, down hole, seismic cone penetrometer (SCPT), suspension logger and spectral analysis of surface waves (SASW). During the past two decades, several procedures have been proposed to estimate liquefaction resistance based on Vs. These procedures were developed from laboratory studies [8, 9, 10, 11, 12, 13, 14, 15], analytical studies [16, 17], penetration- Vs equations [18, 19], in situ Vs measurements at earthquake shaken site [20, 21, 22]. Some of these procedures follow the general format of Seed- Idriss simplified procedure which the Vs is corrected to a reference vertical stress and correlated with the cyclic stress ratio. This paper presents the results of the comparison between two Vs and SPT methods of soil liquefaction potential evaluation in the south of Tehran. Furthermore, liquefaction potential index (PL) is calculated by Iwasaki et al. [23] procedure for both aforementioned methods.

2. GENERAL CONDITION AND SOIL STRATIFICATION

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.

FIGURE 1: The study area and PGA distribution throughout Tehran for an earthquake corresponding to 475 year return period [24].

3. ANALYSIS OF BOREHOLES TO EVALUATE THE LIQUEFACTION POTENTIAL

The peak ground acceleration (PGA) is necessary for the analysis of boreholes to evaluate liquefaction potential of soils. According to Figure 1, PGA values were selected in each boreholes position. In addition, the depth of ground water table in the assessment of liquefaction potential of soils was considered. To define critical ground water level in boreholes, the maps of variations of underground water depth in Tehran Plain were used. In Shear wave velocity (Vs) measurement method based on Andrus et al. [25] process for assessing liquefaction potential, Vs amounts were calculated using empirical equations between shear wave velocity and SPT blow count (N) for all soil types as follow [26]:

$$V_s = 61.N^{0.5} \quad (1)$$
$$V_s = 97.N^{0.314} \quad (2)$$
$$V_s = 76.N^{0.33} \quad (3)$$
$$V_s = 121.N^{0.27} \quad (4)$$
$$V_s = 22.N^{0.85} \quad (5)$$

4. ASSESSMENT OF LIQUEFACTION POTENTIAL

The evaluation procedures based on Standard Penetration Test (SPT) (Seed and Idriss, 1985, simplified method) and measurement of shear wave velocity (Vs) (Andrus and Stokoe, 2004) require the measurement of three parameters: (1) the level of cyclic loading on the soil caused by the earthquake, expressed as a cyclic stress ratio (CSR); (2) the stiffness of the soil, expressed as a overburden stress corrected SPT blow count and shear wave velocity; and (3) the resistance of the soil to liquefaction, expressed as a cyclic resistance ratio (CRR). Guidelines for calculating each parameter are presented below:

4.1 Cyclic stress ratio (CSR)

The cyclic stress ratio at a particular depth in at soil deposit level can be measured by Eq.(6) in both methods [1]:

$$CSR = \frac{\tau_{av}}{\sigma'_v} = 0.65\left[\frac{a_{max}}{g}\right]\left[\frac{\sigma_v}{\sigma'_v}\right]\times r_d \quad (6)$$

Where a_{max}, is the peak horizontal ground surface acceleration (based on Figure 1), g is the acceleration of gravity, σ_V is the total vertical (overburden) stress at the desired depth, σ'_V is the effective overburden stress at the same depth, and r_d is the shear stress reduction coefficient (Figure 2).

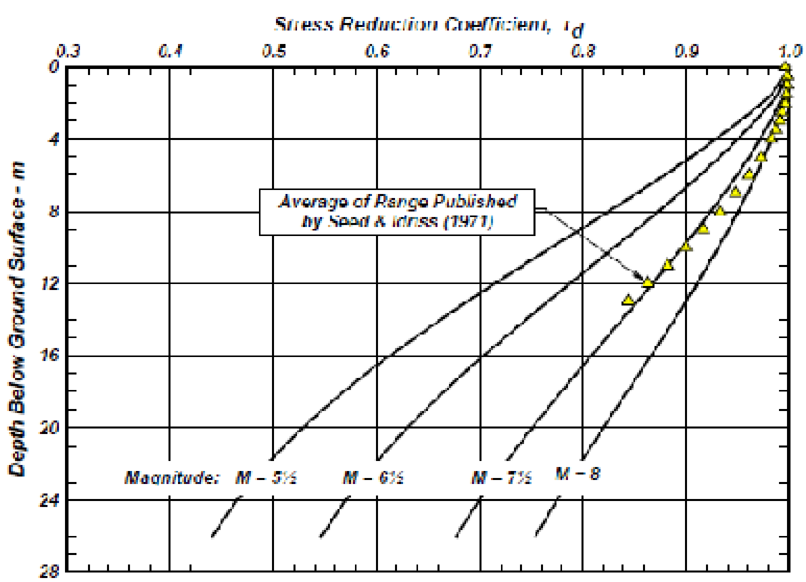


FIGURE 2: Variations of stress reduction coefficient with depth and earthquake magnitudes [27, 28]

4.2 Corrected SPT Blowcount and Shear Wave Velocity

In addition to the fines content and the grain characteristics, other factors affect SPT results, as noted in Table 1. Eq. (7) incorporates these factors:

$$(N_1)_{60} = N_{SPT}.C_N.C_E.C_B.C_R.C_S \tag{7}$$

Where $(N_1)_{60}$ corrected standard penetration test blow count, N_{SPT} represents the measured standard penetration resistance, C_N is a factor to normalize, N_{SPT} represents the effective overburden stress, C_E, represents the correction for hammer energy ratio (ER), C_B is the correction factor for borehole diameter, C_R is the correction factor for rod length, and C_S is the correction factor for samplers with or without liners.

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	C Pa=100kPa
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 $\rangle 30$m	C_R	0.75 0.85 0.95 1.0 $\langle 1.0$
Sampling method	Standard sampler Sampler without liners	C_S	1.0 1.1 to 1.3

TABLE 1: Correction Factors of SPT [29]

In the procedure of liquefaction potential evaluation proposed by Andrus et al. [24], shear wave velocity should be corrected to overburden stress. Eq.(8) is suggested:

$$V_{S1} = V_S(\frac{P_a}{\sigma_V^{'}})^{0.25}.(\frac{0.5}{K_0^{'}})^{0.125} \tag{8}$$

Where V_s is the shear wave velocity (m/s), V_{s1} is the stress-corrected shear wave velocity (m/s), P_a is the atmosphere pressure equal to 100kPa, σ'_V, shows the effective overburden stress and $K_0^{'}$, is the coefficient of effective earth pressure (in this study assumed equal to 0.5).

4.3 Cyclic Resistance Ratio (CRR)

In the simplified procedure, Figure 3 is a graph of calculated CSR and corresponding $(N_1)_{60}$ data from sites where liquefaction effects were or not observed following the past earthquakes of approximately 7.5 magnitude. CRR Curves on this graph were conservatively positioned to separate the regions with data indicative of the liquefaction from the regions with data indicative of non-liquefaction. Curves were developed for granular soils with the fine contents of 5% or less, 15% and 35% as shown on the plot.

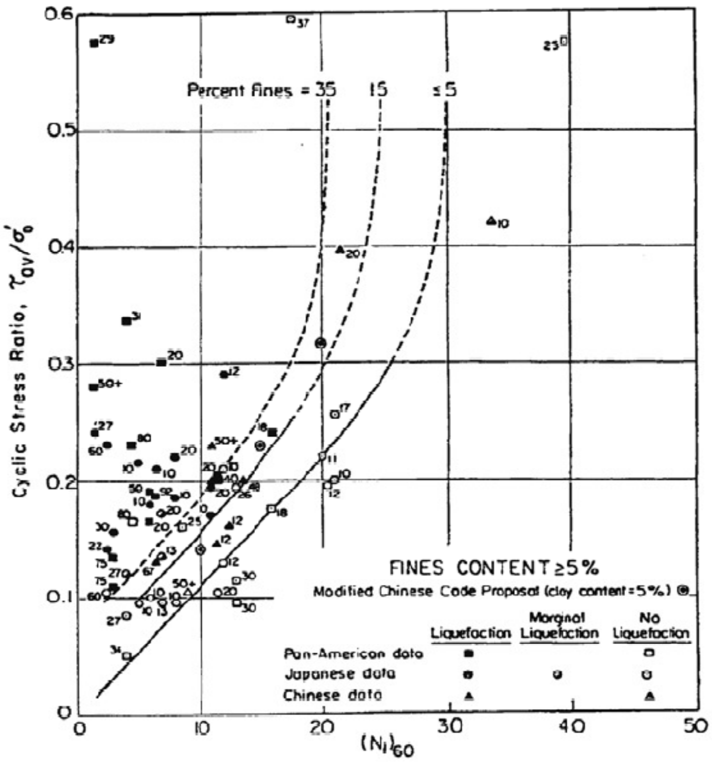


FIGURE 3: The liquefaction resistance curves by Seed et al. for the earthquakes of 7.5 magnitude [4]

Furthermore, in shear wave velocity measurement method, the cyclic resistance ratio (CRR) can be considered as the value of CSR that separates the liquefaction and non-liquefaction occurrences for a given V_{s1}. Shown in Figure 4 are the CRR-V_{s1} curves by Andrus et al. [24] for the earthquakes of 7.5 magnitudes.

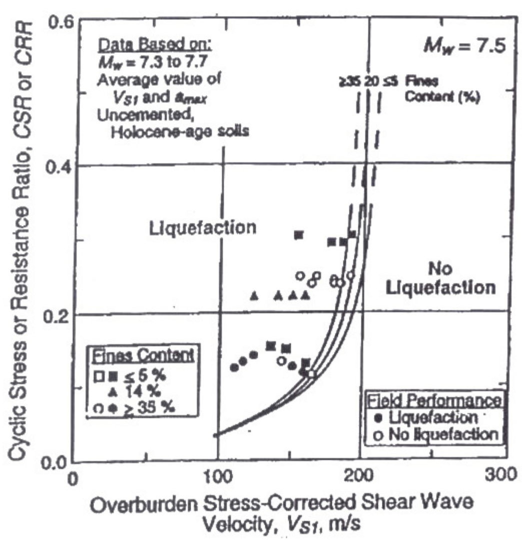


FIGURE 4: The liquefaction resistance curves by Andrus et al. [24] for the 7.5 magnitude earthquakes

The CRR-V_{s1} curves shown in Figure 4 can be defined by Eq. (9):

$$CRR = K_{a2}\{0.022(\frac{K_{a1}V_{s1}}{100})^2 + 2.8(\frac{1}{V_{s1}^* - K_{a1}V_{s1}} - \frac{1}{V_{s1}^*})\}MSF \quad (9)$$

Where MSF is the magnitude scaling factor, V_{s1}^* is the limiting up value of V_{s1} for liquefaction occurrence, K_{a1} is a factor to correct for high V_{s1} values caused by aging, and K_{a2} is a factor to correct the influence of age on CRR. Andrus and Stokoe [24] suggest the following relationships for estimating MSF and V_{s1}^*:

$$MSF = (\frac{M_w}{7.5})^{-2.56} \quad (10)$$

$$V_{s1}^* = 215 \qquad FC \le 5\% \text{ (FC=Fines content)} \quad (11a)$$

$$V_{s1}^* = 215 - 0.5(FC - 5) \qquad 5 \prec FC \prec 35\% \quad (11b)$$

$$V_{s1}^* = 200 \qquad FC \ge 35\% \quad (11c)$$

In this study, the earthquake magnitude (Mw) is assumed 7.5. Therefore, MSF is equal to 1.0. Both K_{a1} and K_{a2} factors are equal to 1.0 for uncemented soils of Holocene age. For the older and cemented soils, K_{a1} factor is evaluated using curves in figure 5. If the soil conditions are unknown and penetration data is not available, the assumed value for K_{a1} is 0.6 [24].

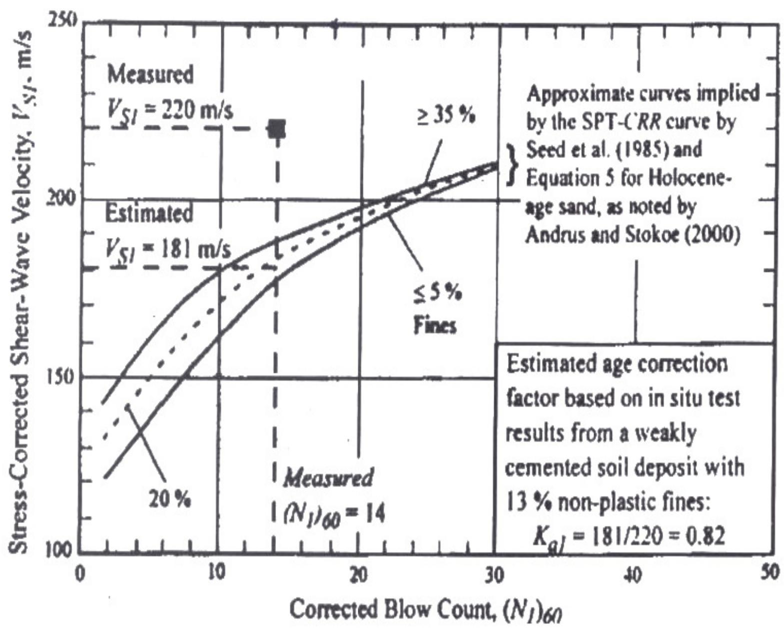


FIGURE 5: Suggested method for estimating K_{a1} from SPT and V_s measurements at the same site [24]

In both methods, if the effective overburden stress is greater than 100kPa at in question depth, CRR value is corrected using following equations and Figure 6. [30]:

$$CRR_J = CRR.K_\sigma \quad (12)$$

$$K_\sigma = (\frac{\sigma_V^{'}}{100})^{f-1} \quad (13)$$

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.

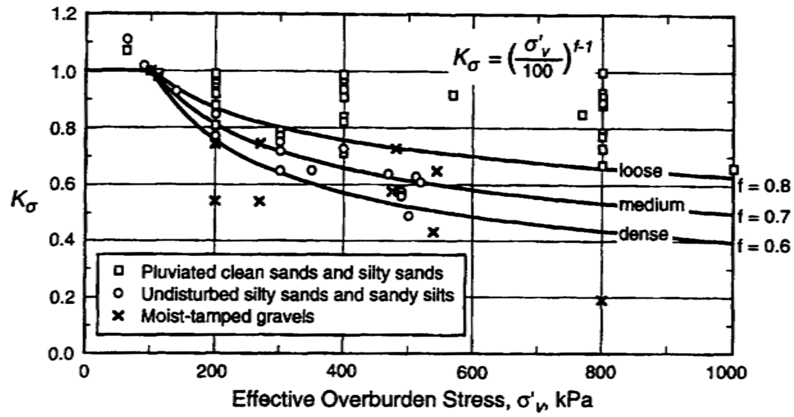


FIGURE 6: Variations of K_σ values versus effective overburden stress [30]

4.4 Safety Factor

One way to quantify the potential for liquefaction is the safety factor. Factor of safety (F_S) against liquefaction is commonly measured using the following formula:

—— 14)

Where CRR_J is corrected value of CRR estimated by Eq.(12). By convention, the liquefaction is predicted to occur when $F_S \leq 1$.When $F_S > 1$, the liquefaction is predicted not to occur.

4.5 Liquefaction Potential Index (PL)

Iwasaki et al [23] quantified the severity of possible liquefaction at any site by introducing a factor called the liquefaction potential index (P_L) defined as:

(15)

$F(Z)= 1-F_S$ (16)
$W(Z)=10-0.5Z$ (17)

Where Z is the depth in question, F (Z) is the function of the liquefaction safety factor (F_S) and W(Z) is the function of depth. The range of P_L according to Table 2 is from 0 to 100. In this study PL values were measured and then compared for both methods.

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

TABLE 2: Liquefaction potential index (P_L) and its describes [23]

5. EVALUATING THE RESULTS OF DATA ANALYSIS

The results of the data analysis based on both methods mentioned above using five empirical relationships as:

1- Liquefaction potential index (P_L) values based on SPT method is observed in Table 3. Results show that 51% of the data according to Table 2, ranking 2 have low liquefaction risk.

PL- Value	P_L=0	0<P_L≤ 5	5<P_L≤ 15	P_L> 15
Number	15	34	18	0
Percent	23	51	26	0

TABLE 3: Liquefaction potential index (P_L) values based on SPT analysis

2- P_L values based on shear wave velocity (Vs) method using five empirical relationships (Eqs.1 to 5) in two uncemented and cemented soils are seen in Tables 4 and 5. The results show that the used relations are overestimated and most of them have shown nonliquefaction condition for soils in the studied area.

PL- Value	P_L=0	0<P_L≤ 5	5<P_L≤ 15	P_L> 15
		Eq.1		
Number	63	3	1	0
Percent	94	4.5	1.5	0
		Eq.2		
Number	60	6	1	0
Percent	90	9	1	0
		Eq.3		
Number	61	6	0	0
Percent	91	9	0	0
		Eq.4		
Number	60	7	0	0
Percent	89.5	10.5	0	0
		Eq.5		
Number	61	6	0	0
Percent	91	9	0	0

TABLE 4: The liquefaction potential index (P_L) values based on Vs analysis in the cemented soils

PL- Value	P_L=0	0<P_L≤ 5	5<P_L≤ 15	P_L> 15
		Eq.1		
Number	66	1	0	0
Percent	98.5	1.5	0	0
		Eq.2		
Number	65	2	0	0
Percent	97	3	0	0
		Eq.3		
Number	66	1	0	0
Percent	98.5	1.5	0	0
		Eq.4		
Number	66	1	0	0
Percent	98.5	1.5	0	0
		Eq.5		
Number	67	0	0	0
Percent	100	0	0	0

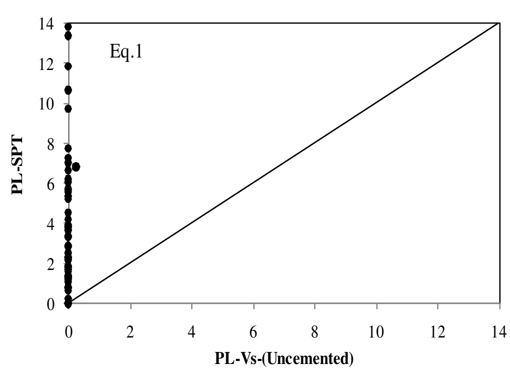
TABLE 5: Liquefaction potential index (P_L) values based on Vs analysis in the uncemented soils

3- In 67 boreholes, about 529 soil layers analyzed and liquefaction potential of soils calculated the results of which for all types of soils are presented in Table 6. **The r**esults show that there is no compatibility between two procedures in soil liquefaction expression for two states. On the contrary, both of them present suitable harmony in nonliquefaction condition for soils.

Type of Soil	SPT	Vs Cemented				
	Liquefied	**Liquefied in Eq.1**	**Liquefied in Eq.2**	**Liquefied in Eq.3**	**Liquefied in Eq.4**	**Liquefied in Eq.5**
Silt	57	2	2	1	1	5
Sand	81	2	5	3	5	3
Gravel	16	0	0	2	2	0
		Uncemented				
Silt	57	1	1	0	0	0
Sand	81	0	1	1	1	0
Gravel	16	0	0	0	0	0
	SPT	**Vs Cemented**				
Type of Soil	**Non Liquefied**	**Non Liquefied in Eq.1**	**Non Liquefied in Eq.2**	**Non Liquefied in Eq.3**	**Non Liquefied in Eq.4**	**Non Liquefied in Eq.5**
Silt	123	178	178	179	179	175
Sand	193	272	269	271	269	271
Gravel	59	75	75	73	73	75
		Uncemented				
Silt	123	179	179	180	180	180
Sand	193	274	273	273	273	274
Gravel	59	75	75	75	75	75

TABLE 6: The results of the estimating liquefaction potential in question depths using SPT and Vs methods based on five empirical relationships

4- The comparative diagrams related to the liquefaction potential index (PL) values based on SPT and shear wave velocity methods in uncemented and cemented states for soils are presented in Figures 7 and 8. As seen, the results are consistent with the values in the tables shown above and the liquefaction potential of soils that is based on shear wave velocity method is overestimated using empirical relationships.



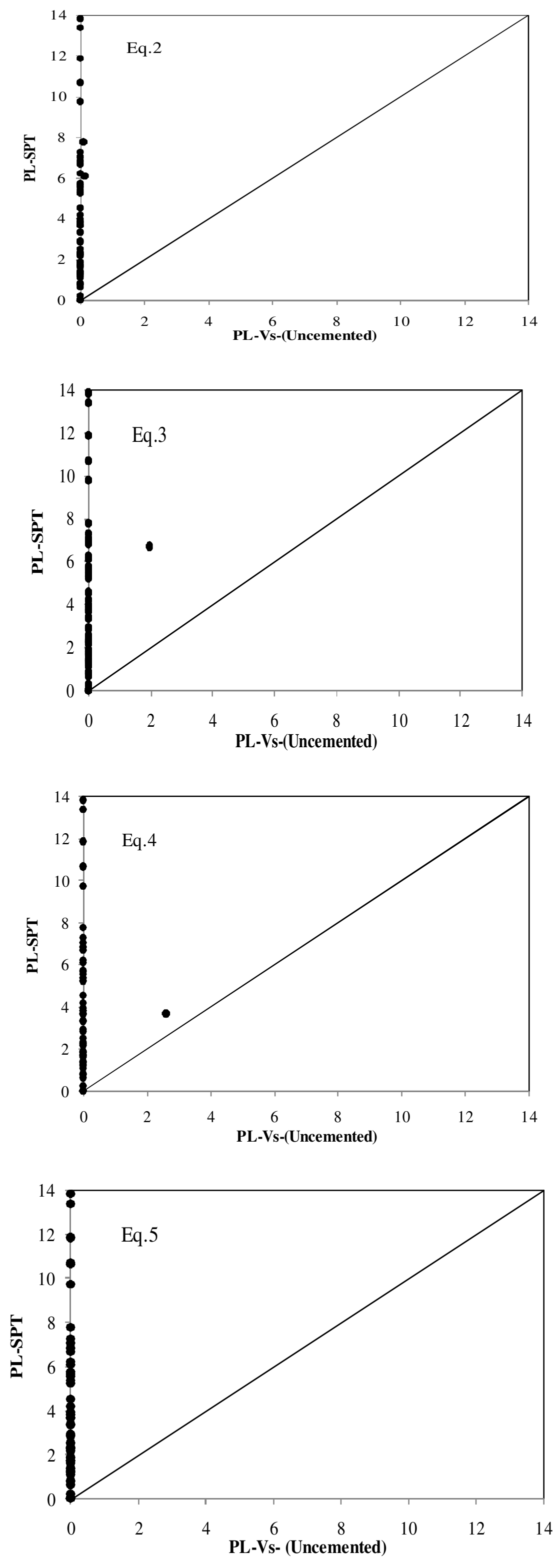
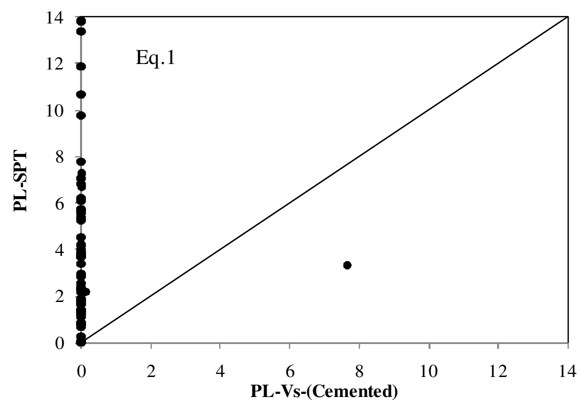
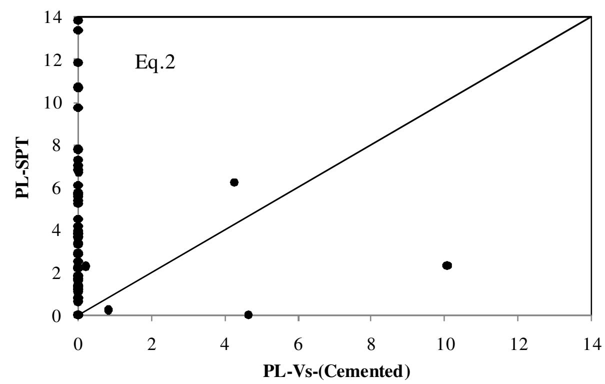
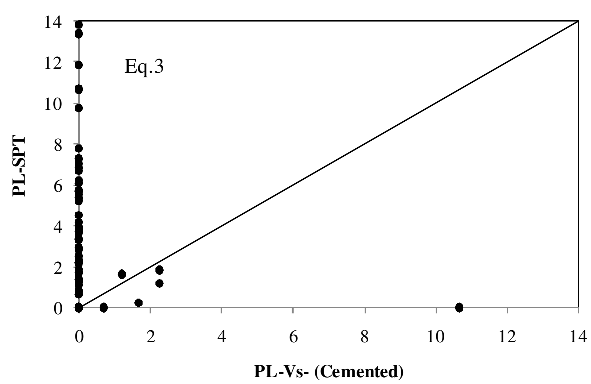
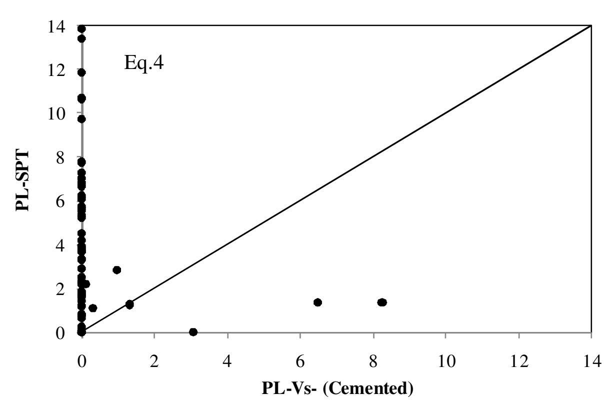


FIGURE 7: The comparison of PL values based on SPT and Vs analyses in the deep layers of soil in uncemented state


Eq.1
PL-SPT
PL-Vs-(Cemented)
0
2
4
6
8
10
12
14


Eq.2
PL-SPT
PL-Vs-(Cemented)
0
2
4
6
8
10
12
14


Eq.3
PL-SPT
PL-Vs- (Cemented)
0
2
4
6
8
10
12
14


Eq.4
PL-SPT
PL-Vs- (Cemented)
0
2
4
6
8
10
12
14

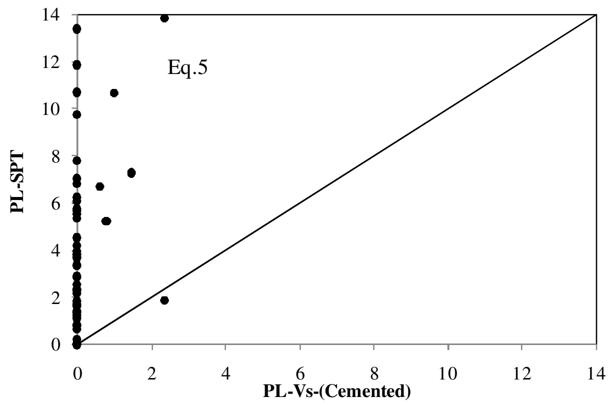


FIGURE 8: The comparison of PL values based on SPT and Vs analyses in the deep layers of soil in cemented sate

5- In order for the accurate /precise comparison, the consistency and mismatch of two methods at the same depth based on safety factor values were evaluated. The results are presented in Table 7. As illustrated below, there is proper/perfect adaption in the non-liquefaction of soil condition.

Type of Soil	State of Soil	Liquefied in SPT and Vs-Eq.1	Liquefied in SPT and Vs-Eq.2	Liquefied in SPT and Vs-Eq.3	Liquefied in SPT and Vs-Eq.4	Liquefied in SPT and Vs-Eq.5
Silt	**Cemented**	1	0	1	0	4
	Uncemented	1	1	0	0	0
Sand	**Cemented**	2	2	1	0	3
	Uncemented	0	1	1	1	0
Gravel	**Cemented**	0	0	2	0	0
	Uncemented	0	0	0	0	0
		Non-Liquefied in SPT and Vs-Eq.1	**Non-Liquefied in SPT and Vs-Eq.2**	**Non-Liquefied in SPT and Vs-Eq.3**	**Non-Liquefied in SPT and Vs-Eq.4**	**Non-Liquefied in SPT and Vs-Eq.5**
Silt	**Cemented**	114	115	114	115	111
	Uncemented	114	114	115	115	115
Sand	**Cemented**	192	192	193	194	191
	Uncemented	194	193	193	193	194
Gravel	**Cemented**	58	58	56	58	58
	Uncemented	58	58	56	58	58

TABLE 7: The comparison of analyses in layers at the same depth based on SPT and Vs methods using Five empirical relationships

As it can be observed, there is a significant difference between Seed and Idriss (1971-1985) simplified procedure based on Standard Penetration Test (SPT) results and the field performance curves proposed by Andrus et al. (2004) established on Shear wave velocity (Vs). This difference might be due to the inherent uncertainties in field performance data methods and empirical relationships.

The uncertainties in the field performance data methods include:
1- The uncertainties in the plasticity of the fines in the in situ soils.
2- Using post earthquake properties that do not exactly reflect the initial soil states before earthquakes.

3- The assumption that CRR_{field} is equal to CSR obtained from Seed and Idriss [1]. This may result in a significant overestimation of CRR_{field} when the safety factor is less than 1.
4- In determining the cyclic strength ratio (CRR) in shear wave velocity method the soil cementation factors (K_{a1} and K_{a2}) are calculated. The value of these parameters proposed by Andrus and Stokoe may be inappropriate in the study area.
5- The maximum shear wave velocity (V_{s1}^*) values for occurring liquefaction in soil recommended by Andrus et al. [25] may be unsuitable for the study area.
6- The value of parameters a and b in CRR equation in the shear wave velocity method perhaps is improper for the data range studies.
The uncertainties in the empirical relationships are:
1- The standard penetration resistance (N_{SPT}) is not estimated accurately and the test apparatus can be in error.
2- The empirical relationships used for the study perhaps is inappropriate for the data range and the type of soils in the study area.

6. CONCLUSION
The present study investigated the two field methods used to evaluate liquefaction potential of soils including Standard penetration Test (SPT) and Shear wave velocity test (Vs) based on empirical relationships between them. The comparison of the safety factor values and liquefaction potential index revealed that the severity/seriousness of liquefaction occurrence in the studied area based on Vs method is was lower than the one based on SPT based method. Furthermore, it can be observed that the relationships between Standard Penetration Test and shear wave velocity are not appropriate. Because the relationships used in the present study are dependent on soil type, fines content (clay and silt), type of tests and their accuracy, it would be much safer to perform both methods for the same place and then compare the results in order to evaluate the liquefaction potential. Last but not least, further studies are called for to obtain better relationships based on the type of soils within the area of the study.

7. REFERENCES

[1] Seed H B, Idriss I. M. "Simplified Procedure for Evaluating Soil Liquefaction Potential", Journal of the Soil Mechanics and Foundations Division, ASCE , Vol .97 ,SM9, pp 1249-1273, 1971.

[2] Seed H B, Idriss I. M. "Ground Motion and Soil Liquefaction during Earthquakes", EERI, ASCE, Vol.109, No.3, pp 458-482, 1982.

[3] Seed H B, Idriss I M, Arongo I. "Evaluation of Liquefaction Potential Using Field Performance Data", journal of Geotechnical Engineering, ASCE,Vol.109,NO.3,pp 458-482, 1982.

[4] Seed H. B, Tokimatso K, Harder L. F. "The influence of SPT procedures in soil liquefaction resistance evaluation", Geotechnical engineering, ASCE, No.111, Vol.12, 1985.

[5] Hardin B. O., Drnevich V. P. "Shear Modulus and Damping in Soils: Design Equation and Curve", Journal of Soil Mechanics and Foundation, Division, ASCE, Vol. 98, SM7, pp. 667-692, 1972.

[6] Kramer S. "Geotechnical Earthquake Engineering", Prentice Hall, New Jersey, 1996.

[7] Ishihara K. "Soil Behaviour in Earthquake Geotechnics", Oxford Engineering Science Series.Oxford University Press, 1996.

[8] Rauch A. F., Duffy M, Stokoe K. H. "Laboratory correlation of liquefaction resistance with shear wave velocity", Geotechnical Special Publishing, 110, pp 66–80, 2000.

[9] Huang Y. T., Huang A. B., Kuo Y. C., Tsai M. D. "A laboratory study on the undrained strength of a silty sand from Central Western Taiwan", Soil dynamic and Earthquake Engineering, Vol.24, pp 733–743, 2004.

[10] Chen Y. M., Ke H., Chen R. P. "Correlation of shear wave velocity with liquefaction resistance based on laboratory tests", Soil Dynamic and Earthquake Engineering, Vol.25, No.6, pp. 461–469, 2005.

[11] Zhou Y G, Chen Y M, Huang B "Experimental study of seismic cyclic loading effects on small strain shear modulus of saturated sands", Journal of Zhejiang University SCIENCE, 6A(3),pp 229-236, 2005.

[12] Ning Liu S M, Mitchell J K, Hon M. "Influence of non plastic fines on shear wave velocity-based assessment of liquefaction", Journal of Geotechnical and Geoenviromental Engineering, Vol. 132, No. 8, pp 1091-1097, 2006.

[13] Zhou Y G, Chen Y M "Laboratory investigation on assessing liquefaction resistance of sandy soils by shear wave velocity", Journal of Geotechnical and Geoenvironmental Engineering, ASCE,133(8),pp 959-972, 2007.

[14] Baxter C D P, Bradshaw A S, Green R A, Wang J. "Correlation Between Cyclic Resistance and Shear Wave Velocity for providence silts", Journal of Geotechnical and Geoenviromental Engineering, Vol.134, No. 1, pp 37-46, 2008.

[15] Askari F, Dabiri R, Shafiee A and Jafari M K "Liquefaction Resistance of Sand-Silt Mixtures using Laboratory based Shear Wave Velocity", International Journal of Civil Engineering ,Vol.9, No.2, pp. 135-144, 2011.

[16] Andrus R D "In-situ characterization of gravelly soils that liquefied in the 1983 Borah Peak Earthquake", Unpublished Ph.D. Thesis, University of Texas, 1994.

[17] Dabiri R, Askari F, Shafiee A and Jafari M K "Shear Wave Velocity-based Liquefaction Resistance of Sand-Silt Mixtures: Deterministic versus Probabilistic Approuch", Iranian Journal of Science and Technology- Transaction of Civil Engineering, Vol.35, No.C2, pp.199-215, 2011.

[18] Lodge A L "Shear Wave Velocity Measurements for Subsurface Characterization. Ph.d Dissertation" , University of California at Berkeley, 1994.

[19] Kayabali K "Soil Liquefaction Evaluation Using Shear Wave Velocity", Engineering Geology,Vol.44,No.4,pp 121-127, 1996.

[20] Andrus R D, Stokoe K H "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 4-5January 1996, Salt lake City, UT, National Center for Earthquake engineering Research,Buffalo,NY,pp 89-128, 1997.

[21] Juang C H, C J Chen "CPT-based liquefaction analysis, Part 1: Determination of limit state function", Geotechnique, Vol. 50, No. 5, pp 583-592, 2000.

[22] Andrus R D, Stokoe K H "Liquefaction resistance of soils from shear wave velocity", ASCE, 126 (11),pp 1015 – 1025, 2000.

[23] Iwasaki T, et al. "Microzonation for soil liquefaction potential using simplified methods", Proceeding, 1982, pp.1319-1330.

[24] Shafiee A, Kamalian M, Jafari M K and Hamzehloo H "Ground Motion Studies for Microzonation in Iran", Natural Hazard, pp.1-25, 2011.

[25] Andrus R D, Stokoe K H, Juang C H "Guide for Shear-Wave-Based Liquefaction Potential Evaluation", Eartquake Spectra, 2004.

[26] Jafari M. K., Shafiee A. and Razmkhah A "Dynamic Properties of Fine Grained Soils in South of Tehran", Journal of Seismology and Earthquake Engineering, Vol.4, No.1, pp.25-35, 2002

[27] Idriss I M "Evaluation of Liquefaction Potential, Consequences and Mitigation—An Update, Presentation Notes for Geotechnical Society Meeting", Held 17 Feb. 1998, Vancover, Canada.

[28] Idriss I M. An Update of the Seed-Idriss Simplified Procedure For Evaluation Liquefaction Potential, Presentation Notes for Transportation Research Board Workshop on New Approaches to Liquefaction Analysis, Held 10 Jan. 1999, Washington D.C.

[29] Skempton A. K."Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Aging and Overconsolidation", Journal of Geotechnique,Vol.36,No.3,pp.425-447, 1986.

[30] Hynes M E, Olsen R S "Influence of Confining Stress on Liquefaction Resistnce. Proc.International workshop on Physics and Mechanics of Soil Liquefaction, Balkema, Rotterdom, The Netherlands, pp 145-152, 1999.