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Study of Soil Liquefaction Potential in the South of Urmia plain, reviewing the Japanese specifications for highway bridges 1999

Ali Alizadeh[1], Tohid Malekzadeh Dilmaghani*[1]

[1]*Department of Physics, Faculty of Basic Science, Ahar Branch, Islamic Azad University, Ahar, Iran*

ARTICLE INFORMATION

ABSTRACT

Liquefaction is one of the main topics of seismic geotechnics. The effects of liquefaction on structures and installations during an earthquake can be very destructive. In the two earthquakes Alaska and Niigata Japan in 1964, spectacular examples of earthquake-induced failures, such as rupture of slopes, deformation of foundation in buildings and bridges, and the floating of buried structures a result of the flow of soil bed occurred. In liquefaction in general, soils tend to be denser when non-adherent, saturated and loose soils are exposed to earthquake vibrations. However, in certain granulation ranges, drainage is somewhat slow, as the rapid fluctuations in the soil due to lack of drainage will necessarily increase the pore pressure. Increasing pore water pressure results in a reduction in effective stress, which results in reduced shear resistance of the soil. Based on the relationship between the geostatic stresses in the soil, the increase in pore pressure may reduce the effective stress in the soil, which results in low or even zero shear strength and, consequently, Soil flows into fluid state, which is called liquefaction. In this research, we will study this phenomenon in the East of Urmia plain using the Japanese specifications for highway bridges 1999. The basis of this method is the evaluation of the liquefaction potential, by comparing the liquefaction resistance assessed from the soil SPT with the shear stress ratio induced by the earthquake to the normal effective stress from the earthquake. Of the 6 study boreholes, only one of the boreholes with high-liquefaction risk was evaluated. So the risk of liquefaction is very low. In general, the southern part of Urmia region requires more geotechnical studies due to the density of existing industries and roads, and also geophysical studies such as the use of shear waves.

1. Introduction

Liquefaction is one of the main topics of seismic geotechnics. The effects of liquefaction on structures and installations during an earthquake can be very destructive (Canaslan Comut, 2016). In the two earthquakes Alaska and Niigata Japan in 1964, spectacular examples of earthquake-induced failures, such as rupture of slopes, deformation of foundation in buildings and bridges, and the floating of buried structures a result of the flow of soil bed occurred (Motagh et al 2017). Soils tend to be denser when non-adherent, saturated and loose soils are exposed to earthquake vibrations. However, in certain granulation ranges, drainage is somewhat slow, as the rapid fluctuations in the soil due to lack of drainage will necessarily increase the pore pressure (Das, 2010). Increasing pore water pressure results in a reduction in

* Corresponding author.
E-mail address: t-malekzadeh@iau-ahar.ac.ir
Assistant Professor, Academic Staff.

effective stress, which results in reduced shear resistance of the soil. Based on the relationship between the geostatic stresses in the soil, the increase in pore pressure may reduce the effective stress in the soil, which results in low or even zero shear strength and, consequently, fluid state of the soils, which is called liquefaction (Alizadeh, 2011). Generally, liquefaction occurs in uniformly saturated sand deposits with a loose or moderate condensation state (Islami, 2007). Considering the importance of the southern regions of Urmia Plain where construction of major industrial and road projects occurs such as Zobahan, Petrochemical, Power Plant, National Railways and Human Population in this region and the development of civil engineering projects, this research has been performed for determination of the liquefaction ability of the studied area (Alizadeh, 2011). Fig. 1 is presents the location of studied area.

2. Material and Methods

Several methods have been introduced to evaluate the liquefaction potential. In this study, the Japanese specifications for highway bridges 1999 have been used. The basis of this method is the evaluation of the liquefaction potential, by comparing the liquefaction resistance assessed from soil SPT with the shear stress ratio induced by the earthquake to the normal effective stress from the earthquake. (Robert , 2002)

2.1. Properties of soils susceptible to liquefaction

The following procedures conducted to performing liquefaction on studied site:
- The hydrostatic level is greater than 10 meters,
- The desired layer has a depth of less than 20 meters,
- The soil size have to $0.02 < D_{50} \leq 2$ mm,
- The $PI \leq 15\%$ and $F_C \leq 35\%$ as Japanese specifications for highway bridges 1999.

According to new studies, sandy soils with a D_{50} of over 2 mm can also be liquefying (Noorzad, 2010).

2.2. Cyclic Stress Ratio (CSR)

With the CPR ratio presented in Eq. 1 and the CSR Cyclic stress ratio, we can obtain the confidence coefficient for the studied soil for an earthquake of 7.5 magnitudes. Given the fact that the magnitude of the Investigated plan earthquake may be above or below 7.5, the earthquake correction factor should be defined (Sonmez et al, 2005).

$$(CSR) = \frac{\sigma_{max}}{\sigma'_{vo}} = \frac{a_{max}}{g} \cdot \frac{\sigma_{vo}}{\sigma'_{vo}} \cdot rd \qquad (1)$$

The Cyclic stress ratio is equal to equation 1, in which is the maximum ground acceleration, is total stress, is effective stress, is gravity and is stress reduction coefficient, which is a function of the depth of the desired layer and its value is obtained from Eq. 2 (Japanese specifications for highway bridges, 1999).

$$rd = 1 - 0.015z \qquad (2)$$

2.3. Cyclic Resistance Ratio

From the accumulation of three factors of slag pressure, grain size and fineness, the cyclic shear value is obtained, which its equations are given as Eqs. 3 to 7.

$$R_L = CRR = R_1 + R_2 + R_3 \qquad (3)$$

$$R_3 = 0.0882 \sqrt{\frac{N_j}{\sigma'_v + 0.7}} \qquad (4)$$

$$R_2 = \begin{cases} 0.19 & 0.02mm \leq D_{50} < 0.05 \\ 0.225 \log \dfrac{0.35}{D_{50}} & 0.05mm \leq D_{50} < 0.6mm \\ -0.05 & 0.6mm \leq D_{50} < 2.0n \end{cases} \qquad (5)$$

$$R_3 = \begin{cases} 0.0 & 0\% \leq Fc < 40\% \\ 0.004FC - 0.16 & 40\% \leq Fc < 100\% \end{cases} \qquad (6)$$

$$N_j = 0.833(N_1)_{60} \qquad (7)$$

where D_{50} is mean grain size, Fc is grain percent, σ'_v Effective vertical stress at study depth (Kpa), and NJ: SPT number (Japanese specifications for highway bridges 1999).

$$F_L = \left(\frac{CRR_{7.5}}{CSR}\right) \cdot MSF \cdot K_\sigma \cdot K\alpha \qquad (8)$$

and

$$K_\sigma = \left(\frac{\sigma'\partial}{P_a}\right) f - 1 \qquad (9)$$

where MSF is the correction factor for earthquake magnitudes, K_σ the correction coefficient for the overhead stresses on the soil and K_α the correction coefficient of the initial shear stresses on the soil in the static state (Zhang, 2015). Also, σ' is effective overhead stress on the soil, P_a is atmospheric pressure; f is function power of relative density, tensile history, sediment age and pre-consolidation ratio. D_r for relative density of 40 to 60 percent of f (mainly between 0.8 and 0.7); and σ' is relative density (referring to 60%-80% for f value) which lies between 0.7 and 0.6. In this regard, it can be explain as:

$$\alpha = \frac{\tau st}{\sigma' vo} \qquad (10)$$

$$W(Z)d(Z)P_L = \int_0^Z F(Z) \qquad (11)$$

$$F(Z) = 1 - F_L \qquad (12)$$

$$W(Z) = 10 - 0.5Z \qquad (13)$$

where τ_{st} is static shear stress on the soil layers (the weight of the upper heavy structure or the soil slope, if any); σ'_{v0} is effective overhead stress; Z depth from the earth surface in meters, F(Z) is a function of the fluid resistance parameter FL, obtained from Eq. 8. (Maurer, 2015); F_L is the confidence coefficient of liquefaction occurrence, if $F_L>1$ then F(Z) is equal to zero. The P_L value is between zero and 100 (Sonmez et al., 2005)

3. Results and Discussions

Six boreholes have been excavated to study the soil properties and field experiments in the area, which are mostly focused on the Barandouzchay River Bridge (Alizadeh, 2011). To determine soil liquefaction potential, we first study the soil properties according to 1. In case of soil liquefaction conditions, we will investigate the CSR according to 2 and the cyclic resistance ratio (RL) using clause 3. Given the above values, in the case of obtaining values less than 1 for FL, the desired layer bears the liquefaction potential. The results of field experiments and soil characteristics for the BH1 borehole are presented in Table 1. For BH1 boreholes, soil conditions are only prone to a depth of 15 meters. The proportions of stress ratios and RL ratios, as well as the relationships listed in Table 2 for BH1 boreholes, are presented. According to Table 2, the BH1 borehole is not susceptible to liquefaction at a depth of 15 m. The results of field experiments and soil

characteristics for the BH2 borehole are given in Table 3. For BH2 boreholes, soil conditions are prone to 13, 11, 8, 4, 2 and 16 m in depth. The coefficients of cyclic stress ratio and cyclic resistance ratio (RL) as well as FL are given using the equations listed in Table 4 for the BH2 borehole. According to Table 4 at depths of 4.2 and 13 m, BH2 is prone to liquefaction.

The results of field experiments and soil characteristics for the BH3 borehole are given in Table 5. A depth of 2 m of groundwater level was reported for BH3 and BH4 boreholes during drilling. For BH3 samples, soil conditions are not susceptible to liquefaction, so they are ignored. The results of field experiments and soil characteristics for the BH4 borehole are presented in Table 6. For BH4 boreholes, soil conditions are only prone to a depth of 13 meters. The coefficients of cyclic stress ratio and cyclic resistance ratio (RL) as well as FL are given using the equations listed in Table 7 for the BH4 borehole. According to Table 7, BH4 borehole is not susceptible to liquefaction at a depth of 13 meters. The results of field experiments and soil characteristics for the BH5 borehole are given in Table 8. The depth of 1.5 meters of groundwater level for BH5 boreholes was reported at drilling time. For BH5, the soil conditions are not susceptible to liquefaction; therefore, they are neglected. The results of field experiments and soil characteristics for the BH6 borehole are presented in Table 8. The depth of 1.5 m of groundwater level for BH6 boreholes was reported at drilling time. For the BH6 borehole, soil conditions are only prone to 6 m depth. The coefficients of cyclic stress ratio and cyclic resistance ratio (RL) as well as FL are given using the equations listed in Table 9 for the BH6 borehole. According to Table 9, the BH6 borehole is not prone to liquefaction at a depth of 6 m.
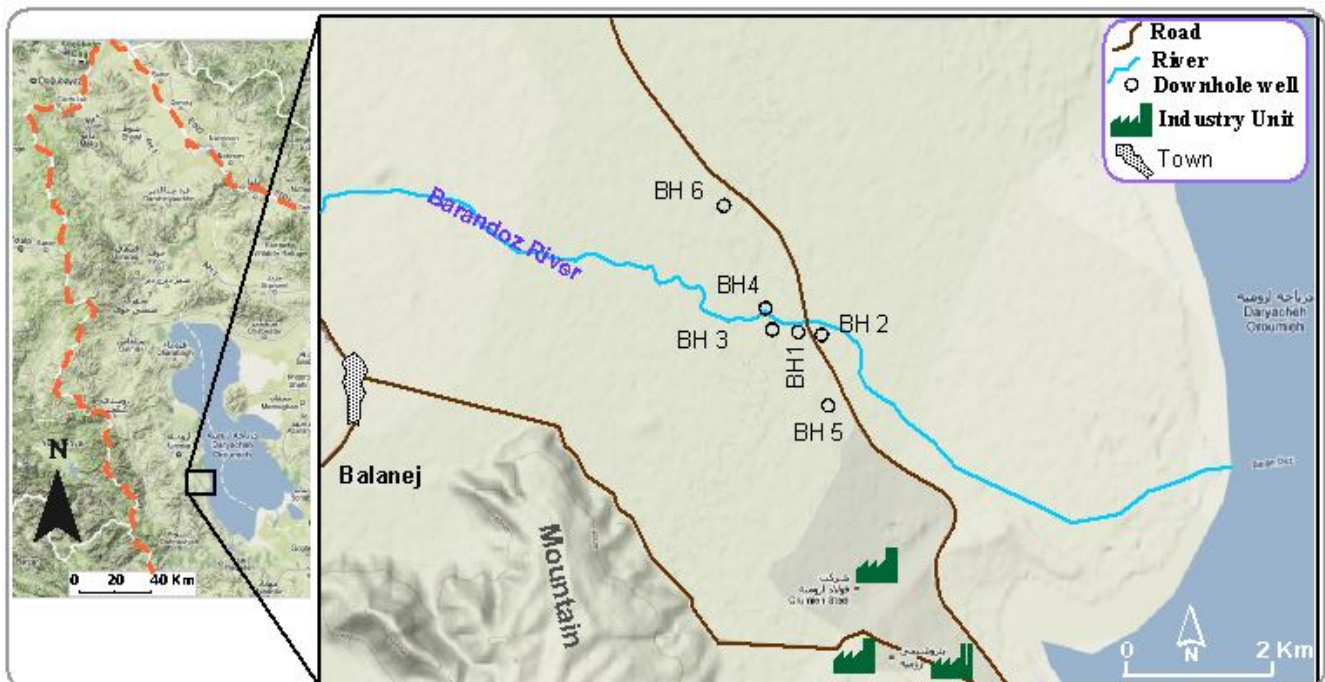

Figure 1. The geographic location of the east of Urmia plain and the location of drilled boreholes

Table 1. Results of field and laboratory experiments in BH1 borehole

Depth (m)	SPT (N)	Geology	Descried of materials									
			USCS	#200	#4	PI (%)	LL (%)	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	nature
3	11	Clay with low Softness	CL	90	100	24	45	<0.001	0.0062	12	1.7	Average
5	17	Clay with low Softness	CL	92	100	24	43	<0.001	0.007	17	1.7	Stiff
7	30	Sand with low gravel	GP-GM	9	48	NP	-	0.1	6.0	30	1.7	Dense
10	31	Clay with low Softness	CL	91	100	21	35	<0.001	0.023	27	1.7	Very stiff
12	24	Clay with low gravel	CL	89	100	18	35	<0.001	0.016	20	1.7	Very stiff
15	11	Clay with low gravel	CL	77	100	12	29	<0.001	0.04	8	1.7	Stiff
17	14	Clay with low softness and gravel	CL	80	100	23	41	<0.001	0.01	10	1.7	Stiff
18	17	Clay with high softness	CH	91	100	35	56	<0.001	0.0026	12	1.7	Very stiff
20	26	Clay with high softness	CH	90	100	34	55	<0.001	0.0025	18	1.7	Very stiff
24	19	Clay with low softness and gravel	CH	64	95	32	51	<0.001	0.007	12	1.7	Very stiff
25	18	Clay with low softness and gravel	CL	67	93	11	29	<0.001	0.05	11	1.7	Very stiff

Table 2. Calculation of FL and PL coefficient of BH1 borehole

Parameter	Z	SPT	σ'_{v0}	σ_{v0}	CSR	$CRR_{7.5}$	MSF	K_σ	K_α	FL	PL
Unit	m	N	kPa	kPa	-	-	-	-	-	-	%
Value	15	11	177.6	310.64	0.298	0.355	1.32	1.33	1.0	2.1	0.0
										PL (%)	0.00

Table 3. Results of Field and Laboratory Experiments in BH2 Borehole

Depth (m)	SPT (N)	Geology	Descried of materials									
			USCS	#200	#4	PI (%)	LL (%)	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	nature
2	12	Silty gravel with sand	SM	31	71	NP	-	0.02	0.35	15	1.7	Average
4	39	Silty gravel	SM	24	63	NP	-	0.02	1.3	38	1.7	Dense
6	24	Silty gravel	SM	8	60	NP	-	0.17	2.8	23	1.7	Dense
8	5	Clay with silt and gravel	CL-ML	58	99	5	22	<0.006	0.062	5	1.7	Loose
9	14	Clay with gravel	CL	74	99	13	33	<0.001	0.024	13	1.7	Stiff
10	23	Clay with gravel	CL	82	99	21	40	<0.001	0.032	20	1.7	Very stiff
11	26	Clay with gravel	CL	76	9	14	31	<0.001	0.018	22	1.7	Very stiff
13	13	Silty gravel	SM	46	100	NP	-	<0.003	0.14	10	1.7	Average
15	13	Clay with high softness	CH	86	100	33	51	<0.001	0.004	10	1.7	Stiff
16	15	Clay with gravel	CL	74	100	11	26	<0.001	0.048	11	1.7	Stiff
18	17	Clay with high softness	CH	77	94	37	60	<0.001	0.002	12	1.7	Stiff
20	19	Gravely clay	CL	62	92	22	39	<0.001	0.035	13	1.7	Very stiff
22	26	Gravely clay	CL	68	86	29	48	<0.001	0.008	16	1.7	Very stiff
24	30	Clay with gravel	CL	73	93	19	34	<0.001	0.028	18	1.7	Very stiff

Table 4. Calculation of FL and PL coefficient of BH2 borehole

Parameter	Z	SPT	σ'_{v0}	σ_{v0}	CSR	$CRR_{7.5}$	MSF	K_σ	K_α	FL	PL
Unit	m	N	kPa	kPa	-	-	-	-	-	-	%
	2	12	37.24	40.24	0.231	0.051	1.32	1.0	1.0	0.3	13
	4	39	58.84	81.84	0.288	0.015	1.32	1.0	1.0	0.1	29
	8	5	102.04	165.04	0.313	0.258	1.32	1.01	1.0	1.1	0.0
Value	9	14	112.84	185.84	0.313	0.353	1.32	1.06	1.0	1.6	0.0
	11	26	134.44	227.44	0.311	0.366	1.32	1.16	1.0	1.8	0.0
	13	13	156.04	269.04	0.305	0.134	1.32	1.31	1.0	0.8	9.0
	16	15	188.44	331.44	0.294	0.345	1.32	1.31	1.0	2.1	0.0
										PL (%)	7.00

Table 5. Results of Field and Laboratory Experiments in BH3 Boreholes

Depth (m)	SPT (N)	Geology	USCS	#200	#4	PI (%)	LL (%)	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	nature
2	12	Clay with low softness	CL	82	99	17	37	<0.001	0.0042	15	1.5	Average
4	11	Clay with low softness	CL	50	96	17	34	<0.001	0.013	11	1.5	Average
7	16	Clay with low softness and gravel	CL	82	97	20	39	<0.001	0.0037	16	1.5	Stiff
9	17	Clay with low softness and gravel	CL	83	100	19	39	<0.001	0.005	15	1.5	Very stiff
11	30	Clay with low softness	CL	72	100	20	40	<0.001	0.0065	26	1.5	Very stiff
14	26	Clay with low softness	CL	77	98	19	39	<0.001	0.005	20	1.5	Very stiff
16	23	Clay with low softness and gravel	CL	77	98	24	44	<0.001	0.0042	17	1.5	Very stiff
18	58	Clay with low softness	CL	82	100	25	47	<0.001	0.0022	40	1.5	Strong

Table 6. Results of Field and Laboratory Experiments in BH4 Borehole

Depth (m)	SPT (N)	Geology	USCS	#200	#4	PI (%)	LL (%)	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	nature
2	9	Clay with low softness	CL	83	100	22	43	<0.001	0.0035	11	1.5	Average
5	8	Clay with low softness	CL	87	100	17	36	<0.001	0.0055	8	1.5	Average
8	31	Clay with low softness and gravel	CL	84	99	22	42	<0.001	0.0035	12	1.5	Stiff
11	21	Clay with low softness	CL	80	97	17	35	<0.001	0.005	10	1.5	Average
13	30	Clay with low softness	CL	68	100	12	28	<0.001	0.018	10	1.5	Very stiff
15	5	Clay with low softness	CL	84	98	22	41	<0.001	0.005	4	1.5	Loose
17	33	Clay with low softness	CL	79	100	23	42	<0.001	0.0038	24	1.5	Strong
19	28	Clay with low softness	CL	80	100	20	39	<0.001	0.0048	19	1.5	Very stiff

Table 7. Calculation of FL and PL coefficient of BH4 Borehole

Parameter	Z	SPT	σ'_{v0}	σ_{v0}	CSR	$CRR_{7.5}$	MSF	K_σ	K_α	FL	PL
Unit	m	N	kPa	kPa	-	-	-	-	-	-	%
Value	13	12	154.2	269.2	0.309	0.322	1.32	1.24	1.0	1.7	0.0
										PL (%)	0.00

Table 8. Results of Field and Laboratory Experiments in BH6 Borehole

Depth (m)	SPT (N)	Geology	USCS	#200	#4	PI (%)	LL (%)	D_{10}(mm)	D_{50}(mm)	$(N_1)_{60}$	h_w (m)	nature
2	6	Clay with low softness	CL	89	100	23	46	<0.001	0.0038	7	2.0	Soft
4	8	Clay with low softness	CL	91	100	18	37	<0.001	0.0082	8	2.0	Average
6	6	Clay with low softness	CL	88	100	11	29	<0.001	0.003	6	2.0	Average
8	4	Clay with low softness	CL	90	100	16	36	<0.001	0.013	4	2.0	Soft
10	17	Clay with low softness	CL	88	100	20	42	<0.001	0.0063	14	2.0	Very stiff
13	18	Clay with low softness	CL	88	100	17	36	<0.001	0.01	14	2.0	Very stiff
15	22	Clay with low softness and gravel	CL	85	100	13	30	<0.001	0.033	16	2.0	Very stiff

Descried of materials heads span USCS...nature.

Table 9. Calculation of FL and PL coefficient of BH6 Borehole

Parameter	Z	SPT	σ'_{v0}	σ_{v0}	CSR	$CRR_{7.5}$	MSF	K_σ	K_α	FL	PL
Unit	m	N	kPa	kPa	-	-	-	-	-	-	%
Value	6	6	83.2	123.2	0.296	0.404	1.32	1.0	1.0	1.8	0.0
										PL (%)	0.00

4. Conclusion

Of the 6 existing boreholes, the liquefaction potential obtained for the BH1 borehole is zero. Therefore, the risk of liquefaction is very low. The liquefaction potential for the BH2 borehole is equal to 7. The liquefaction risk is high, therefore, this range requires further research. The BH5 and BH3 boreholes are not prone to liquefaction. For BH6 borehole it is equal to zero. Therefore, the risk of liquefaction is very low. In general, due to the density of industries and existing roads, the southern part of Urmia plain requires further geotechnical studies including boreholes, as well as geophysical studies, including the use of shear waves.

Acknwledgements

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