Article

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Evaluation of liquefaction potential of soil using the shear wave velocity in Tehran, Iran

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ABSTRACT: Shear-wave velocity (V_S) offers a means to determine the seismic resistance of soil to liquefaction by a fundamental soil property. Iwasaki's (1982) method is used to measure the liquefaction potential index for both of them. It follows the general format of the Seed-Idriss (1685) simplified procedure based on standard penetration test blow count and shear wave velocity (V_S) on the basis of Andrus et al. (2004) using case history data from 43 boreholes in soils ranging from fine sand, silty sand, gravely sand to profiles including silty clay layers and the average soil shear wave velocity (V_S^{30}) in the south Tehran. Liquefaction resistance curves were established by applying a modified relationship between the shear-wave velocity and cyclic stress ratio for the constant average cyclic shear strain. The study area is the south-east of Tehran and the route of Tehran Metro Line 7. 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 Standard Penetration Test (SPT) method is more than that of the V_S method. Liquefaction potential index (PL) values based on shear wave velocity (V_S) using five empirical relationships in two un-cemented and cemented soil show that the used relations are overestimated and most of them have shown non-liquefaction condition for soils in the studied area.

Key words: liquefaction, standard penetration test, shear wave velocity, liquefaction potential index, southeast of Tehran

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1. INTRODUCTION

Of the several field techniques routinely used to assess triggering of seismic soil liquefaction [standard penetration test (SPT), cone penetration test (CPT), Becker hammer test (BHT), shear-wave velocity (V_S)], only the shear-wave velocity test measures a fundamental property of the soil. Nevertheless, liquefaction assessment correlations based on in situ penetration index tests are more widely used in engineering practice to estimate the potential for triggering or initiation of seismically-induced soil liquefaction. Compared with V_S, SPT and CPT penetration methods have the advantage of correlating more directly with relative density, which has a strong effect on the cyclic behavior of saturated soil (Idriss et al., 2008). In contrast, V_S is considerably less sensitive to problems of soil compression and reduced penetration resistance when soil fines are present, compared with SPT and CPT penetration methods. Therefore, V_S requires only minor corrections for fines content (FC).

Evaluation of the liquefaction resistance of soils is an important step in many geotechnical investigations in earthquake prone regions. The procedure widely used in the United States and throughout much of the world for evaluating soil liquefaction resistance is termed the "simplified procedure." This simplified procedure was originally developed by Seed et al. (1971) using blow counts from the standard penetration test (SPT) correlated with a parameter called the cyclic stress ratio that represents the cyclic loading on the soil. Since 1971, this procedure has been revised and updated (Seed, 1979; Seed et al., 1982; Seed et al., 1985; Youd et al., 1997). In the mid-1980s, a parallel procedure based on the cone penetration test (CPT) was introduced by Robertson et al. (1985), which also has been revised and updated (Seed et al., 1986; Stark et al., 1995; Olsen, 1997; Robertson et al., 1998).

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Over the past 20 years, numerous studies have been conducted to investigate the relationship between V_S and liquefaction resistance. These studies involved field performance observations (Stokoe et al., 1985; Robertson et al., 1992; Kayen et al., 1992; Andrus et al., 1997), penetration-V_S correlations (Seed et al., 1983; Lodge, 1994), analytical investigations (Bierschwale et al., 1984; Stokoe et al., 1988), and laboratory tests (Dobry et al., 1981; De Alba et al., 1984; Tokimatsu et al., 1990). Several of the liquefaction evaluation procedures developed from these studies follow the general format of the Seed-Idriss simplified procedure, where V_S is corrected to a reference overburden stress and correlated with the cyclic stress ratio. Nearly all were developed with limited or no field performance data.

Some of these procedures follow the general format of Seed-Idriss simplified procedure in which the V_S is corrected to a reference vertical stress and correlated with the cyclic stress ratio. This paper presents the results of the comparison between the V_S and SPT methods of soil liquefaction potential

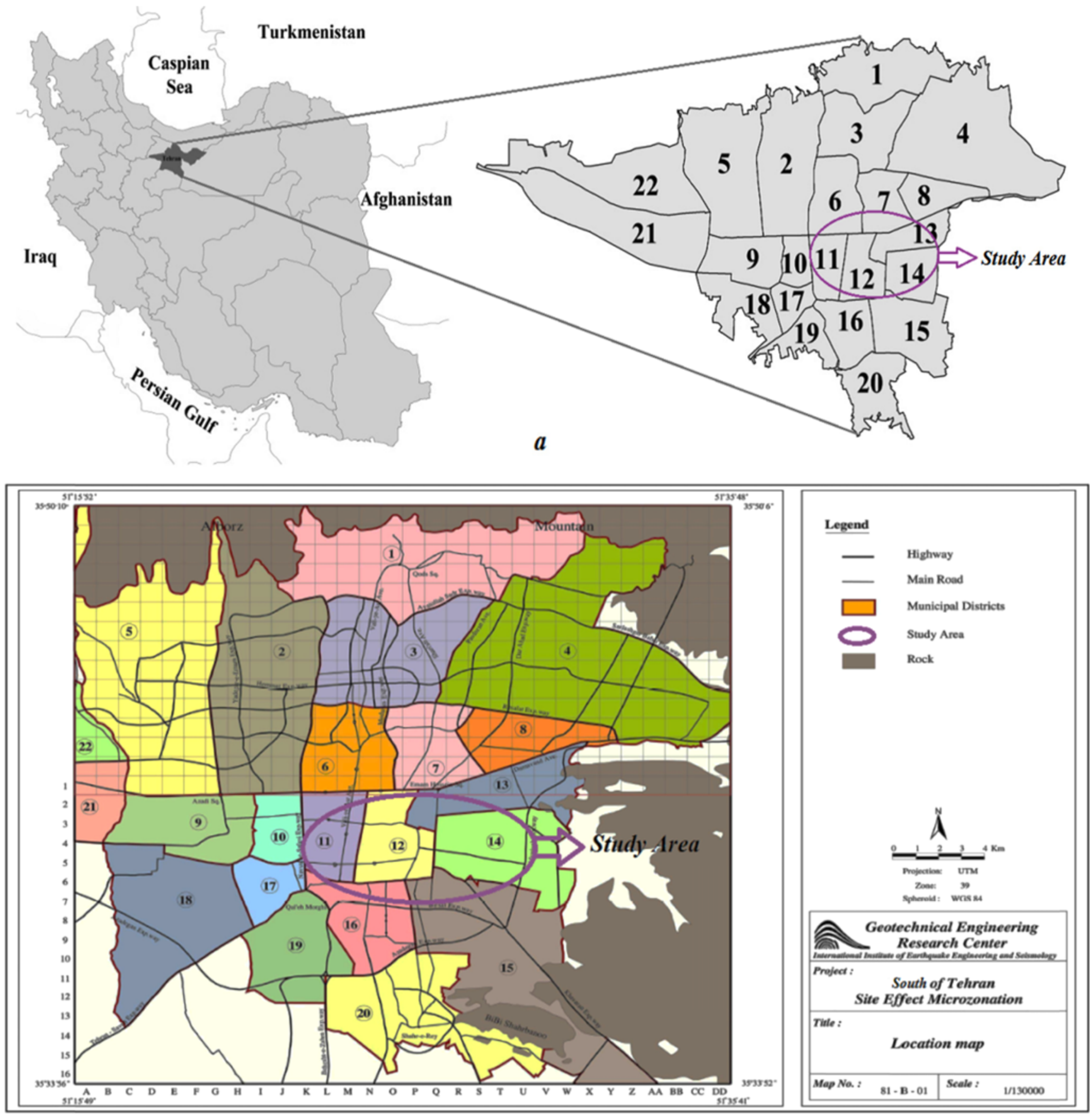

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Fig. 1. (a) Tehran geographic location; (b) The study area in the Southeast of Tehran.

evaluation in the south-east of Tehran. The liquefaction potential index is also calculated by Iwasaki et al. (1982) procedure for both methods.

2. GENERAL CONDITION AND SOIL STRATIFICATION

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.

In order to evaluate the liquefaction potential of soils using two field methods, geotechnical information of 13 boreholes in the southeast of Tehran including 11, 12 and 14 municipality areas were collected (Fig. 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. A total of 13 boreholes were drilled but the data of 6 of them were available for analysis (Table 1; Fig. 2).

3. ANALYSIS OF BOREHOLES TO EVALUATE THE LIQUEFACTION POTENTIAL

The peak ground acceleration (PGA) is necessary for the

Table 1. Computed values of soil layers of the study

Depth (m)	γ	σ_v	σ'_v	C_N	r_d	$(N_1)_{60}$	τ_{cyc}	CSR	$\tau_{cyc,L}$	FS
5	15.7	78.5	78.5	1. 104	0.975	10.33	11.641	0.125	9.846	0.845
8	16	126.5	126.5	0.87	0.941	13.78	18.105	0.152	19.288	1.06
10	16.87	160.1	160.1	0.773	0.924	18.39	22.50	0.171	27.377	1.21
16	16.1	256.7	253.04	0.612	0.775	11.016	30.259	0.117	29.60	0.97
17	17	281.33	277.94	0.585	0.708	14.32	30.297	0.138	38.355	1.26
22	16.7	357.35	356.05	0.517	0.612	7.07	33.262	0.087	30.979	0.93
24	17.2	391.73	390.38	0.494	0.579	12.45	34.498	0.124	48.407	1.4
26	16.9	433.14	432.45	0.470	0.556	10.82	37.288	0.119	51.46	1.38
28	17.6	460.42	459.05	0.456	0.545	11.16	38.166	0.114	52.33	1.37
33	17.9	549.92	544.89	0.419	0.521	14.78	43.577	0.118	63.752	1.46
45	17.2	756.32	750.90	0.357	0.482	9.51	55.447	0.0679	51.027	0.92
50	18.1	846.82	836.74	0.339	0.441	18.30	56.801	0.0651	54.471	0.958

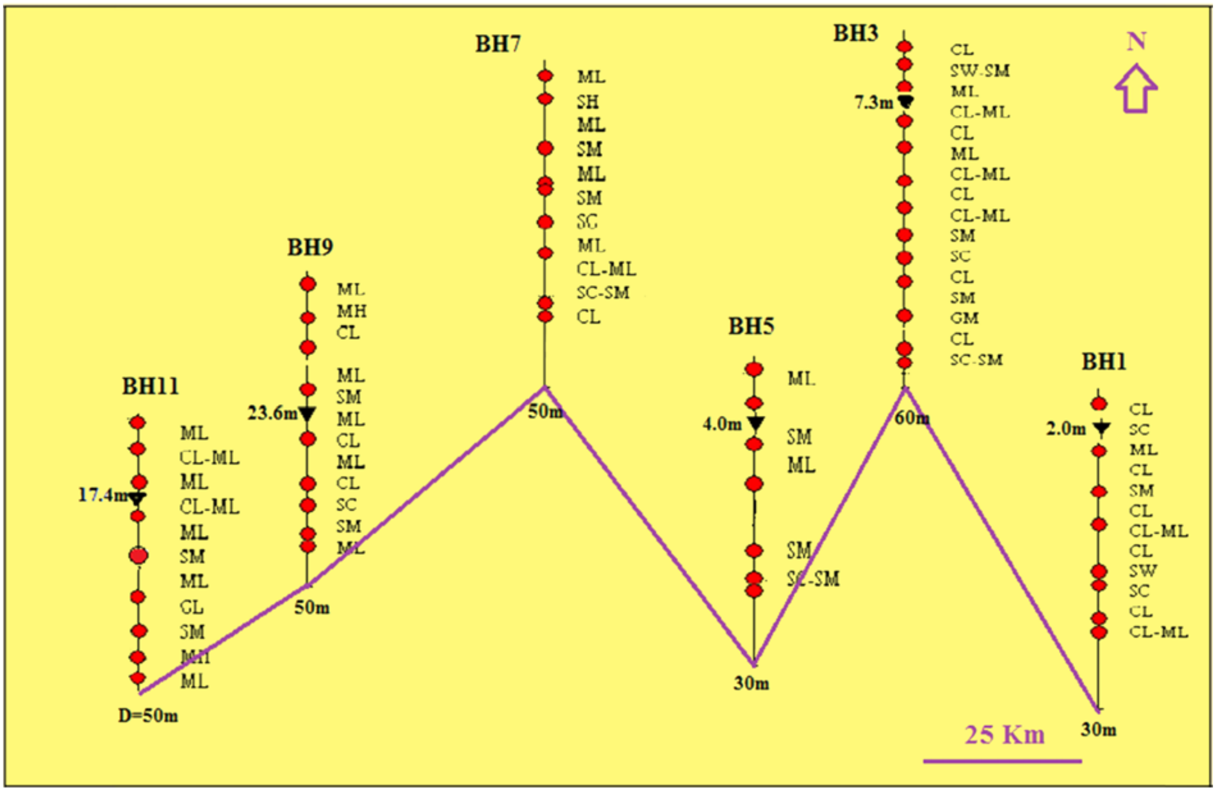

Fig. 2. Boreholes location and layer classification and groundwater level on Boreholes of the area. (BH7) Is there none of GWL.

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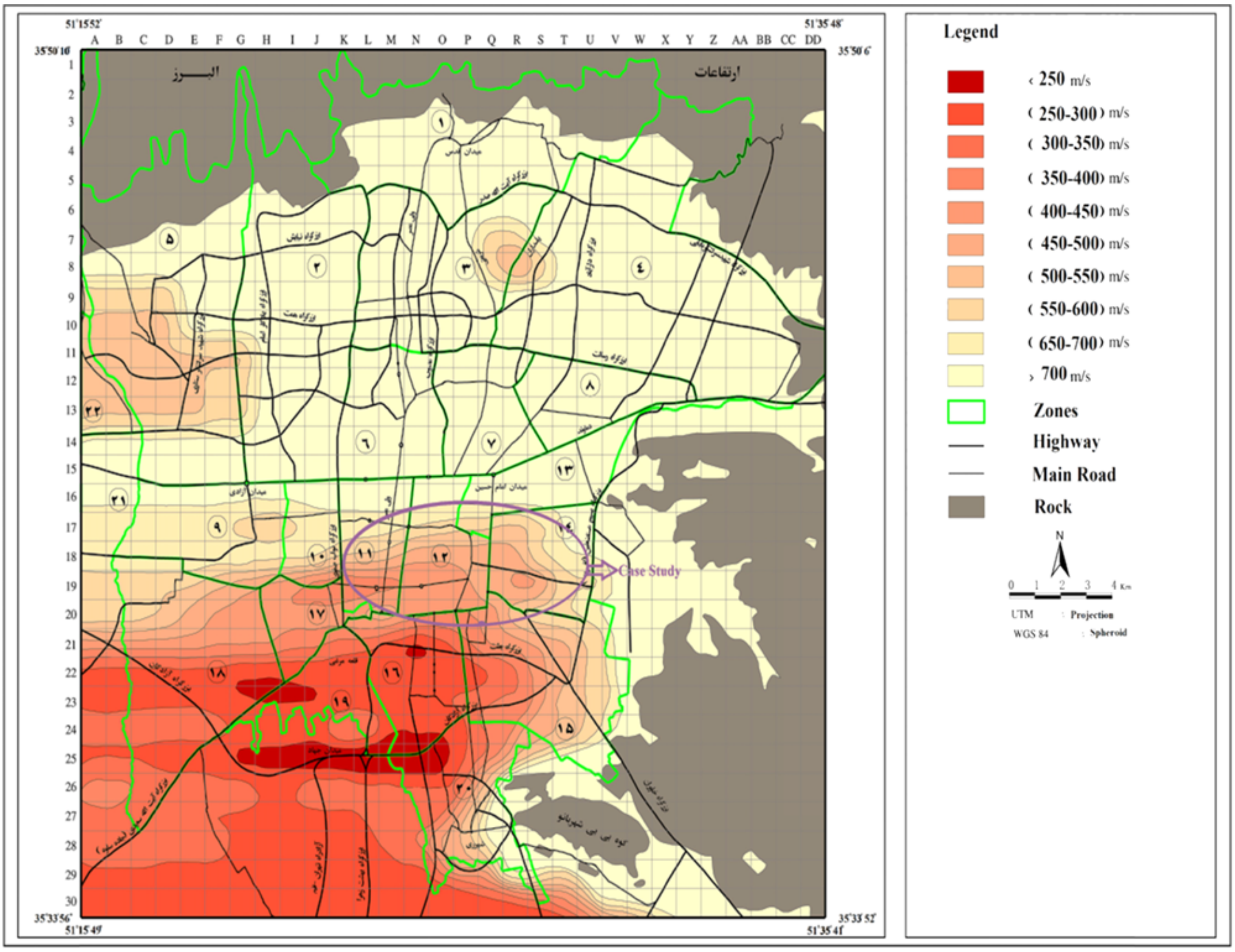

Fig. 3. The average soil shear wave velocity (V_S^{30}) in south Tehran and study area.

analysis of boreholes to evaluate liquefaction potential of soils. Therefore, the maximum acceleration is assumed to 0.35 for earthquake with magnitude 7.5. 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 (V_S) measurement method based on Andrus et al. (2004) (Fig. 3). Andrus et al. (2004) process for assessing liquefaction potential, V_S amounts were calculated using empirical equations between shear wave velocity and SPT blow count (N) for all soil types as follow,

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

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

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

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

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

4. ASSESSMENT OF LIQUEFACTION POTENTIAL

The evaluation procedure based on Standard Penetration Test (SPT) by simplified method Seed et al. (1985) and measurement of shear wave velocity (V_S) by Andrus et al. (2004) requires the calculation of three parameters: (1) The level of cyclic loading on the soil caused by the earthquake, expressed as a cyclic stress ratio (CSR); (2) stiffness of the soil, expressed as an overburden stress corrected shear wave velocity; and (3) resistance of the soil to liquefaction, expressed as a cyclic resistance ratio (CRR). Guidelines for calculating each parameter is discussed below.

4.1. Cyclic Stress Ratio (CSR)

The cyclic stress ratio, CSR, at a particular depth in a level soil deposit can be expressed (Seed and Idriss, 1971):

$$\mathrm{CSR} = \frac{t_{av}}{\sigma_v} = 0.65\left(\frac{a_{max}}{g}\right)\cdot\left(\frac{\sigma_v}{\sigma_v'}\right)\cdot r_d\ , \quad (6)$$

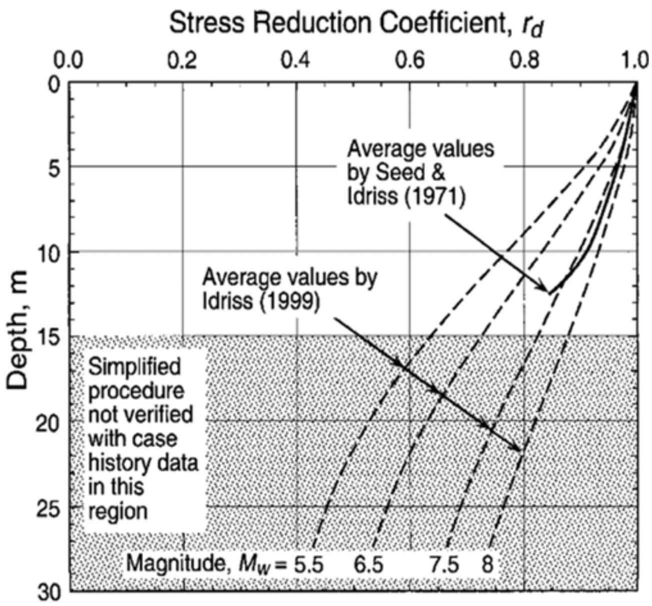

Fig. 4. Variations of stress reduction coefficient with depth and earthquake magnitudes (Idriss, 1999).

where τ_{av} is the average equivalent uniform cyclic shear stress caused by the earthquake and is assumed to be 0.65 of the maximum induced stress, a_{max} is the peak horizontal ground surface acceleration at the surface, g is the acceleration of gravity, σ_V is the total vertical overburden stress, σ'_V is the initial effective vertical (overburden) stress in kPa,, and rd is a shear stress reduction coefficient to adjust for flexibility of the soil profile (generally in the range ≈ 0.8 to 1). Values of rd are commonly estimated from the chart by Seed et al. (1971) (Fig. 4).

Their average curve was determined analytically using a variety of earthquake motions and soil conditions. Revised average r_d values have been proposed by Idriss (1999) based on the analytical work by Golesorkhi (1989). Unlike the original r_d values, these revised r_d values are magnitude dependent. As shown in Figure 4, the revised r_d curve for moment magnitude M_W = 7.5 is almost identical to the average curve published by Seed and Idriss (1971).

4.2. Corrected SPT Blow Count and Shear Wave Velocity

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

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

where $(N_1)_{60}$ is the SPT blow count number normalized to an overburden pressure of 1 ton/ft^2 (96 kPa) and corrected to an energy ratio of 60%, 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.

The shear-wave velocity of soil is influenced by effective overburden stress, and the void ratio of the soil (Hardin et al., 1972). For a given soil, V_S correlates directly with liquefaction resistance through the relationship between void ratio and relative density. It is possible that a soil type of unusual origin will correlate differently given the soil's specific void ratio-relative density relationship. The development of a generalized V_{S1}-liquefaction correlation requires the cautionary understanding that some soils with unusual void ratio-relative density characteristics exhibit liquefaction behavior that differs from the generalized relationships proposed in the past, as well as those proposed in this paper. Typically, the field measurement of V_S (as with SPT and CPT) is corrected to a normalized V_{S1} at the reference stress of 100 kPa. Liquefiable soils on approximately level ground are assumed to be normally consolidated (K'_0~0.5), and by convention (Kayen et al., 1992; Robertson et al., 1992),

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

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	Pa = 100 kPa
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

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normalized reference stress (P_a), and the stress exponent 0.25, shear wave velocity should be corrected to overburden stress by (8),

$$V_{S1} = V_S\left(\frac{Pa}{\sigma_v'}\right)\cdot\left(\frac{0.5}{K_0'}\right)^{0.125}, \tag{8}$$

where V_{S1} is the overburden stress-corrected shear-wave velocity, V_S is the shear wave velocity (m/s), P_a is a reference stress of 100 kPa or about atmospheric pressure, σ'_V is the initial effective vertical (overburden) stress in kPa, and K_0 is the coefficient of effective earth pressure ($K'_0 = 0.5$).

4.3. Cyclic Resistance Ratio (CRR)

The value of CSR separating liquefaction and non liquefaction occurrences for a given V_{S1}, or corrected penetration resistance, is called the cyclic resistance ratio CRR (Fig. 5). 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.

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} (Fig. 6).

For the earthquakes of 7.5 magnitudes, Andrus et al. (1997) proposed the following relationship between CRR and V_{S1},

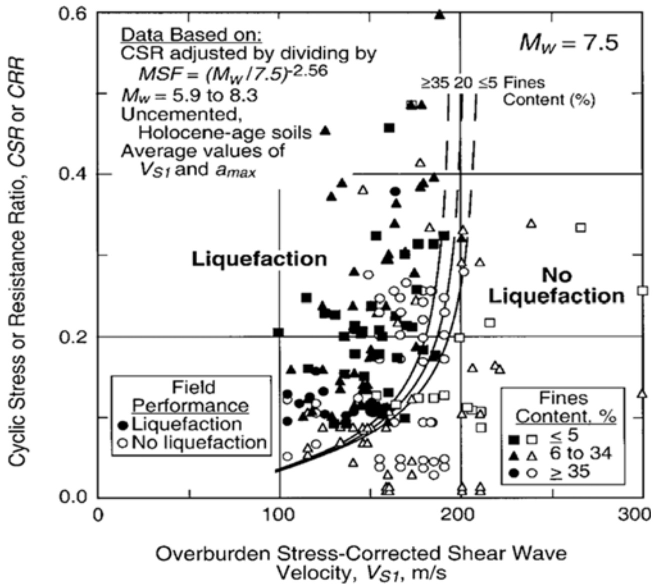

Fig. 6. the CRR- V_{S1} curves by Andrus et al. (2004) For the earthquakes of 7.5 magnitudes.

$$CRR = K_{a2}\left\{0.022\left(\frac{K_{a1}V_{S1}}{100}\right) + 2.8\left(\frac{1}{V_{S1}^{*} - K_{a1}V_{S1}} - \frac{1}{V_{S1}^{*}}\right)\right\}MSF, \tag{9}$$

where K_{a1} is a factor to correct for high V_{S1} values caused by aging, K_{a2} is a factor to correct the influence of age on CRR (Ka_1 and Ka_2 = 1.0), V_{S1} is the overburden stress-corrected shear-wave velocity, V_{S1}^{*} is the limiting upper value of V_{S1} for

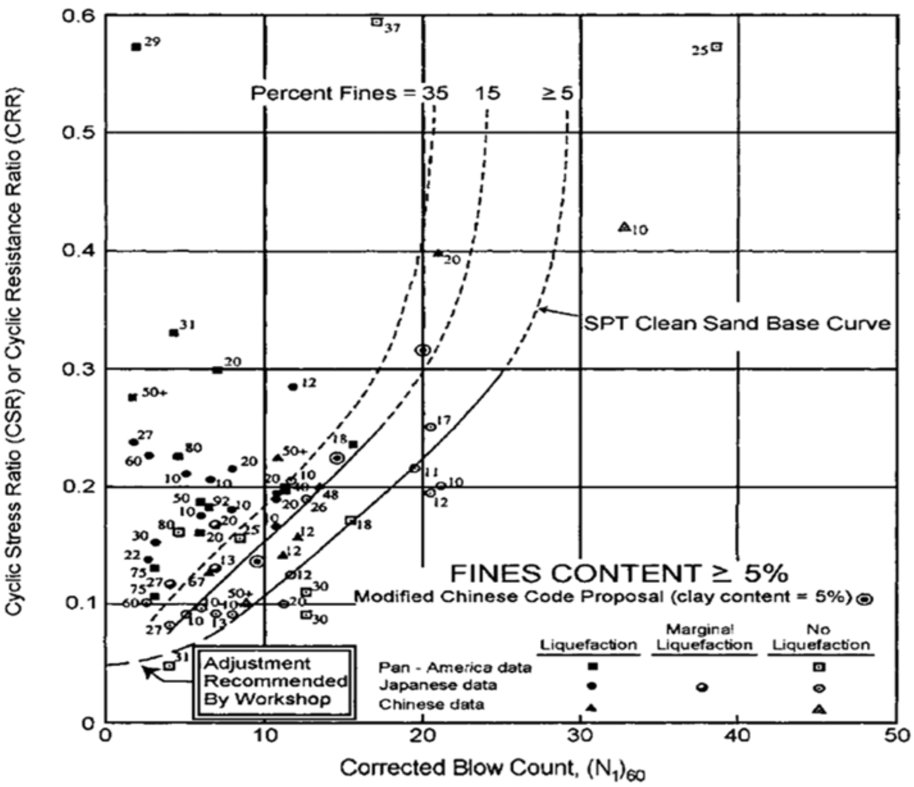

Fig. 5. Relationship between cyclic stress ratios causing liquefaction and $(N1)_{60}$ values for silty sands in M = 7.5 earthquakes (Seed et al., 1985).

cyclic liquefaction occurrence, and MSF magnitude scaling factor to account for the effect of earthquake magnitude.

Andrus et al. (2004) assumed Ka_1 and Ka_2 = 1.0 for all the Holocene-age case histories they considered. This assumption is evaluated in this paper.

The magnitude scaling factor is traditionally applied to CRR, rather than the cyclic loading parameter CSR, and equals 1.0 for earthquakes with a magnitude of 7.5. For magnitudes other than 7.5 the 1996 NCEER workshop (Youd et al., 1997) recommended a range of factors that can be represented by,

$$MSF = \left(\frac{M_w}{7.5}\right)^{-2.56}, \tag{10}$$

where Mw is the moment magnitude ($M_w = 7.5$).

MSF is equal to 1.0. Both K_{a1} and K_{a2} factors are equal to 1.0 for uncemented soils of Holocene age. If the soil conditions are unknown and penetration data is not available, the assumed value for K_{a1} is 0.6 (Andrus et al., 2004).

To permit the CRR-V_{S1} curves for magnitude 7.5 earthquakes to have V_{S1} values between 195 and 210 m/s at CRR near 0.6, values of V^*_{S1} are assumed to range linearly from 200 to 215 m/s. The relationship between V^*_{S1} and fines content can S_1 be expressed by,

$$V^*_{S1} = 215,\ FC \le 5\%, \tag{11}$$
$$V^*_{S1} = 215–0.5(FC–5),\ 5\% < FC < 35\%, \tag{12}$$
$$V^*_{S1} = 200,\ FC \ge 35, \tag{13}$$

where FC is average fines content in percent by mass.

In both methods, if the effective overburden stress is greater than 100 Kpa at in question depth, CRR value is corrected using following equations,

$$CRR_j = CRR.K_\sigma, \tag{14}$$

$$K_\sigma = \left(\frac{\sigma'_V}{100}\right), \tag{15}$$

where K_σ is the overburden correction factor, σ'_V is the initial effective vertical (overburden) stress in kPa, 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. 7).

5. FACTOR OF SAFETY

A common way to quantify the hazard for liquefaction is in terms of a factor of safety. The factor of safety against liquefaction can be defined by,

$$F_S = \frac{CRR_J}{CSR}. \tag{16}$$

Liquefaction is predicted to occur when $F_S \le 1$, and liquefaction is predicted not to occur when $F_S > 1$. The acceptable value of F_S for a particular site will depend on several factors, including the acceptable level of risk for the project. potential for ground deformation, the extent and accuracy of seismic measurements, the availability of other site information, and the conservatism in determining the design earthquake magnitude and the expected value of a_{max}.

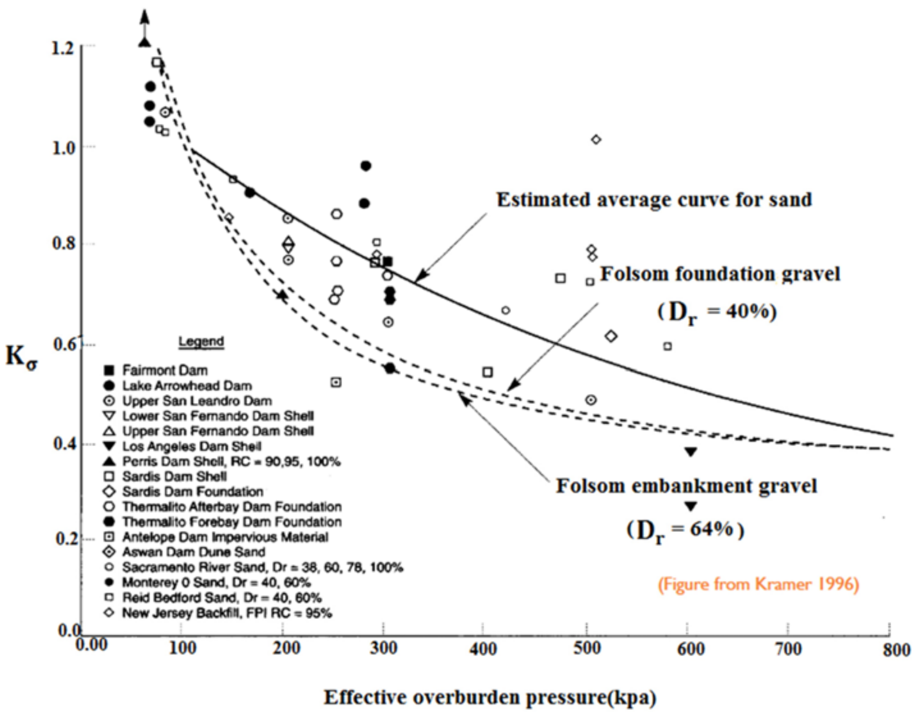

Fig. 7. Variations of Kσ values versus effective overburden stress (Hynes, 1999).

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5.1. Liquefaction Potential Index (P_L)

Iwasaki et al. (1982) quantified the severity of possible liquefaction at any site by introducing a factor called the liquefaction potential index (P_L) defined as,

$$P_L = \int_0^{20} F(Z).W(Z).dz \, , \tag{17}$$

$$F(Z) = 1 - F_L \, , \tag{18}$$

$$W(Z) = 10 - 0.5Z \, , \tag{19}$$

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 this study P_L values were measured and then compared for both methods.

To prove and verification of the applied method in this study, a comprehensive comparison between the liquefaction resistance factors, safety factor, shear modulus reduction curve and damping ratio curves were performed for the idealized soil profile, and the resulting liquefaction potential, for this area was determined and compared with known procedures as indicated in Figure 8 and contour map and 3D view of safety factor are given in Figure 9 respectively.

6. EVALUATING THE RESULTS OF DATA ANALYSIS

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

(1) Liquefaction potential index (PL) values based on SPT method is observed in Table 4. Results show that 51% of the data according to Table 3, ranking 2 have low liquefaction risk.

(2) PL values based on shear wave velocity (V_S) using five empirical relationships (Eqs. 1–5) in two un-cemented and cemented soils are seen in Tables 5 and 6. The results show that the used relations are overestimated and most of them have shown non-liquefaction condition for soil in the studied area.

(3) In 13 boreholes, about 229 soil layers analyzed and liquefaction potential of soils calculated the results of which for all types of soils are presented in Table 7. The results 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 non-liquefaction condition for soils.

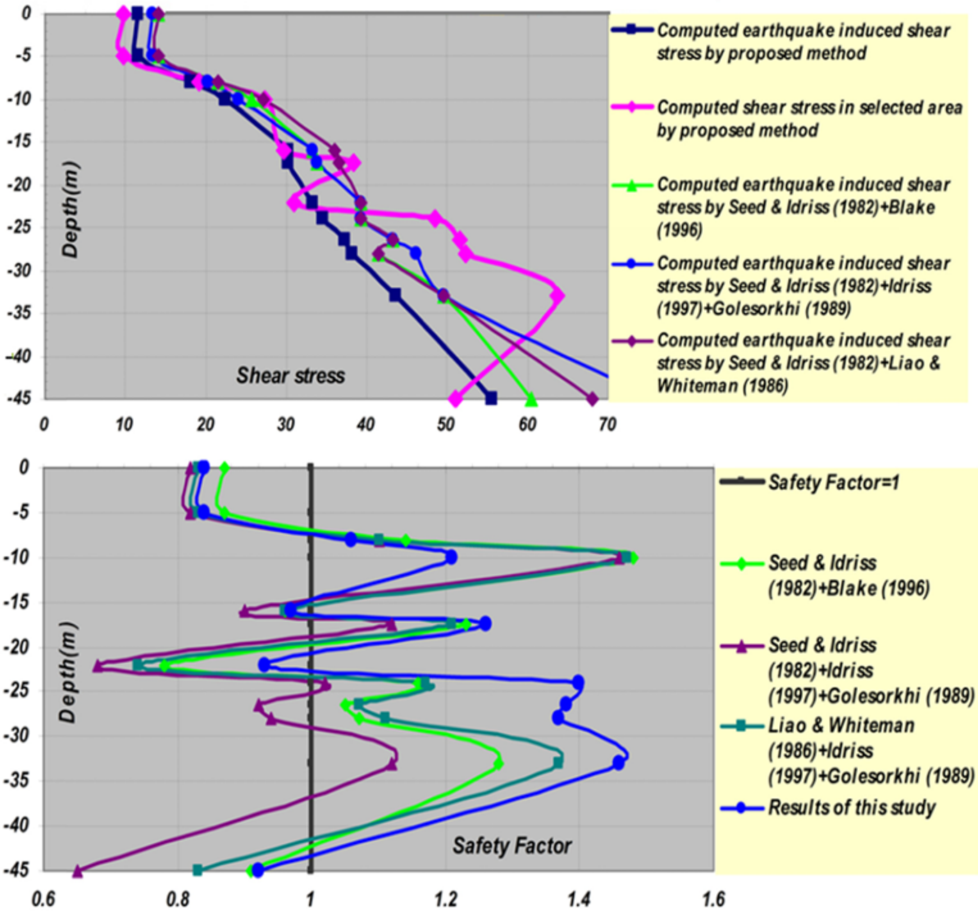

Fig. 8. Computed shear stress and comparison of safety factors between the known procedures and proposed in this study.

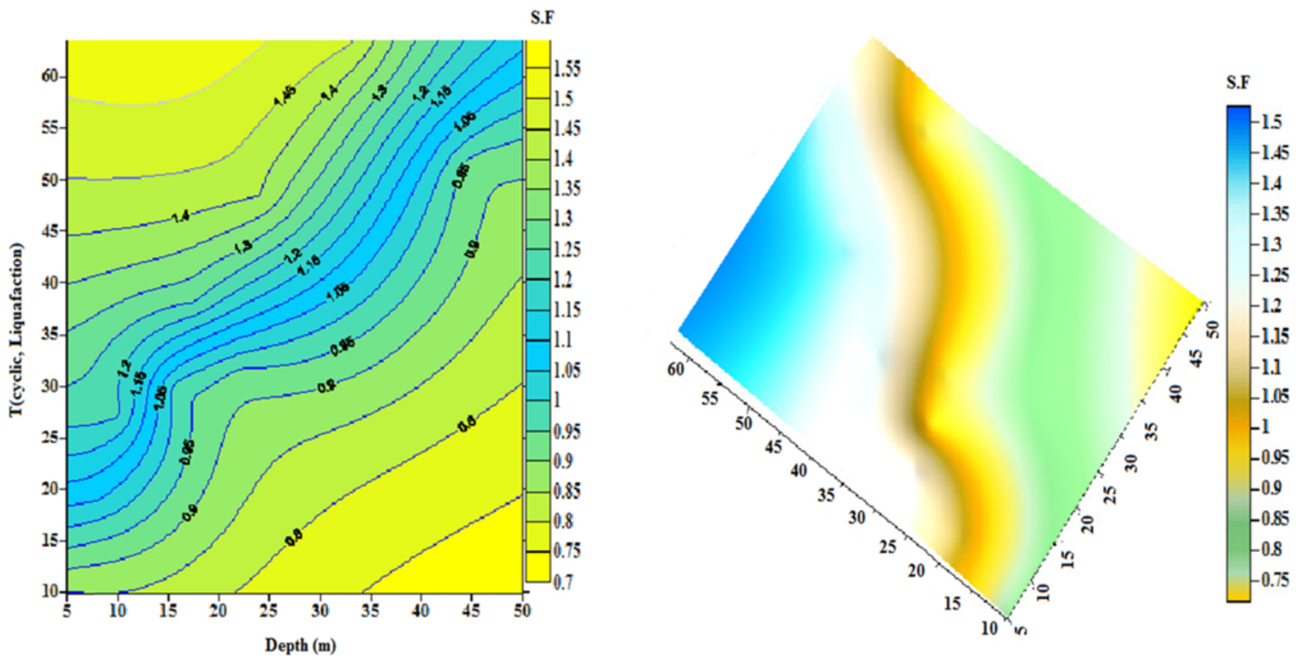

Fig. 9. Contour map and 3D view of the integration of data obtained for selected area. Horizontal axis (x) indicates the depth in terms of m and the vertical axis (y) indicates the cyclic shear stress required to cause liquefaction (Tcyc, L = CSR L. σ'V0, where σ'V0 is the initial vertical effective stress, CSR L is the cyclic stress ratio). The vertical column indicates the scale safety factor.

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

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

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

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

Table 5. The liquefaction potential index (PL) values based on the V_S analysis in the cemente soils

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

Table 6. Liquefaction potential index (PL) values based on the V_S analysis in the uncemented soils

PL-Value	PL = 0	0 < PL ≤ 5	5 < PL ≤ 15	PL > 15
		Equation (1)		
Number	66	1	0	0
Percent	98.5	1.5	0	0
		Equation (2)		
Number	65	2	0	0
Percent	97	3	0	0
		Equation (3)		
Number	66	1	0	0
Percent	98.5	1.5	0	0
			Equation (4)	
Number	66	1	0	0
Percent	89.5	1.5	0	0
		Equation (5)		
Number	67	0	0	0
Percent	100	0	0	0

(4) The comparative diagrams related to the liquefaction potential index (PL) values based on SPT and shear wave velocity methods in un-cemented and cemented states for soils are presented in Figures 10 and 11. As seen, the results are consistent

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

	SPT			V_S 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	1
Gravel	16	0	0	0	0	0
	SPT			V_S 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	110	128	128	129	129	125
Sand	123	172	169	171	169	171
Gravel	59	75	75	73	73	75
				Un-cemented		
Silt	113	152	157	157	157	157
Sand	135	163	163	164	164	164
Gravel	59	75	75	75	75	75

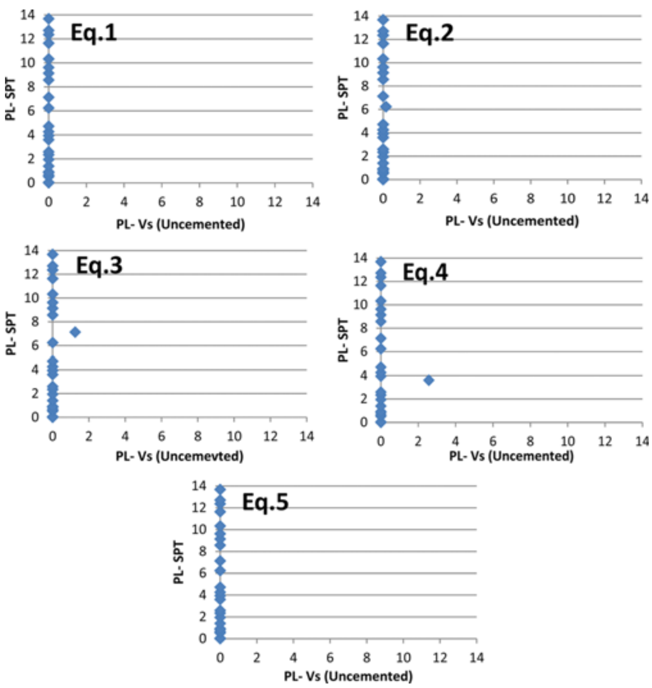

Fig. 10. The comparison of PL values for the deep layers of soil in the un-cemented state based on the SPT and the V_S.

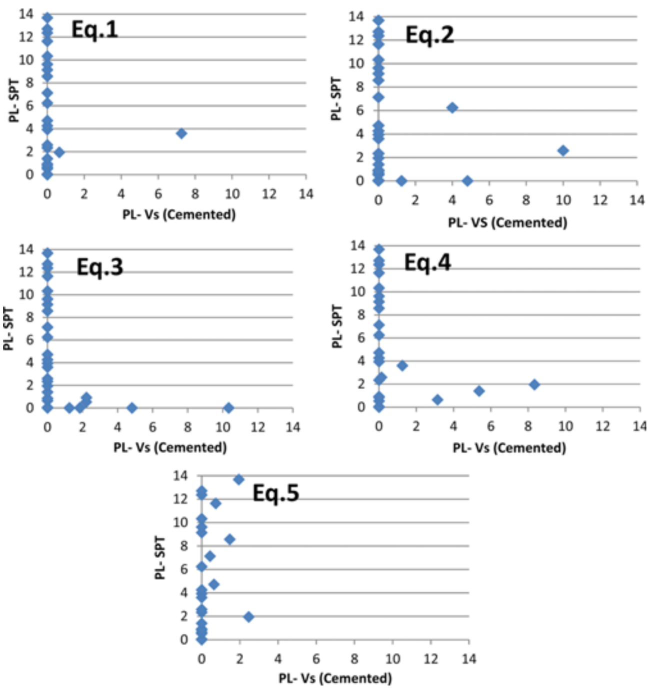

Fig. 11. The comparison of PL values for the deep layers of soil in the cemented sate based on the SPT and the V_S.

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.

(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 8. As illustrated below, there is proper/perfect adaption in the non-liquefaction of soil condition.

As it can be observed, there is a significant difference between Seed et al. (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 (V_S). 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:

Table 8. The comparison of analyses in layers at the same depth based on SPT and V_S methods using five empirical relationships

Type of Soil	State of soil	Liquefied in SPT and V_S - Eq.1	Liquefied in SPT and V_S - Eq.2	Liquefied in SPT and V_S - Eq.3	Liquefied in SPT and V_S - Eq.4	Liquefied in SPT and V_S - Eq.5
Silt	Cemented	1	0	1	0	4
	Uncemented	1	1	0	0	0
Sand	Cemented	2	2	1	0	3
	Uncemented	0	0	2	0	0
Gravel	Cemented	0	0	2	2	0
	Uncemented	0	0	0	0	0
Type of Soil	State of soil	Non Liquefied in SPT and V_S - Eq.1	Non Liquefied in SPT and V_S - Eq.2	Non Liquefied in SPT and V_S - Eq.3	Non Liquefied in SPT and V_S - Eq.4	Non Liquefied in SPT and V_S - Eq.5
Silt	Cemented	104	105	104	105	101
	Uncemented	104	104	105	105	105
Sand	Cemented	122	122	123	124	121
	Uncemented	124	123	123	123	124
Gravel	Cemented	58	58	56	58	58
	Uncemented	58	58	56	58	58

(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 et al. (1971). 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. (2004) 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 are inappropriate for the data range and the type of soils in the study area.

7. 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 (V_S) 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 V_S 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.

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