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Considering of Soil Liquefaction Hazard at the Downstream Area of Sattarkhan Dam Using Shear Wave Velocity during Ahar-Varzeghan Earthquake 2012

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Abstract

One of the important problems in earthquake geotechnical engineering is liquefaction phenomenon that happens in loose saturated granular soils. In this study, 11 boreholes from Govanjik and dayalar area in downstream of Sattarkhan Dam collected and evaluated. Results of research are showed that with assuming cementation in soils, liquefaction potential is weak to moderate than non-cemented condition. Also, value of LPI based on Vs (both of cementation and non-cementation status) is more than SPT.

Keywords: Liquefaction; Sattarkhanb Dam; Govanjik village; Dayalar village; SPT; Shear Wave Velocity.

1- Introduction

Liquefaction in soil due to earthquake is one of the important happens and cause of sever damages on structures and lifelines. The pore water pressure during earthquake in loose saturated granular soils (in special condition clayey soil) increases and in continue soil tend to reduce volume and confine stress decreases (Seed and Idriss- 1971). Finally, shear strength in soil is about equal to zero and in this state liquefaction has happened (Seed and Idriss- 1971).. This phenomenon occurs such as extended ground settlements, sand boiling and water seepage on ground. Several factors can be affected on occurrence of liquefaction such as earthquake magnitude and duration, void ratio, relative density, fines content, plasticity index and etc (Seed and Idriss-1971). Liquefaction resistance of soils can be evaluated with using laboratory tests such as cyclic simple shear test, cyclic triaxial tests and cyclic torsional test or field tests e.g. standard penetration test (SPT) (Idriss and Boulanger-2006),, cone penetration test (CPT) (Robertson and Wride-1998) and shear wave velocity (Vs) (Andrus et al., 2004a, 2004b). Main aim in this study, evaluation of soils liquefaction potential in the downstream deposits of Sattarkhan Dam near to Ahar city by Vs . Furthermore, comparison between Vs and standard penetration methods carried out. In final, liquefaction potential index (LPI) assessed. In this research, the results of liquefaction potential analyses in soil layers at downstream area of Sattarkhan Dam based on both SPT and Vs methods are compared. Idriss and Boulanger (2006) procedure is used in SPT method and Andrus and et al. (2004a, 2004b) process is carried out for Vs measurement. Finally, Liquefaction potential index (LPI) evaluated for both of them by Iwasaki et al. (1978, 1982) method.

2- Geology and Ahar-Varzeghan earthquake 2012

The Ahar-Varzeghan area is underlain by a wide variety of sedimentary and volcanic rocks and unconsolidated sedimentary deposits, most of wich range in age from Cretaceous through

Quaternary. The rocks vary greatly in composition, degree of consolidation and depth of weathering. Marl, sandstone and volcanic rocks predominate. . Based on geological map of Ahar and Tabriz-Poldasht (Fig. 1) the main formations of the area include: Quaternary deposits including terraces and alluviums; Pliocene conglomerate and siltstone; Oligocene dasitic breccias; Miocene gypsiferous and sily marl, siltstone and sandstone and Cretaceous marl, molasses and sandstone. During Ahar-Varzeghan 2012 earthquake two lateral spread, sinkhole and liquefaction phenomena occurred (Figs 2, 3 and 4).

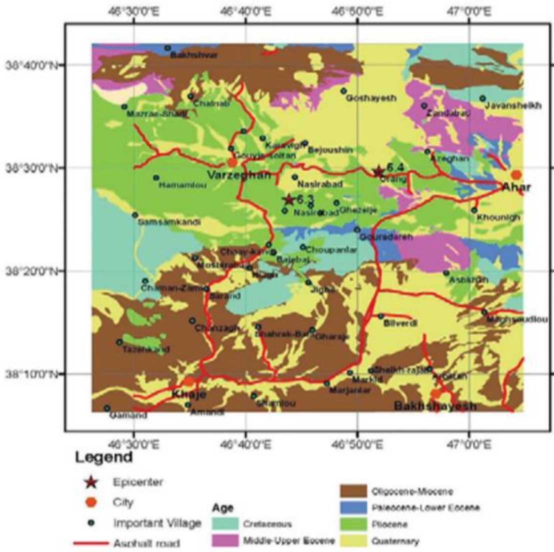

Figure 1) Geology map of study area (from Memarian and Mahdavifar, 2012).

Figure 2) The lateral spreading in Nosham plain near Gamand village with 0.5 meter settlement of the river bank (from Memarian and Mahdavifar, 2012).

Figure 3) Liquefaction zone in the vicinity of Marjanlar village (Memarian and Mahdavifar, 2012).

Figure 4) Sinkhole with ten meters width in the vicinity of Marjanlar village (Memarian and Mahdavifar, 2012).

In this research, 11 boreholes log in downstream of Sattarkhan dam area is collected and evaluated and position of Dam and study area is shown in Figure 5. Boreholes have been drilled rotary. The geotechnical properties of the studied soil layer are composed of four parts (East Azerbaijan water authority, 2009). In part one, soil layers type near ground surface is gravel with sand and silty clay. Relative density in first part between moderate to dense is variable. In part two, type of soil layers are generally silty sand with gravel and relative density moderate to very dense change. In third part type of soil layers mostly fines content silty clay and clayey sand. Finally, in part four types of soils include granular particles such as gravel and cobble stone and in terms of relative density is very dense. Ground water table level is one of the main factors in soil liquefaction potential evaluation. According to observations and piezometric information, the ground water's table depth changes between 2 and 4 meters.

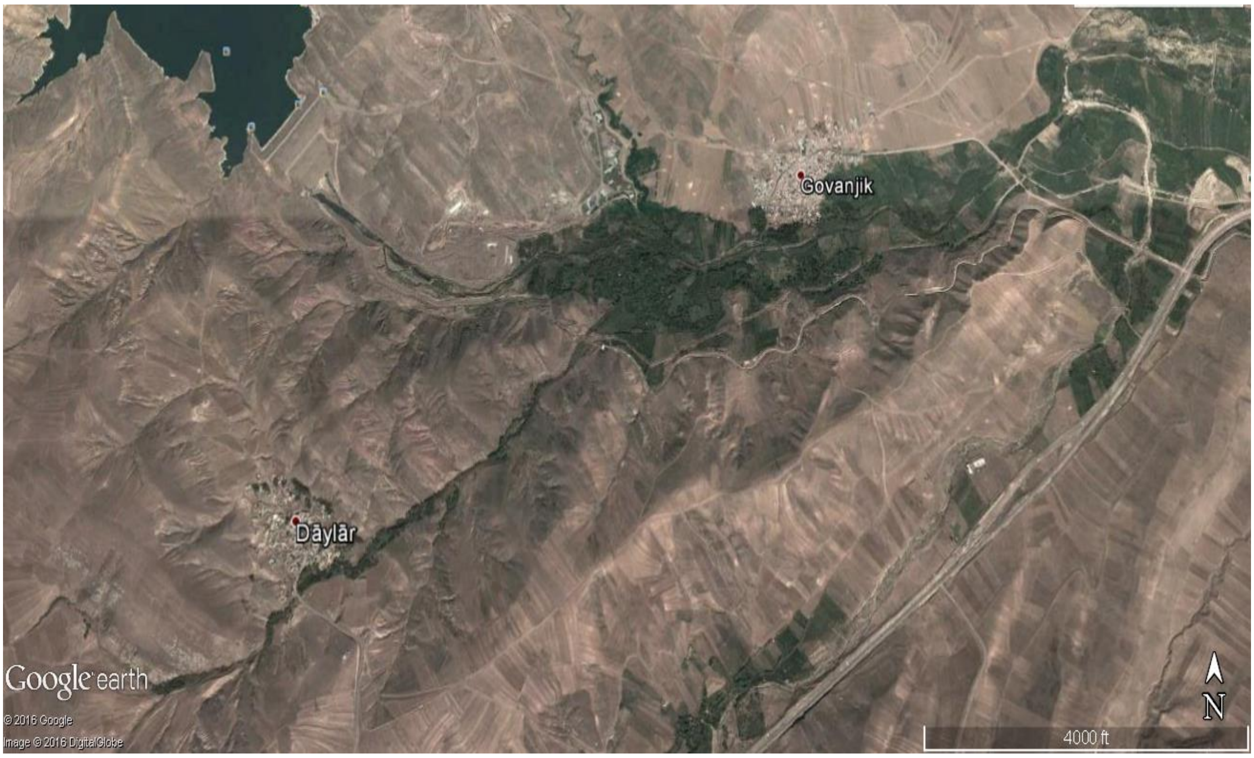

Figure 5) Position of Sattarkhan dam and study area in this research (www.earth.google.com/2016)

3- Analysis of boreholes log for evaluating the liquefaction potential

The peak ground acceleration (PGA) is necessary for the analysis of boreholes to evaluate liquefaction potential of soils (Seed-1971). The PGA value due to Ahar- Varzeghan 2012 earthquake is about equal 0.25g (Memarian and Mahdavifar-2012). Therefore this value is selected for PGA. Also, as reported by IRSC magnitude (Mw) is equal to 6.2 considered (Fig. 6). In shear wave velocity measurement method based on Andrus et al. (2004a, 2004b) process for assessing liquefaction potential, according to type of soil layers and geology condition in study area five empirical equations between Vs and SPT blow count (N) were selected then Vs values calculated. Relations have been mentioned in Table 1.

Table 1) Empirical relations between Vs and N_{SPT} used in this research.

$V_s = 61N^{0.5}$	(Seed and Idriss, 1971)	Eq.1
$V_s = 97N^{0.314}$	(Imai and tonouchi, 1982)	Eq.2
$V_s = 76N^{0.33}$	(Imai and yoshimur,1970)	Eq.3
$V_s = 121N^{0.27}$	(Yokota *et al.*, 1991)	Eq.4
$V_s = 22N^{0.85}$	(Jafari *et al.*, 1997)	Eq.5

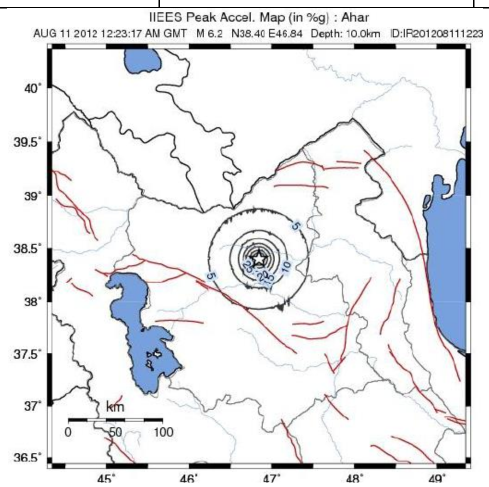

Figure 6) Variation of PGA in the Ahar- Varzeghan earthquake 2012 (Memarian and Mahdavifar-2012).

3.1- Evaluation of Liquefaction Potential based on Standard Penetration Test method (SPT)

In the assessment of the liquefaction potential of the soils in the study area, the simplified method by Idris and Bolanger (2006) is used. At 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 estimated using the equation bellow:

$$CSR_{7.5} = 0.65\frac{a_{max}}{g}.\frac{\sigma_V}{\sigma_V^{'}}.r_d.\frac{1}{MSF} \qquad (6)$$

In the above equation, amax is the peak ground acceleration, g is acceleration of gravity, σV total stress in the depth in the question, σ΄V effective stress in the same depth, rd 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 (1997) researches using equation 2. In this equation Mw parameter is equal magnitude of earthquake:

$$MSF = \left(\frac{M_W}{7.5}\right)^{-3.3} \qquad (7)$$

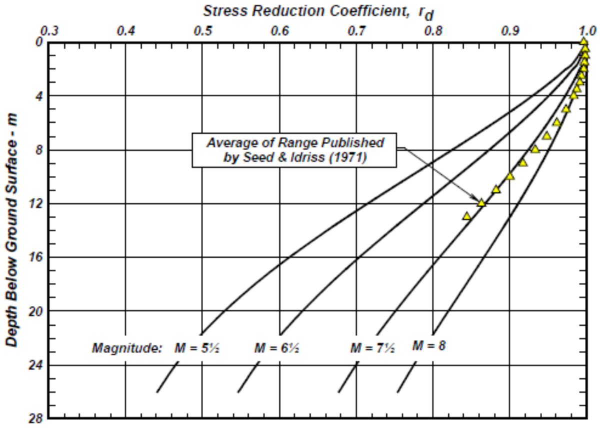

Figure 7) Diagrams for evaluation Stress reduction coefficient (rd) (Idriss-1999).

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

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

Factor	Equipment Variable	Term	Correction
Overburden Pressure		C_N	$C_N = \left(\frac{Pa}{\sigma'_V}\right)^{0.5} \leq 1.7$ Pa=100kPa
Energy ratio	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	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

$$(N_1)_{60} = N_{SPT} \times C_N \times C_E \times C_B \times C_R \times C_S \qquad (8)$$

In this equation, N_{SPT}, the number of standard penetration resistance test, C_N coefficient of the over burden stress, C_E the coefficient of the hammer energy, C_S the coefficient of the sampling method, C_B the coefficient of the bore hole diameter, C_R the coefficient of the rod length and $(N_1)_{60}$ is the modified number of the standard penetration test. After that, according to the presented proposal by Idriss and Boulanger (2006), the overburden tension correction factor (C_N) is determined using the following equation:

$$C_N = \left(\frac{P_a}{\sigma_V'}\right)^{\alpha} \leq 1.7, P_a = 100kPa \quad (9)$$

$$\alpha = 0.784 - 0.0768\sqrt{(N_1)_{60}} \quad (10)$$

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

$$(N_1)_{60CS} = (N_1)_{60} + \Delta(N_1)_{60} \quad (11)$$

$$\Delta(N_1)_{60} = 1.63 + \exp\left(1 + \frac{9.7}{FC + 0.1}\right) - \left(\frac{15.7}{FC + 0.1}\right)^2 \quad (12)$$

$$CRR = \exp\left(\left(\frac{(N_1)_{60CS}}{14.1}\right) + \left(\frac{(N_1)_{60CS}}{126}\right)^2 - \left(\frac{(N_1)_{60CS}}{23.6}\right)^3 + \left(\frac{(N_1)_{60CS}}{25.4}\right)^4 - 2.8\right) \quad (13)$$

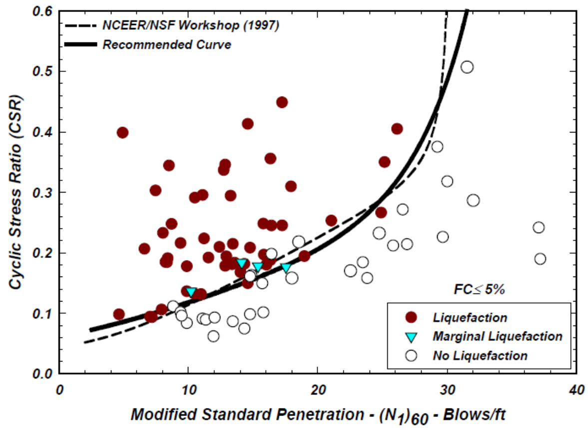

Figure 8) Diagrams for evaluation cyclic resistance ratio (CRR) based on modified Standard Penetration Test Results (Mw=7.5) (Idriss and Boulanger, 2006)

3.2- Evaluation of Liquefaction Potential based on Shear Wave Velocity method (Vs)

Assessment of soils liquefaction potential based on shear wave velocity (Vs) is new procedure in comparison with SPT number. In this context can be pointed to researches Askari et al. (2003, 2006, and 2007), Shafiee et al. (2008 and 2009) and Askari et al. (2011). In this study Andrus *et al*. (2004a, 2004b) method was used. In the procedure shear wave velocity should be corrected to overburden stress. Equation 14 is suggested that:

$$V_{S1} = V_S\left(\frac{P_a}{\sigma_V'}\right)^{0.25} \cdot \left(\frac{0.5}{K_0'}\right)^{0.125} \quad (14)$$

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). 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}. The CRR value can be defined by equation 15 (Andrus *et al*., 2004a, 2004b):

$$CRR = K_{a2}\{0.022\left(\frac{K_{a1}V_{s1}}{100}\right)^2 + 2.8\left(\frac{1}{V_{s1}^* - K_{a1}V_{s1}} - \frac{1}{V_{s1}^*}\right)\}MSF \quad (15)$$

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 et al. (2004a, 2004b) suggest the following relationships for estimating MSF and V_{s1}^*:

$$V_{s1}^* = 215 \qquad FC \leq 5\% \text{ (FC=Fines content)} \quad (16a)$$

$$V_{s1}^* = 215 - 0.5(FC - 5) \qquad 5 \prec FC \prec 35\% \quad (16b)$$

$$V_{s1}^* = 200 \qquad FC \geq 35\% \quad (16c)$$

Both Ka_1 and Ka_2 factors are equal to 1.0 for uncemented soils of Holocene age. For the older and cemented soils, Ka_1 factor is evaluated based on estimated and measured values of shear wave velocity in study area (Andrus et al., 2004b). If the soil conditions are unknown and penetration data is not available, the assumed value for Ka_1 is equal 0.6. Ka_2 value determined based on geological age that proposed in Table 3.

Table 3) Value of Ka2 based on geological age (Andrus et al., 2004b)

year	K_{a2}
< 10000	1
10000	1.1
100000	1.3
1000000	1.5

3.3- Correction cyclic resistance ratio

In both methods, if the effective overburden stress is greater than 100kPa at in question depth, CRR value is corrected using following equations (Hynes and Olsen, 1998):

$$CRR_J = CRR.K_\sigma \qquad (17)$$

$$K_\sigma = \left(\frac{\sigma'_V}{100}\right)^{f-1} \qquad (18)$$

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.

3.4- 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:

$$Fs = \frac{CRR_J}{CSR} \qquad (19)$$

Where CRR_J is corrected value of CRR estimated by equation 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.

3.5- Liquefaction Potential Index (LPI)

Iwasaki *et al*. (1978, 1982) quantified the severity of possible liquefaction at any site by introducing a factor called the liquefaction potential index (LPI) defined as:

$$LPI = \int_0^{20} (10 - 0.5Z).(1 - F_S)\,dz \qquad (20)$$

Where Z is the depth in question. The ranges of LPI vary from 0 to 100 according to Table 4. In this study LPI values were measured and then compared for both methods.

Table 4) Liquefaction potential index (LPI) and its describes (Iwasaki et al., 1978)

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

4- Results of Analyses

Results of liquefaction potential of 57 soil layers in 11 bore holes in study area based on both SPT and Vs (with assuming cementation and un cementation in soil) can be stated below:

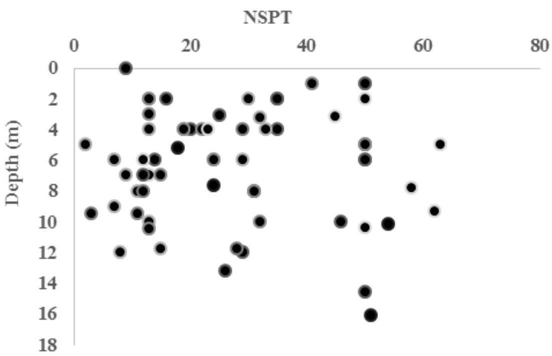

Figure 9) Variations of NSPT versus depth in study area.

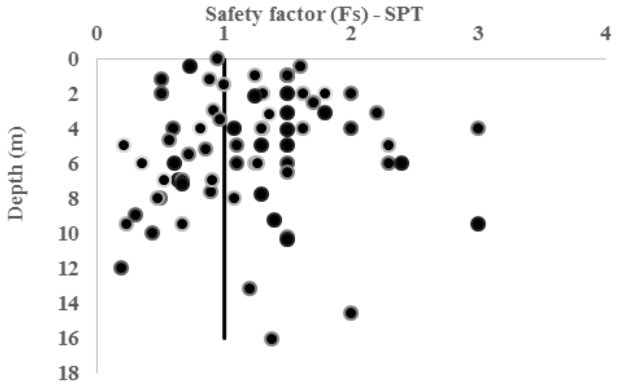

Figure 10) Variations of safety factor in soil layers based on Idriss-Boulanger method.

1- Variations of NSPT values in study area Can be found that about 70% of SPT values less than 40 (Fig. 9). Liquefaction analysis based on SPT showed safety factor in 35% of layers less than 1 (Fig. 10).

2- Vs values based on five empirical relationship mentioned above in 11 boreholes in study area can be found in Figure 11. According to diagrams of Vs values are between 200 and 500 m/s.

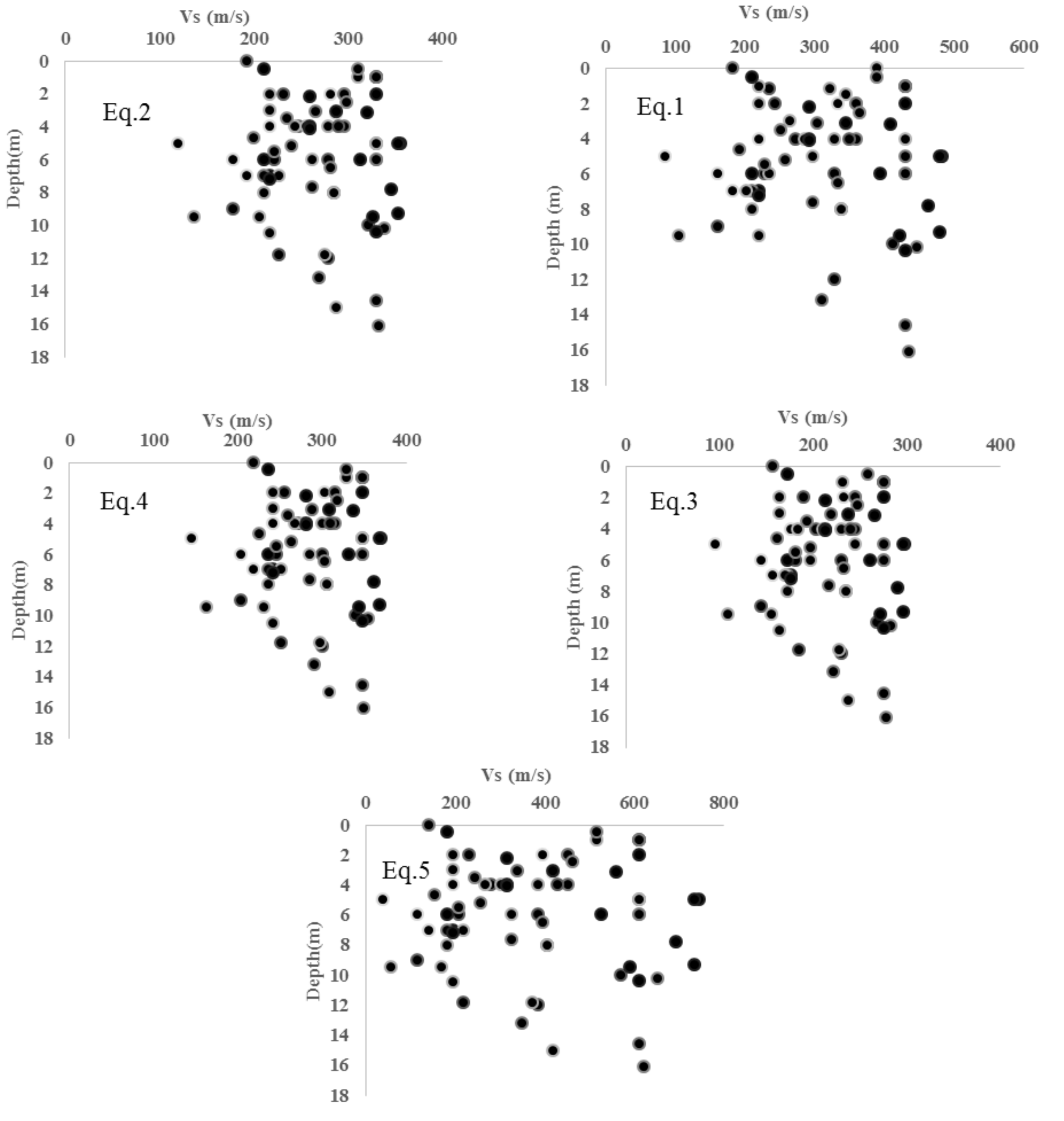

Figure 11) Variations of shear wave velocity in soil layers based five empirical relationships.

3- Results of liquefaction potential analysis based on Andrus et al. (2004a, 2004b) method with using shear wave velocity (with assuming cemented and uncemented in soil) in 11 boreholes and 57 soils layer are shown in Figures 12 and 13. According to diagrams can be found that by assuming cementation condition in soil layers liquefaction potential of soils increase and empirical equation No. 2 has proposed the most hazard of liquefaction in comparison with other equations. Also, generally can be mentioned with assuming cementation condition in soil layers liquefaction hazard in study area is more than uncementation situation.

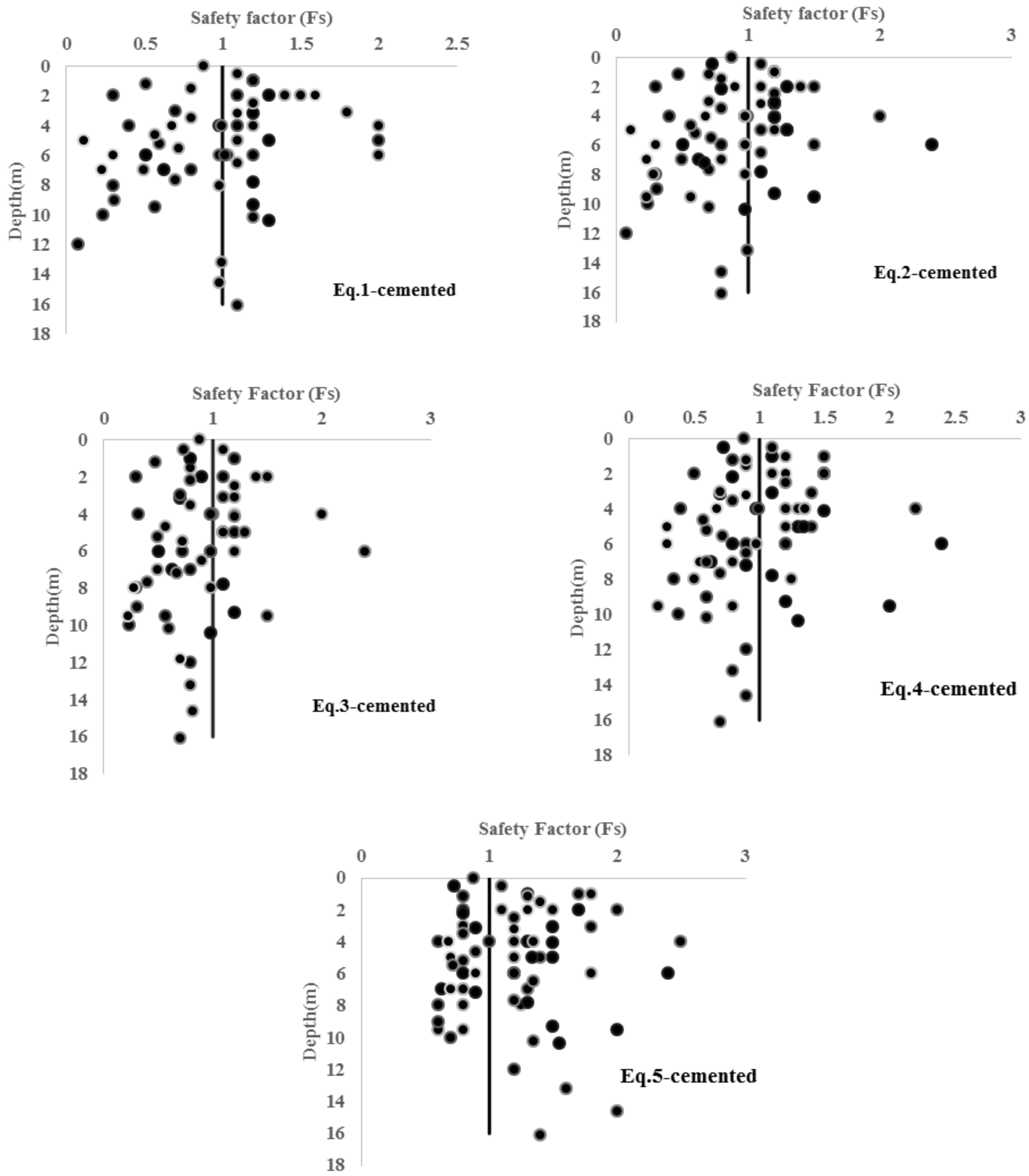

Figure 12) Variations of safety factor in soil layers based on Andrus and Stokoe (1997) method (with assuming cementation condition)

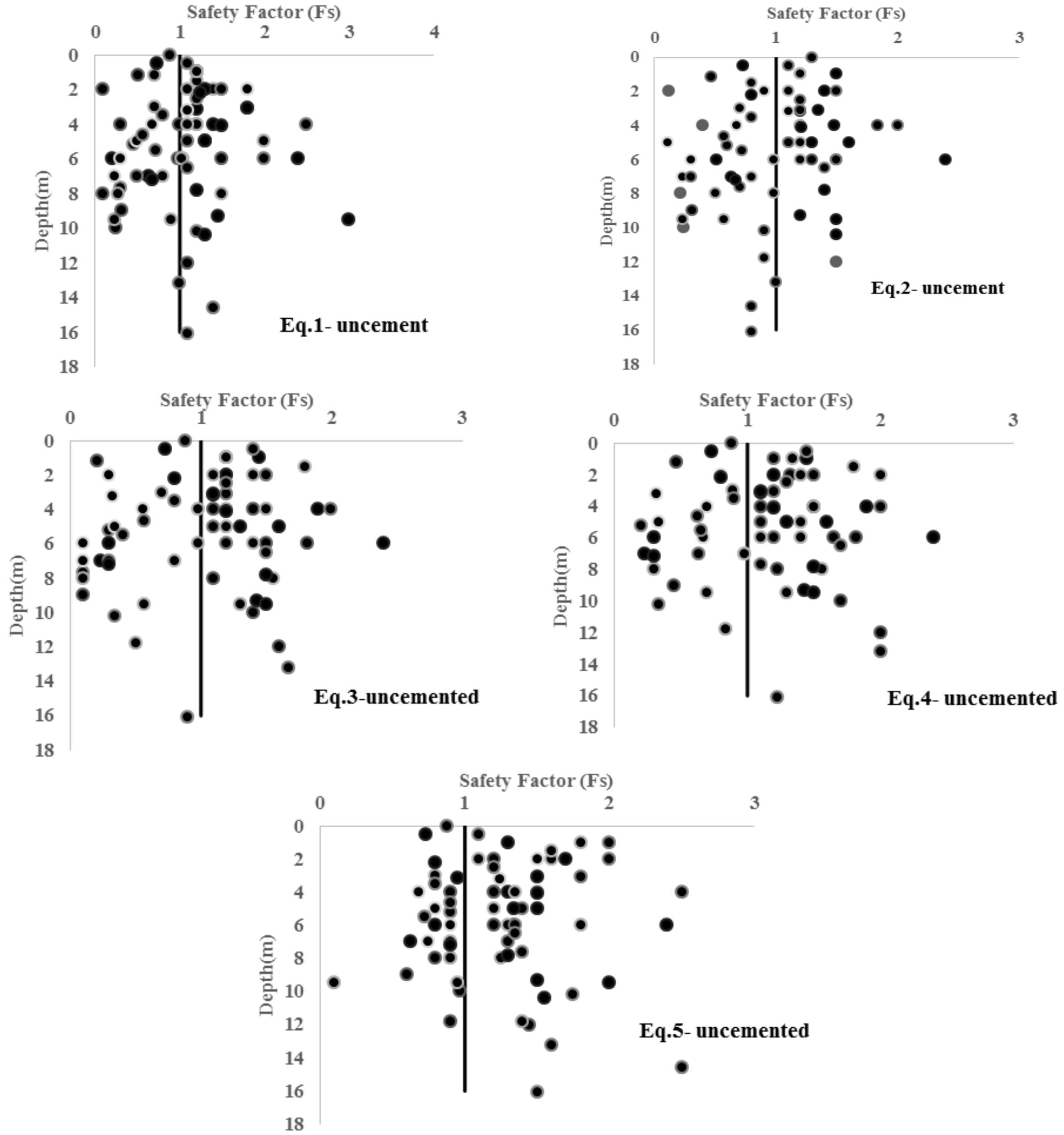

Figure 13) Variations of safety factor in soil layers based on Andrus and Stokoe(1997) method (with assuming uncementation condition).

4- LPI values for both methods determined and variations have proposed in Figures 14 (a, b). According to diagrams can be observed in Vs method that with assuming cementation condition and uncementation situation in soils LPI value estimated respectively from equation No. 2 and equation No. 3 are more than LPI estimated form NSPT procedure.

5- For evaluation rate of adaption between two applied methods LPI values for each of empirical equations are comprised. According to Figures 15 and 16 (respectively in cemented and uncemented) can be seen first, between two methods adaption is unsuitable. Secondly, as mentioned previous paragraph empirical equations No. 2 and No. 3 in Vs method propose the most LPI than SPT.

6- Results of liquefaction potential evaluation of soil layers (respectively cemented and uncemented status) in depths by Vs methods were compared. According to Tables 5 and 6 can be seen firstly, adaption between two assumptions is inadequate and secondly empirical equations No. 2 (in cemented) and No. 3 (uncemented) most soil layers the risk of liquefaction evaluated.

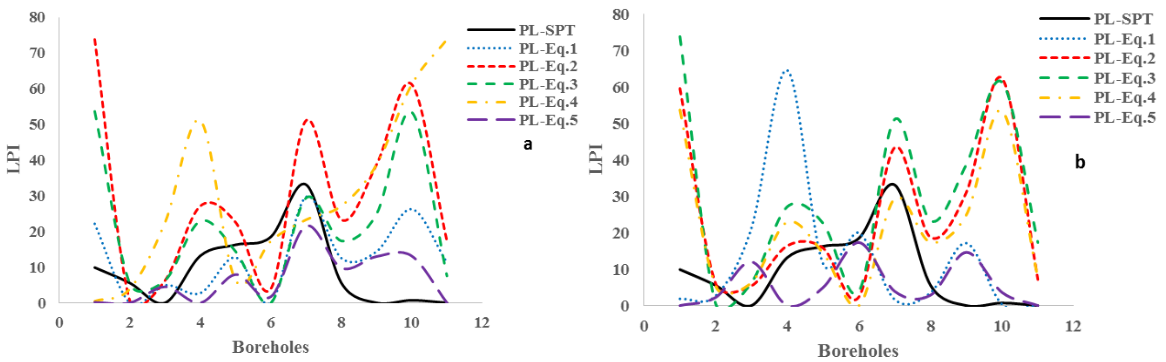

Figure 14) Comparison of LPI based on SPT and Vs methods (a- cemented soil condition, b-uncemented soil condition).

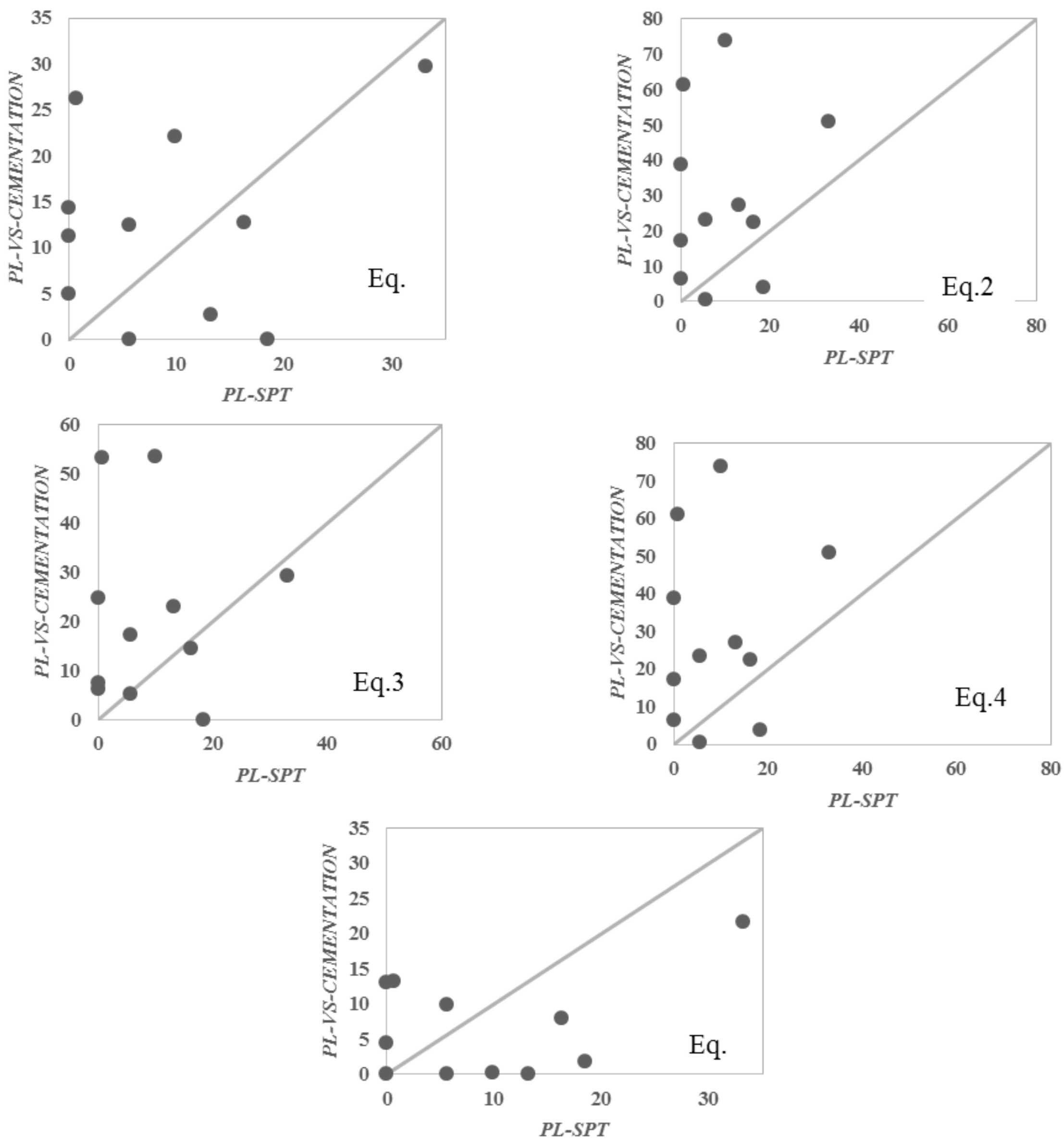

Figure 15) Comparison of LPI values adaption based on SPT and Vs methods (cemented soil condition condition).

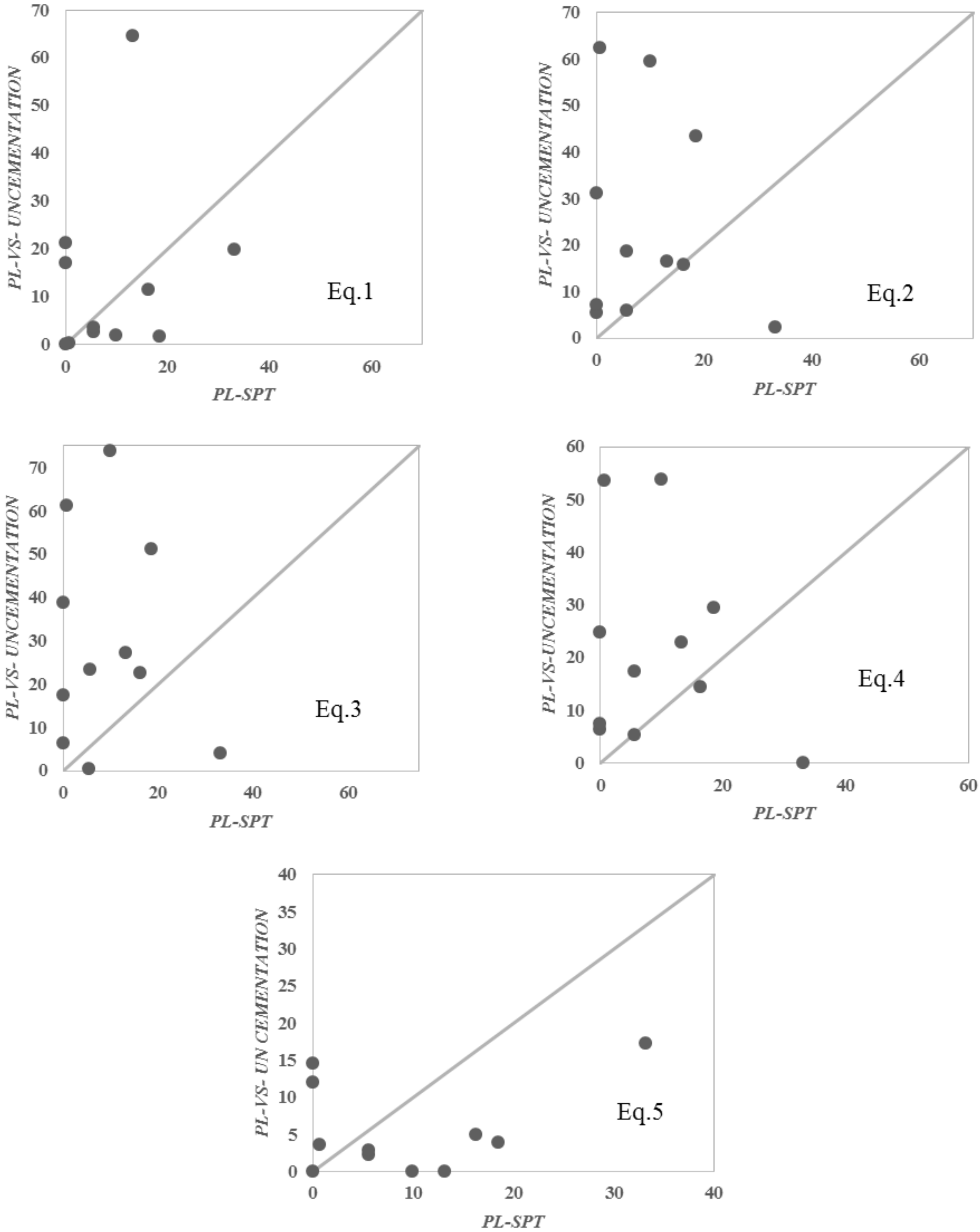

Figure 16) Comparison of LPI values adaption based on SPT and Vs methods (uncemented soil condition).

Table 5) Comparison of soil liquefaction potential results in depths based on Vs method (in cemented).

Used empirical equations	Total of liquefied layers in 11 boreholes	Total of non-liquefied layers in 11 boreholes
Eq.1	26	31
Eq.2	43	16
Eq.3	41	14
Eq.4	41	16
Eq.5	16	41

Table 6) Comparison of soil liquefaction potential results in depths based on Vs method (in uncemented)

Used empirical equations	Total of liquefied layers in 11 boreholes	Total of non-liquefied layers in 11 boreholes
Eq.1	19	38
Eq.2	30	27
Eq.3	41	16
Eq.4	31	26
Eq.5	12	45

5- Conclusion and discussion

The main aim of this study is Ahar-Varzeghan 2012 earthquake effects on liquefaction potential of soils in downstream of Sattarkhan dam based on shear wave velocity method. In study area (Govanjik and Dayalar villages) 11 boreholes logs collected and evaluated. As regards available data was based on SPT therefore five empirical equations (between Vs and SPT) adapt to soil layers condition selected. In general results showed that liquefaction hazards with considering ground water table and peak ground acceleration due to Ahar-Varzeghan earthquake could exist. Although, liquefaction hazard based on shear wave velocity method in comparison with N_{SPT} method is more. Reasons of incompatibility between results of two methods are as follow:

1- In determining the cyclic strength ratio (CRR) in Vs method the soil cementation factors (K_{a1} and K_{a2}) are calculated. The value of these parameters proposed by Andrus and Stokoe (1997) may be inappropriate for study area.

2- The maximum shear wave velocity (V_{s1}^{*}) values for occurring liquefaction in soil recommended by Andrus *et al.* (2004a) may be unsuitable for the study area.

3- The value of *a* and *b* parameters in CRR equation in the Vs method perhaps is improper for the data range studies.

4- The assumption that CRR_{field} is equal to 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.

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