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Evaluating the Liquefaction Potential of Soil in the South and Southeast of Tehran based on the Shear Wave Velocity through Empirical Relationships

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Abstract

The liquefaction resistance of soil can be evaluated using laboratory tests such as cyclic simple shear, cyclic triaxial, cyclic torsional shear as well as field methods like Standard Penetration Test (SPT), Cone Penetration Test (CPT) and Shear Wave Velocity (Vs). In this regard, this study attempts to compare the results of the SPT based on the simplified procedure proposed by Seed and Idriss (1985) and those of the Vs on the basis of Andrus et al.'s (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 the south and southeast of Tehran. It is observed that there is not a perfect agreement between the results of the two methods based on five empirical relationships assuming cemented and non-cemented condition for soils. Moreover, the liquefaction potential index (PL) value in the SPT method is more than that of the Vs method.

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

1. Introduction

The simplified procedure is used to predict the liquefaction resistance of soils worldwide. It was originally developed by Seed and Idriss [1] using the Standard Penetration Test (SPT) blow counts correlated with the cyclic stress ratio (CSR), which is a parameter representing the seismic loading on the soil. This procedure has undergone several revisions since then and has been updated [2-4]. In addition, other procedures have been developed based on the Cone Penetration Test (CPT), Becker Penetration Test (BPT), and small-strain Shear Wave Velocity (Vs) measurements. Among them, the Vs is suitable for determining the liquefaction resistance because both Vs and liquefaction resistance are influenced by factors such as confining stress, soil type/plasticity and relative density [5- 7] and the 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 developed from laboratory studies [8-15], analytical studies [16-17], penetration-Vs equations [18-19], and Vs measurements at earthquake shaken sites [20-22] have been proposed to estimate the liquefaction resistance based on the Vs. Some of these procedures follow the general format of Seed- Idriss' simplified procedure in 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 the Vs and SPT methods of soil liquefaction potential evaluation in the south of Tehran. The liquefaction potential index is also calculated by Iwasaki et al.'s [23] procedure for both methods.

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2. General condition and soil stratification

In order to evaluate the liquefaction potential of soils using the two field methods mentioned above, geotechnical information of 67 boreholes in the south and southeast of Tehran including 11 to 16 municipality areas were collected (Figure 1). As the type of soil and geotechnical properties can affect the liquefaction potential, in this study only the gravely sand, silty sand and silty soils are studied.

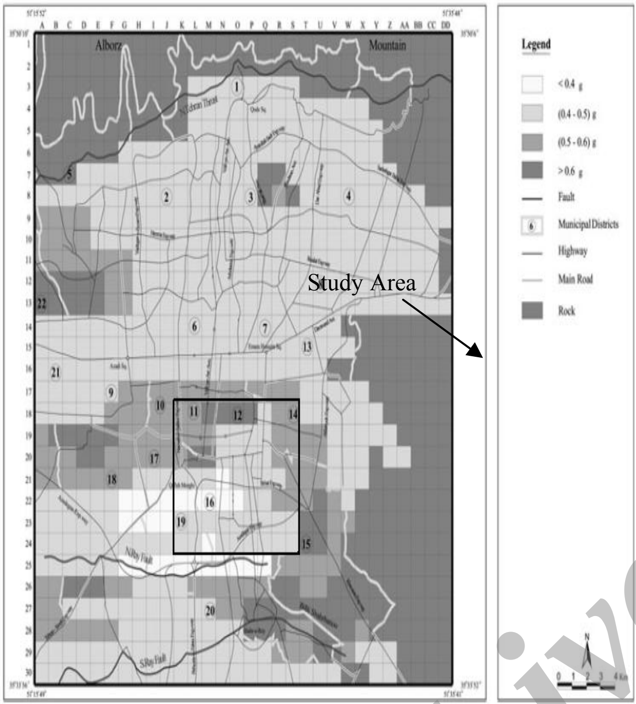

Fig.1. The study area and PGA distribution throughout Tehran for an earthquake corresponding to 475 year return period [24]

The Tehran plain mainly consists of Quaternary formations. These formations are often the result of erosion and redeposition of former sediments. The Tehran plain 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 an alluvium of various types, thicknesses and grain sizes to be formed. The Tehran plain is divided into five units including 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 the Tehran plain (Figure 2). The general characteristics of these different units are presented in Table 1 [24].

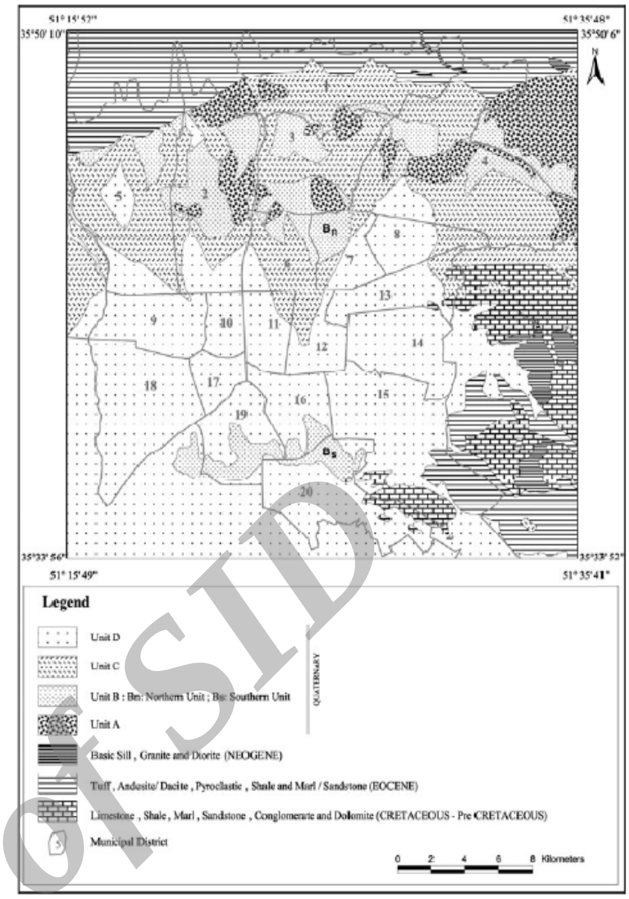

Fig.2. The geological map of Tehran [24]

Table 1. Characteristics of the units of the Tehran plain [24]

Unit	Period	Formation	Constituting material
A	Pilo-Pleistocene	Hezardareh	Conglomerate 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

3. Evaluation of the liquefaction potential in the study area

The peak ground acceleration (PGA) is necessary for the analysis of boreholes to evaluate the liquefaction potential of soils. The PGA values are selected in each borehole position according to Figure 1. In addition, the depth of ground water table is considered in the liquefaction potential assessment of soils. To define the critical ground water level in the boreholes, the maps of variations in the underground water depth in the Tehran plain are used. In the Vs measurement method based on Andrus et al. 's [25] process for assessing the liquefaction potential, the Vs amounts are calculated using empirical equations between the Vs and the SPT blow count (N) for all of soil types as follows [24-26]:

$$V_s = 61.N^{0.5} \quad (1)$$

$$V_s = 97.N^{0.314} \quad (2)$$

$$V_s = 76.N^{0.33} \quad (3)$$

$$V_s = 121.N^{0.27} \quad (4)$$

$$V_s = 22.N^{0.85} \quad (5)$$

4. Assessment of the liquefaction potential

The evaluation procedures based on the SPT [4] and the Vs [25] require the measurement of three parameters: (1) the level of cyclic loading on the soil caused by the earthquake, expressed as the cyclic stress ratio (CSR); (2) the stiffness of the soil, expressed as the overburden stress corrected SPT blow count and Vs; and (3) the resistance of the soil to liquefaction, expressed as the 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 i at the 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 3).

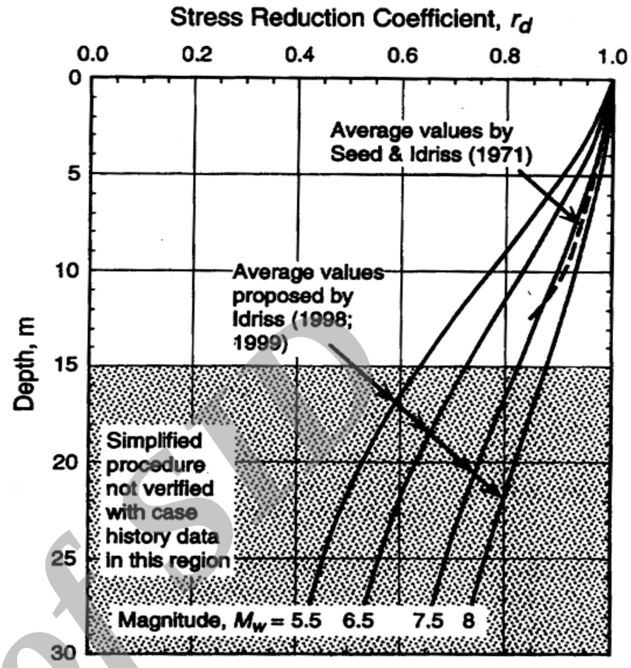

Fig.3. Variations of the stress reduction coefficient with depth and earthquake magnitudes [27, 28]

4.2. Corrected SPT Blow count and Shear Wave Velocity

In addition to the fines content and the grain characteristics, other factors affect the SPT results as reported in Table 2. Eq. (7) incorporates these factors:

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

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

Table 2. Corrections in the SPT [29]

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	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 〉30m	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

In the procedure of liquefaction potential evaluation proposed by Andrus et al. [25], Vs should be corrected to overburden stress. In this regard, Eq.(8) is as follows:

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

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 to100kPa, σ'_V shows the effective overburden stress, and K_0', is the coefficient of effective earth pressure (which is assumed to equal 0.5 in this study).

4.3. Cyclic Resistance Ratio (CRR)

In the simplified procedure, Figure 4 is the graph of the calculated CSR and corresponding $(N_1)_{60}$ data from sites where liquefaction effects were observed following the past earthquakes of approximately 7.5 magnitude. The CRR curves on this graph were conservatively positioned to separate the regions with the data indicative of the liquefaction from the regions with the data indicative of non-liquefaction. The curves were developed for granular soils with the fine contents of 5% or less, 15%, and 35% as shown on the plot.

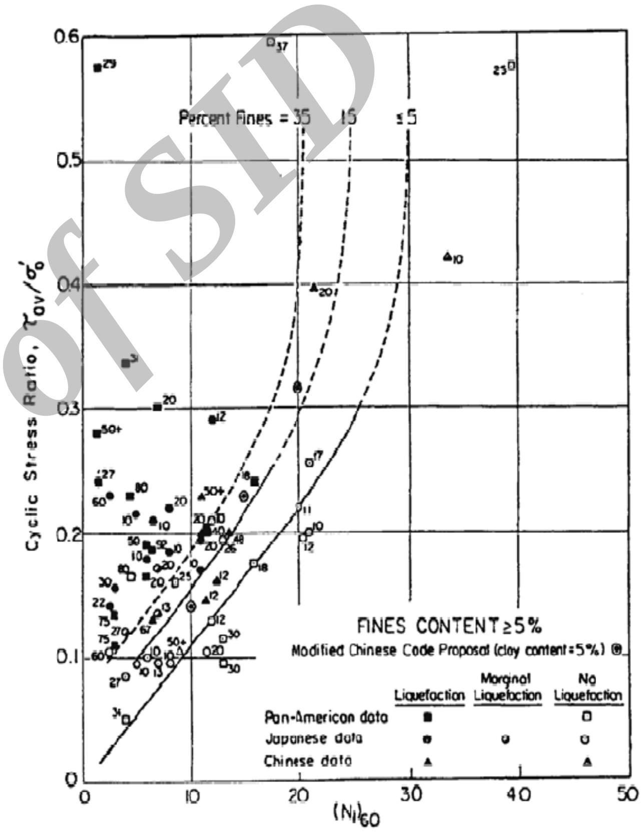

Fig.4. The liquefaction resistance curves by Seed et al. for 7.5 magnitude earthquakes [4]

Furthermore, in the Vs 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}. Figure 5 depicts the CRR-V_{s1} curves by Andrus et al. [25] for 7.5 magnitude earthquakes.

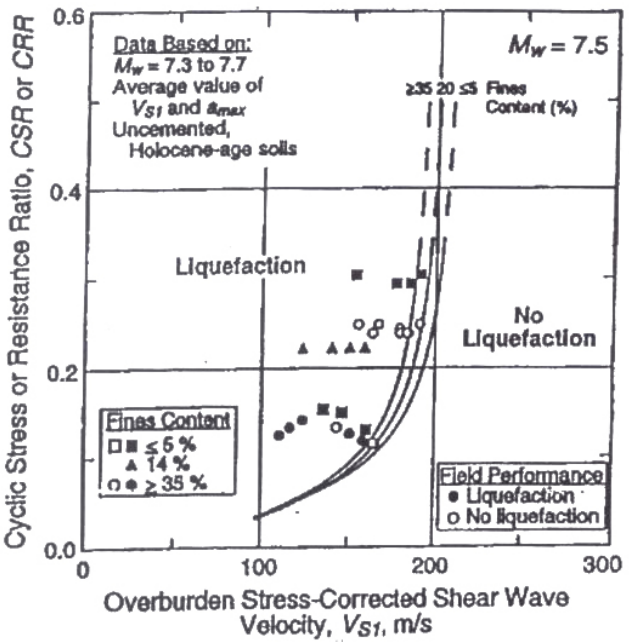

Fig. 5. The liquefaction resistance curves by Andrus et al. [25] for 7.5 magnitude earthquakes

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

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

where MSF is the magnitude scaling factor, V_{s1}^* is the limiting up value of V_{s1} for the liquefaction occurrence, K_{a1} is a factor that corrects high V_{s1} values caused by aging, and K_{a2} is a factor that corrects the influence of age on the CRR. Andrus et al. [25] suggest the following relationships for estimating MSF and V_{s1}^*:

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

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

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

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

where FC is the fines content.

In this study, since the earthquake magnitude (Mw) is assumed to be 7.5, MSF equals to 1.0. Both K_{a1} and K_{a2} factors equal to 1.0 for the incremented soils of the Holocene age. For the older and cemented soils, K_{a1} factor is evaluated using the curves in Figure 6. If the soil conditions are unknown and penetration data are not available, the assumed value for K_{a1} will be 0.6 [24].

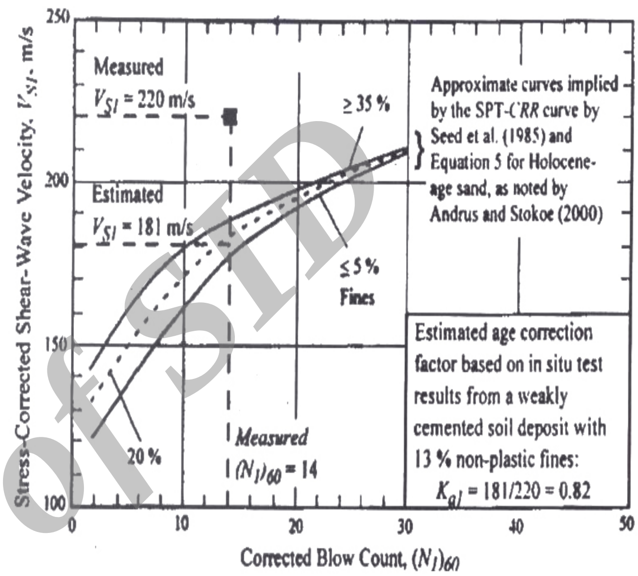

Fig.6. The suggested method for estimating K_{a1} from the SPT and V_s measurements at the same site [24]

In both methods, if the effective overburden stress is greater than 100kPa in the question depth, the CRR value is corrected through Figure 7 and the following equations [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 [30].

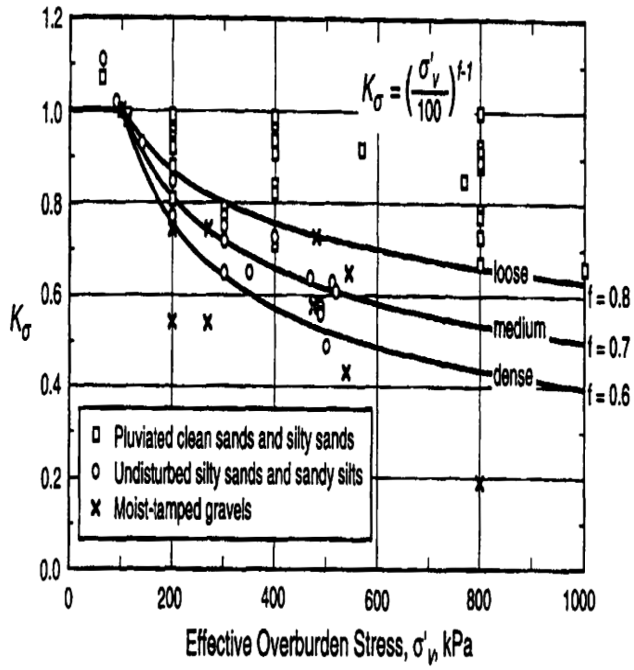

Fig.7. Variations of K_σ values versus the effective overburden stress [30]

4.4. Safety Factor

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

$$Fs = \frac{CRR_J}{CSR} \tag{14}$$

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

4.5. *Liquefaction Potential Index (P_L)*

Liquefaction potential index (P_L) introduced by Iwasaki et al. [23] quantifies the severity of possible liquefaction at any site. It is defined as follows:

$$P_L = \int_0^{20} F(Z).W(Z).dZ \tag{15}$$

$$F(Z)= 1-F_S \tag{16}$$

$$W(Z)=10-0.5Z \tag{17}$$

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 3 is from 0 to 100. In the present study, PL values are measured and then compared for both methods.

Table 3. Description of the liquefaction potential index (P_L)

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 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.

5. Evaluating the results of data analysis

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

1- Liquefaction potential index (P_L) values based on the SPT method are presented in Table 4. The results, drawing on Table 3, show that 51% of the data have a low liquefaction risk.

Table 4. Liquefaction potential index (P_L) values based on the SPT analysis

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

2- P_L values based on the Vs method using the five empirical relationships (Eqs.1 to 5) for both un-cemented and cemented soils are reported in Tables 5 and 6. The results show that the relations used are overestimated and most of them show the non-liquefaction condition for the soils in the studied area.

Table 5. The liquefaction potential index (P_L) values based on the 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	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 6. Liquefaction potential index (P_L) values based on the Vs analysis in the uncemented 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

3- The analysis of about 529 soil layers in 67 boreholes, the calculated liquefaction potential of soils, and the results of all types of soils are presented in Table 7. According to this table, there is no compatibility between the two procedures regarding the soil liquefaction expression for the two states. Yet, both of them show suitable harmony in the non-liquefaction condition for soils.

Table 7. The results of estimating the liquefaction potential in question depths using the SPT and Vs methods based on the five empirical relationships

	SPT	Vs Cemented				
Type of Soil	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
		Un-cemented				
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
		Un-cemented				
Silt	123	179	179	180	180	180
Sand	193	274	273	273	273	274
Gravel	59	75	75	75	75	75

4- The liquefaction potential index (PL) values based on the SPT and Vs methods in the incremented and cemented states for soils are presented in Figures 8 and 9. As these figures indicate, the results are consistent with the values in the tables above, and the liquefaction potential of soils based on the Vs is less than that of the SPT procedure using the empirical relationships.

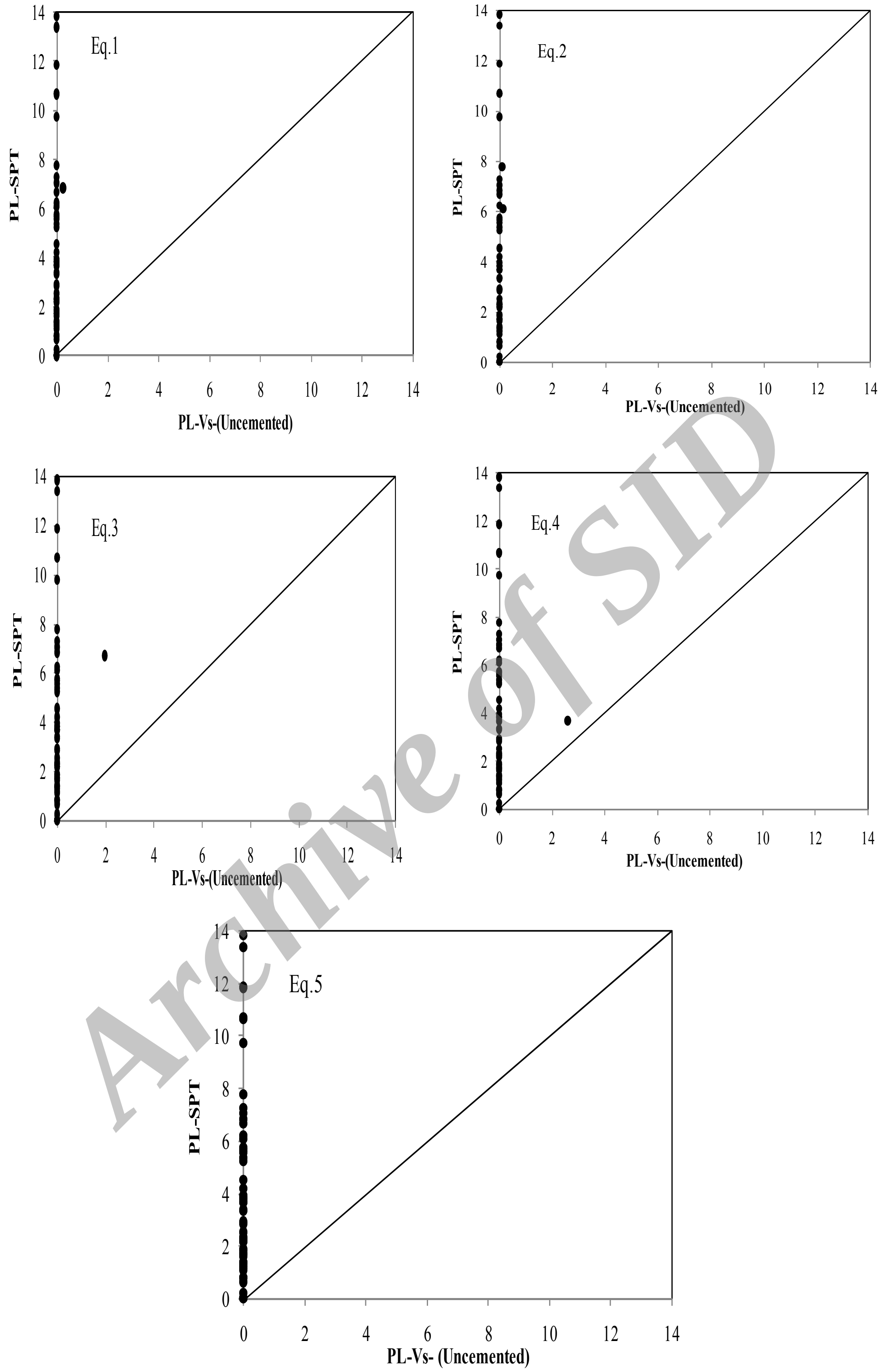

Fig.8. The comparison of PL values for the deep layers of soil in the
Un-cemented state based on the SPT and the Vs

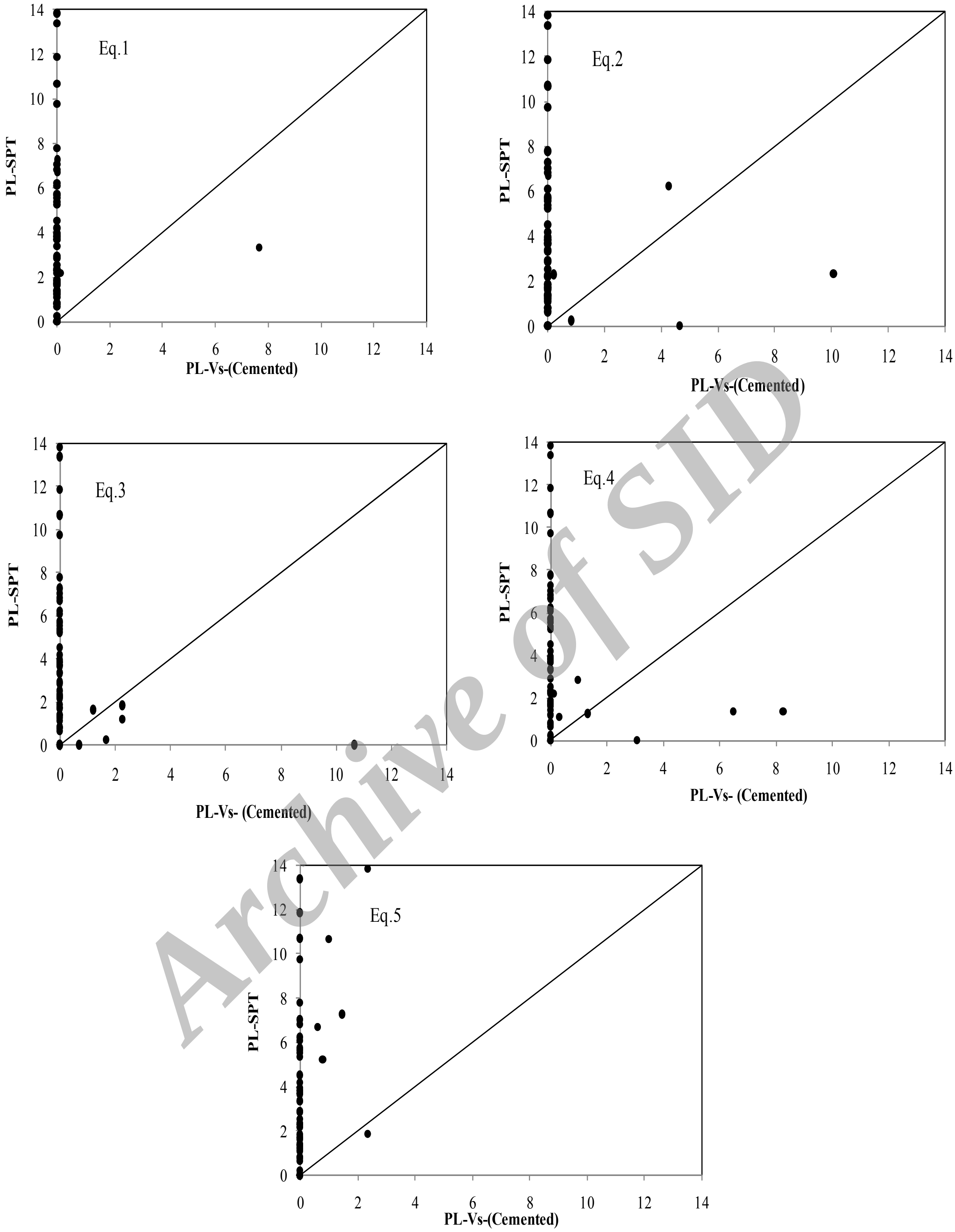

Fig.9. The comparison of PL values for the deep layers of soil in the cemented sate based on the SPT and the Vs

5- In order to compare the two methods accurately, their consistency and mismatch at the same depth based on the safety factor values were evaluated. The results presented in Table 7 reveal that there is proper/perfect adaption in the non-liquefaction condition of soil.
As Table 7 shows, there is a significant difference between Seed and Idriss's (1971-1985) simplified procedure based on the SPT results and the field performance curves proposed by Andrus et al. [25] based on the Vs. This difference may be due to the inherent uncertainties in the 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 the CSR obtained from Seed and Idriss (1971). This may result in a significant overestimation of CRR_{field} when the safety factor is less than 1.
4- To determine the CRR in the Vs method, the soil cementation factors (K_{a1} and K_{a2}) are calculated. The value of these parameters proposed by Andrus et al. [25] may be inappropriate in the study area.
5- The maximum shear wave velocity (V_{s1}^{*}) values for occurring liquefaction in the soil recommended by Andrus et al. [25] may be unsuitable for the study area.
6- The value of parameters a and b in the CRR equation in the Vs method is probably improper for the data range studies.
The uncertainties in the empirical relationships are:
1- The standard penetration resistance (NSPT) is not estimated accurately and the test apparatus can be in error.
2- The empirical relationships used in the study may be inappropriate for the data range and the types of soils in the study area.

Table 7. The comparison of analyses of the layers at the same depth based on the SPT and Vs methods using the five empirical relationships

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

6. Conclusion

The present study was an attempt to investigate the two field methods of SPT and Vs used to evaluate the liquefaction potential of soils based on the empirical relationships between them. The comparison of the safety factor values and the liquefaction potential indexes shows that the severity/seriousness of liquefaction occurrence in the studied area based on the Vs method is lower than that based on the SPT method. Furthermore, it is observed that the relationships between the SPT and the Vs are not appropriate. As the relationships used in the present study are dependent on the 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. For the future research, more studies may be conducted to obtain better relationships based on the types of soils within the area of the study.

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