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Study of Possibility of Liquefaction in the Body and Foundation of Embankment Dams - Case Study of Sattarkhan Dam

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Abstract: Four big tectonic plates: Lut, Caspian, Arabian and Persian tectonic plates across the region covered the whole of Iran. The faults on the country have caused severe dangers during earthquakes. These earthquakes may cause many damages, which are considerable within the important structures. An embankment dam is one but important of them and may lead to large financial losses, floods and mortality in the case of failure. During Earthquake, one of the main dangers that threaten an embankment is liquefaction. The phenomenon of liquefaction summarized as loose of saturated sands, gravels, or silts having a contractive structure and it may occur when such materials are subjected to earthquake`s shear deformations due to increasing pore water pressure. Liquefaction induced ground deformation and building settlements were severe and widespread during the 1990 Manjil, Iran earthquake [1]. But the first fundamental observation of damages attributed to liquefaction was made in the 1964 Niigata, Japan and 1964 Alaska, North America earthquakes. At the present paper, first, we study the phenomenon of liquefaction, parameters and also it's affecting factors; then, we study effects of liquefaction on Sattarkhan embankment dam (East Azarbaijan Province-Iran) which is constructed on Aharchay basin/river using the yield and liquefaction strength ratios; next, our data will be modeled by a computer program. The yield strength ratio is used for correct prediction the occurrence of liquefaction in the upstream hydraulic fill of the dam and the liquefaction shear strength ratio is used for accurate prediction of the subsequent flow failure for up and downstream slopes.

Key words: Liquefaction · Soil Layers · Cyclic Stress Ratio (CSR) · Cyclic Resistance Ratio (CRR)

INTRODUCTION

Liquefaction occurs when the static equilibrium of soil is disturbed; either through cyclic or repetitive loading, causing it to behave like a liquid rather than a solid state. In saturated soils, under quick cyclic loading, there isn`t enough time for dissipation of the generated pore water pressure. Increased pore water pressure from liquefied soils may exert unanticipated burden on the body or foundation of embankments that are located within a liquefiable region, resulting in structural failures or unacceptable differential settlement. Increased excess water may cause buoyancy of buried structures and boiling of sands. Typical geotechnical damage includes: lateral spreading, slope instability, or landslides; ground settlement, oscillation or loss of bearing capacity of structures [2], [3]. This results in a reduction of soil contact forces to sustain loads. The potential for liquefaction in an embankment or its foundation must be evaluated on the basis of empirical knowledge and engineering judgment supplemented by special laboratory tests when necessary [4]. These tests can include SPT and CPT that each of their specific parameters is necessary for assessment of CSR and CRR in an embankment liquefaction studies [5].

After this process we can find the factor of safety to establish whether an embankment liquefies or not. If the factor of safety is around or below 1, due to low strength of soil, the remediation techniques such as Dynamic Compaction, Drainage and Deep Vibro… required reducing this risk.

In order to discover mistakes of manual methods and quick modification of errors, computer software following by a statistical method must be done [6].

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Liquefaction; Function and Phenomenon: Calculation or estimation of two variables is required for evaluation of liquefaction resistance of soils or occurrence of liquefaction. These variables are Cyclic Stress Ratio (CSR) and Cyclic Resistance Ratio (CRR). Previously, CRR was called the cyclic stress ratio required to generate liquefaction, or the cyclic strength ratio.

CSR can be achieved by Equation 1:

$$CSR = \frac{\tau_{ave}}{\sigma'_{v0}} = 0.65\left(\frac{a_{\max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right)r_d \qquad (1)$$

where $\alpha_{\max}$ is the ground horizontal surface acceleration, g is the gravity acceleration, σ_{v0} and σ'_{v0} is the total and effective stresses respectively and r_d is stress reduction coefficient [7].

For practical design in field study at the dam site Equation 2 is used for evaluation of $\alpha_{\max}$ as this pattern:

$$\alpha_{\max} = 0.184\times10^{0.32M} \times D^{-0.8}g \qquad (2)$$

where M is the magnitude of earthquake (Richter) and D is maximum distance of epicenter in kilometer [8]. This factor is described here in Equations 3-1 through 3-4.

Moreover stress reduction coefficient is a depth-dependent and dimensionless factor that can be written as below [9]:

$$r_d = 1.0 - 0.00765z \qquad z \le 9.15m \qquad (3\text{-}1)$$

$$r_d = 1.174 - 0.0267z \qquad 9.15m < z \le 23m \qquad (3\text{-}2)$$

$$r_d = 0.744 - 0.008z \qquad 23m < z \le 30m \qquad (3\text{-}3)$$

$$r_d = 0.5z \qquad z > 30m \qquad (3\text{-}4)$$

For ease of electronic computation and as an alternative, r_d may also be expressed as Equation 4 explained as a ratio:

$$r_d = \frac{A}{B} \qquad (4)$$

where:

$$A = (1.0 - 0.4113Z^{0.5} + 0.04052z + 0.001753z^{1.5})$$
$$B = (1.0 + 0.4177Z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^{2})$$

and z is the depth from surface of the earth in meters. Fig. 1 demonstrates the variation of r_d with the depth [10].

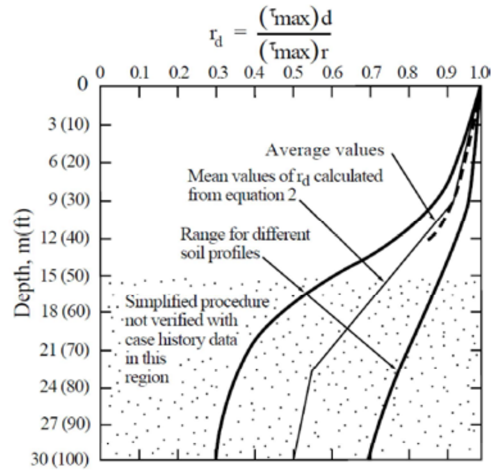

Fig. 1: r_d mutations with depth developed by Seed and Idriss

Criteria for Evaluation of Liquefaction Based on SPT Parameters: Methods related to SPT N-values for liquefaction resistance assessment utilize SPT blow count that is normalized to an effective overburden pressure of 100 KPa. This normalized SPT blow count is denoted as N_1, which is obtained by multiplying the uncorrected SPT blow count by a depth correction factor C_N [11]. A correction factor must be needed to correct the blow count for an energy ratio of 60%, which has been adopted as the average SPT energy for AGP (American Geotechnical Practice).

Additional correction factors are needed to apply for obtaining the corrected normalized SPT-N value, $(N_1)_{60}$ can be written via Equation 5:

$$(N_1)_{60} = N_m\, C_N\, C_E\, C_R\, C_B\, C_S \qquad (5)$$

where N_m is measured SPT value at the site and C_x are correction factors and can be summarized for various equipment parameters in Table 1:

Corrections for fine content, FC, (% passing 0.075mm sieve) could be calculated from Equation 6. After a wide review by NCEER (National Center for Earthquake Engineering Research) workshop, they pointed out that correction for fines content must be a function of penetration resistance as well as fines content itself. The participants also agreed that other grain characteristics such as soil plasticity may affect liquefaction resistance; hence any correlation based solely on penetration resistance and fines content should be used with engineering judgment and caution, based on Seed and Idriss recommendations we can have [12, 13]:

Table 1: Correction Factors for observed SPT Values *. Skempton (1986) and Youd *et al*., (2001)

Factor	Equipment Parameter	Term	Correction Factor
Overburden Pressure	Independent of Equipment	C_N	$\sqrt{P_a/\sigma'_v}$ *
Energy Ratio	Safety Hammer	C_E	0.6 to 1.17
	Donut Hammer		0.45 to 1.0
Rod Length	3.0 to 4.0 m	C_R	0.75
	4.0 to 6.0		0.85
	6.0 to 10.0 m		0.95
	10.0 to 30.0 m		1.0
	>30.0 m		>1.0
Borehole Diameter	65 to 115 mm	C_B	1.0
	150 mm		1.05
	200 mm		1.15
Sampling Method	Standard Sampler	C_S	1.0
	Sampler without liners		1.2

$$(N_1)_{60CS} = \alpha + \beta(N_1)_{60} \tag{6}$$

where α and β are coefficients determined from the following relationships 7-1 through 7-6 [14]:

$$\alpha = 0 \quad FC \leq 5\% \tag{7-1}$$

$$\alpha = \exp\left[1.76 - \frac{190}{FC^2}\right] \quad 5\% < FC < 35\% \tag{7-2}$$

$$\alpha = 5.0 \quad FC \geq 35\% \tag{7-3}$$

$$\beta = 1.0 \quad FC \leq 5\% \tag{7-4}$$

$$\beta = [0.99 + FC^{1.5}/1000] \quad 5\% < FC < 35\% \tag{7-5}$$

$$\beta = 1.2 \quad FC \geq 35\% \tag{7-6}$$

Triggering Liquefaction Potential and Relation Between SPT Values for Calculation of CRR: The most basic procedure used in engineering practice for assessment of the site liquefaction potential is: "Simplified Procedure". This procedure essentially compares the Cyclic Resistance Ratio (CRR) with the earthquake induced Cyclic Stress Ratio (CSR) at that depth from a specified design earthquake (defined by a peak ground acceleration and an associated earthquake magnitude).

Values of CRR were originally established from empirical correlations using extensive databases for sites that was or was not liquefied during past earthquakes, where values of $(N_1)_{60}$ could be correlated with liquefied strata. The current version of the baseline chart defining values of CRR as a function of $(N_1)_{60}$ for earthquakes with magnitudes of 7.5 is shown here in Fig. 2. (Proposed by NCEER workshop and Youd *et al*.) [10, 14].

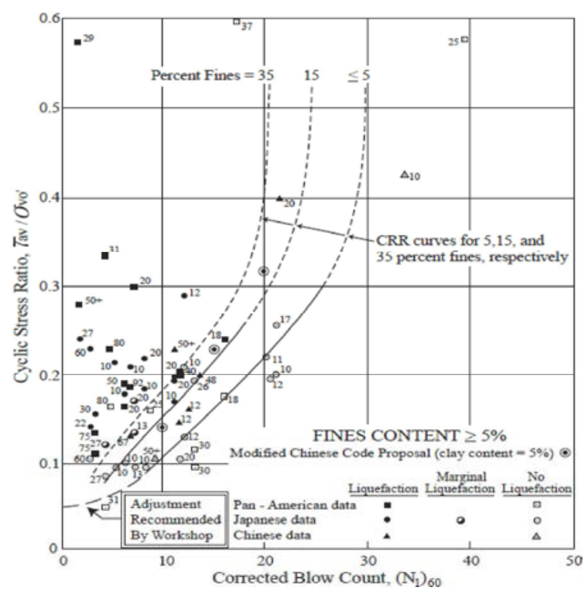

Fig. 2: Correlation Between CSR and SPT $(N_1)_{60}$ Value for M = 7.5 and Varying FC

The CRR curve for fines content less than five percent ($FC < 5\%$, is the basic penetration criterion for the simplified procedure. It should be noted that the CRR curves in Fig. 2 are valid just for earthquakes with magnitude 7.5 [13, 15].

Considering above, it`s very important for us to normalize the CSR with magnitude of M to an earthquake with the magnitude of 7.5 Richter described in Equation 8 [12]:

$$CSR_{7.5} = \frac{CSR}{MSF.K_\sigma} \tag{8}$$

Magnitude Scaling Factor (MSF): MSF is used for adjustment of induced CSR during an earthquake with a

Table 2: Region faults that previously had some activities

Fault Name	Fault Length (Km)	Distance from Epicenter (Km)	Expected Magnitude (Richter)	Average Acceleration
Jonoub-e-Ahar	75	2	7.0	0.23g
Khalaj	26	3	6.5	0.16g
Bahlool	45	55	6.8	0.09g
Kafan	120	60	7.2	0.12g

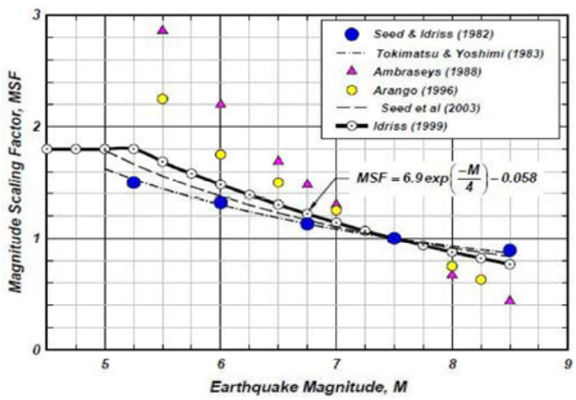

Fig. 3: MSF Values proposed by various researchers

specified magnitude to an earthquake of M = 7.5, thus the intensity of ground motions are accounted in the formulations of the CSR and MSF provides an approximate representation of the effects of shaking duration or equivalent number of stress cycles. Although this can be obtained by several different methods, but according to Iran seismology stations, Equation 9 is used [7, 16]:

$$MSF = \frac{10^{2.24}}{M^{2.56}} \approx \left(\frac{M}{7.5}\right)^{-2.56} \quad (9)$$

Note that this Equation and another MSF relations provided by various researchers are also available (Fig. 3) [16].

Therefore MSF provides an approximate representation of the effects of shaking duration or equivalent number of stress cycles. Instead of above considerations we also can use Equation 10 for MSF calculation:

$$MSF = 6.9\exp\left(\frac{-M}{4}\right) - 0.058 \le 1.8 \quad (10)$$

The value of MSF obtained from Table 2 and the Equations are presented in Fig. 3 as below [12]:

Overburden Correlation Factor (K_σ): K_σ is overburden correction factor and use for adjustment of CSR to the effective overburden equivalent to atmospheric pressure (equal to 760 mm Hg). This correction factor briefly summarized here:

Commonly, by using the Equation 11, we can get the amount of K_σ:

$$K_\sigma = \left(\frac{\sigma'_V}{P_a}\right)^{f-1} \quad (11)$$

where $f \approx 0.6 - 0.8$, but using 0.7 is usual and P_a is atmospheric pressure equal to 1 bar [14].

This Equation which proposed by Hynes and Olsen is very useful because of its simplicity and with no limitations; therefore utilized for the case study of Sattarkhan dam, But there is also another definition in this discussion that authors can`t use because of its severe limitations, which demonstrated below, according to Boulanger and Idriss research:

$$K_\sigma = 1 - C_\sigma Ln\left(\frac{\sigma'_V}{P_a}\right) \quad (12)$$

The coefficient of C_σ is defined as Equation 13 [9]:

$$C_\sigma = \frac{1}{18.9 - 17.3D_R} \le 0.3 \quad (13)$$

where D_R is the Density Ratio and can be calculated using Equation 14 [9]:

$$D_R = \sqrt{(N_1)_{60}/46} \quad (14)$$

which are based on $Q \approx 10$, $K_\sigma \approx 0.45$, $D_R \approx 0.9$, and $\sigma'_V/P_a \le 10$ [9].

Evaluation of the Cyclic Resistance Ratio (CRR): The CRR base curve for clean sands (i.e. <5% fines content) could be approximated by Equation 15:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \quad (15)$$

where:

$a = 0.048$, $b = -0.1248$, $c = 0.004721$, $d = 0.009578$, $e = 0.000613$, $f = -0.0003285$, $g = -1.673 \times 10^{-5}$, $h = 3.714 \times 10^{6}$ and $x = (N_1)_{60}$.

It is necessary to point out that Equation 15 is valid for $(N_1)_{60}$ less than 30. For clean granular soil with $(N_1)_{60}$ more than 30 that is too dense to liquefy and are generally classed as non-liquefiable. Also another expression which often used for common practice about CRR in granular soils that we used it for our case study is Equation 16.

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60CS}} + \frac{(N_1)_{60CS}}{135} + \frac{50}{\left[10(N_1)_{60CS}+45\right]^2} - \frac{1}{200} \qquad (16)$$

where $(N_1)_{60CS}$ is the SPT blow counts that are corrected for overburden stress in critical state condition [7].

After chain calculations with considering of above meanings, finally we can obtain the factor of safety which has a key role in study of liquefaction occurrence on the body and foundation of an embankment dam. Equation 17 shows the factor of safety which we need for our study:

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) MSF.K_{\sigma} \qquad (17)$$

This Equation describes that probabilities is too high, if the coefficient is equal or less than unit in determined condition of earthquake magnitude and peak ground surface acceleration.

MATERIALS AND METHODS

Assessment of Liquefaction in the Case Study of Sattarkhan Embankment Dam: Sattarkhan Dam is one of the Iran Ministry of Energy projects that was constructed to prevent the water flow out of the country borders, irrigation of 12450 Hectares of downstream farms and compensation demands for drinking water in the Ahar(Arasbaran) region, adjacent towns and suburbs in the East Azarbaijan`s north district. This dam is located in 120 Km. north-east of Tabriz city and also in 15 Km. west far from the city of Ahar.

The dam with chimney drainage has a clay core with CL to SC material classification and GP classification for body in both up and downstream. It has a cutoff wall for pore water pressure and seepage control. The overall volume of the tank is 135,000,000 cubic meters [17], [18].

Fig. 4 and Fig. 5 represent the overview and cross section of the dam, respectively:

The Ahar`s climatology station (1967) was converted to synoptic (1987), located in cold and mountainous part of city district with four active pluviometry stations named: Varzeghan, Kasanagh, AseGhadim and KhalifehAnsar and the overall view of region`s climatology can be briefly summarized as:

Fig. 4: Overall view of Sattarkhan Dam

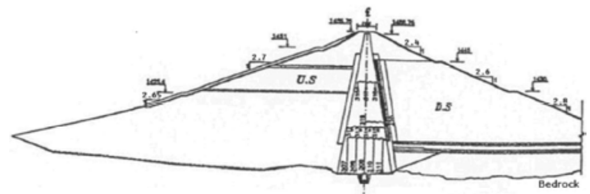

Fig. 5: Cross section of Sattarkhan Dam

Fig. 6: Aharchay Basin Tazehkand (46°53′,38°28′) and Eshdalagh (46°59′, 38°26′) Village

Average Annual Temperature Changes: 60 Celsius
Average Annual Glacial Days: 122 Days
Average Total Annual Sunny Days: 2784 Hours
Average Total Annual Rainfall height: 330 mm/Year

Based our calculations from hydrological aspect, Sattarkhan dam is located in Aharchay basin of 1162 square kilometers area, 179 kilometers pyramid, 703.404 minutes of time of concentration and 317.7 CMS discharge with 50 years return period. Fig. 6 demonstrates the Aharchay basin.

Soil composition is also important. Well-graded, angular particles reduce excess pore water pressure more quickly and effectively and also are less susceptible to liquefaction [19, 20]. Especially history and magnitude of the past earthquake in the region has an important role in the future earthquakes and in liquefaction [21]. In seismological countenance four big faults surrounded region that some of them are active since year 1900, these are listed below in Table 2 with their specifications:

Table 4: Boreholes with geographic coordination and their water tables

Row	BH	Global Positioning System X	Y	Z	Water Table(m)
1	103	665655	4258367	1400.80	1.00
2	106	665596	4258435	1401.80	1.70
3	201	665715	4258353	1400.84	0.90
4	205	665825	4258294	1400.84	1.20
5	206	665794	4258251	1399.55	1.20
6	208	665830	4258218	1398.40	0.00
7	209	665883	4258161	1398.17	0.00
8	212	665771	4258215	1399.43	0.90

Table 5: Liquefaction analysis of Borehole 205 for certain magnitude and acceleration

Row	Depth	σ_v	u	$\sigma`_v$	$(N_1)_{60}$	α	β	$(N_1)_{60CS}$	FC	r_d	$K\sigma$	CSR	$CSR_{7.5}$ (M=6.5)	CRR	FS
1	0.4	80.8	.4	80.4	31	5	1.2	42.2	36	0.99	1.07	0.08	0.05	0.19	3.66
2	1.0	84.3	1	83.3	34	5	1.2	45.8	36	0.99	1.06	0.08	0.05	0.25	4.86
3	2.0	85.5	2	83.5	24	0	1.0	24.0	5	0.96	1.06	0.08	0.05	0.27	5.29
4	3.0	88.9	3	85.9	45	.6	1.0	46.3	9	0.98	1.05	0.08	0.05	0.26	4.92
5	4.0	92.1	4	88.1	60	.6	1.0	61.6	9	0.97	1.04	0.08	0.05	0.42	7.86
6	5.0	92.2	5	87.2	39	.6	1.0	40.2	9	0.96	1.04	0.08	0.05	0.13	2.51
7	6.0	95.3	6	89.2	39	.6	1.0	40.2	9	0.95	1.04	0.08	0.05	0.13	2.48
8	7.0	97.0	7	89.9	20	5	1.2	28.7	34	0.95	1.03	0.08	0.05	0.40	7.40
9	8.5	102.3	9	93.7	15	5	1.2	23.0	100	0.94	1.02	0.08	0.05	0.26	4.74
10	9.7	102.8	10	92.8	23	4	1.1	29.4	23	0.92	1.02	0.08	0.05	0.43	8.00
11	11	107.0	12	95.5	42	0	1.0	42.0	5	0.88	1.01	0.08	0.05	0.18	3.45

Furthermore, Table 3 illustrates two major earthquakes occurred in the dam location areas as follows [17]:

Therefore we can easily find surface acceleration of them via Equation 2 as follows:

$$M = 5.5 \Rightarrow \alpha_{\max} = 0.184\times10^{0.32\times5.5}\ 80^{-0.8}\ g = 0.32g$$
$$M = 6.5 \Rightarrow \alpha_{\max} = 0.184\times10^{0.32\times6.5}\ 90^{-0.8}\ g = 0.604g$$

To study liquefaction possibility, we must have the boreholes data and their location in both up and downstream. First, with gathered information, formulas and provided boreholes data, the following parameters must be calculated:

σ'_V, σ'_V/σ_V, $\alpha_{\max}/g$, r_d for Cyclic Stress Ratio and $(N_1)_{60}$ and $\Delta(N_1)_{60}$ (for given FC), K_σ (for given σ'_V/P_α), MSF (for given M) for Cyclic Resistance Ratio for each borehole in certain depths that there was information for obtained ground surface acceleration, then we obtain factor of safety. Next, with specified unit weight, a computer program needed to control the results of first manual method.

According to author`s field tests and borehole investigations from drilled boreholes, Table 4 indicates the name and their location in the site:

In order to prevent repetition and according to all of gathered data, which necessary for both methods in this research, the process described for a specific borehole with determined acceleration, magnitude and depth as an example (Example and Table 5).

Example: By selection of borehole 205, located in northern downstream side of the dam with an earthquake of a magnitude of 6.5 and surface acceleration of 0.12g, for the 1st and 10th rows:

For 1st row: Depth = 0.4 m

$$(z = 0.4m) \leq 9.15m \Rightarrow rd = 0.9969$$
$$FC \geq 35\% \Rightarrow \alpha = 5, \beta = 1.2$$
$$(N_1)_{60CS} = 42.2, K_\sigma = 1.067635, MSF = 1.441$$
$$CSR = 0.07814, CRR_{7.5} = 0.185871, FS = 3.66078$$

For 10th row, (respectively): Depth=9.7m

$$9.15m < (z = 9.7m) \leq 23m \Rightarrow rd = 0.91501$$
$$(5\% < (FC = 23\%) < 35\%) \Rightarrow \alpha = 4.0585, \beta = 1.1$$
$$(N_1)_{60CS} = 29.3585, K_\sigma = 1.0226, MSF = 1.441$$
$$CSR = 0.07906, CRR_{7.5} = 0.428354, FS = 7.9839$$

Table 6: Calculated FS data for almost all boreholes in up and downstream of the dam

Depth	a_{max}	M	a_{max}	M	a_{max}	M	a_{max}	M	a_{max}	M	a_{max}	M
BH 103	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
4.00	1.017	1.574	1.326	1.870	0.763	1.211						
8.00	2.603	4.184	3.395	4.970	1.953	3.220						
9.50	1.402	2.287	1.828	2.717	1.052	1.760						
BH 106	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
5.20	3.935	6.092	5.132	7.236	2.953	4.688						
7.65	6.066	9.478	7.910	10.000	4.552	7.502						
10.20	4.486	7.317	5.851	8.629	3.366	5.632						
BH 201	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
0.00	1.516	2.637	1.978	3.133	1.138	2.029						
1.00	4.505	7.834	5.875	9.306	3.381	6.029						
2.00	2.440	4.243	3.182	5.040	1.831	3.266						
BH 205	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
0.40	1.260	3.661	1.643	2.603	0.945	1.686						
1.00	2.248	4.855	2.931	4.643	1.686	3.00						
2.00	3.04	5.291	3.968	6.285	2.283	4.072						
BH 206	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
1.00	5.832	9.054	7.605	10.000	4.376	7.805						
2.00	3.574	5.619	4.661	7.382	2.682	4.783						
3.00	2.831	4.481	3.692	5.848	2.124	3.789						
BH 208	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
0.50	7.278	10.000	9.491	10.000	5.461	9.740						
2.50	6.580	10.000	8.581	10.000	4.938	8.806						
3.50	8.223	10.000	10.000	10.000	6.170	10.000						
BH 209	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
0.50	5.885	10.000	7.675	10.000	4.416	7.876						
2.20	4.452	7.741	5.806	9.195	3.341	5.958						
3.10	7.098	10.000	9.256	10.000	5.326	9.499						
BH 212	0.32	5.5	0.12	6.5	0.16	6.5	0.09	6.8	0.23	7.0	0.12	7.2
0.50	7.293	10.000	9.511	10.000	5.473	9.760						
2.00	4.351	7.566	5.674	8.988	3.265	5.823						
3.00	4.317	7.507	5.630	8.917	3.239	5.777						

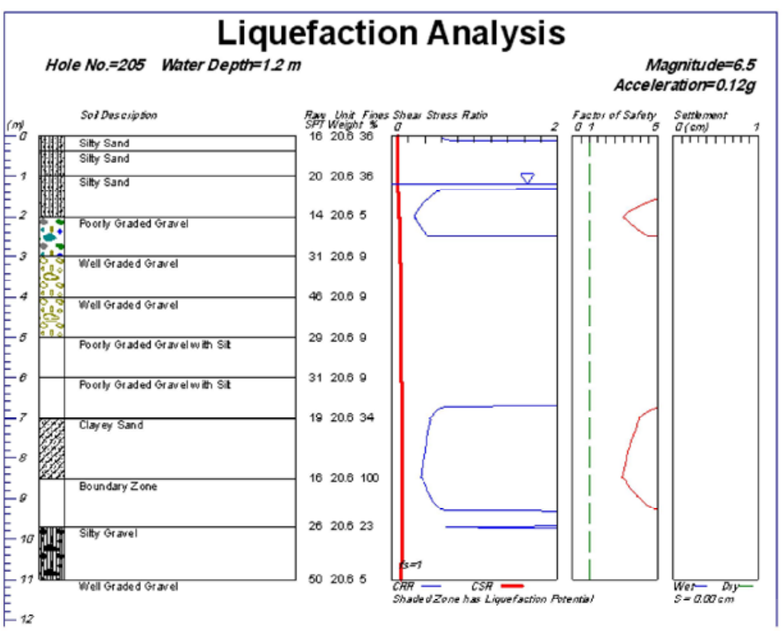

Fig. 7: Plotted CRR-CSR curves and calculated FS for BH205 of Sattarkhan Dam with LiguefyPro™ (Soil layer`s specification at the dam site investigated by laboratory tests) ($M = 6.5$, $\alpha_{max} = 0.12g$)

As described, in order to check the accuracy of archived results, Fig. 7 illustrates summarized computer results of BH 205. This process completed for all boreholes.

Therefore, by step-by-step and accurate controlling of results, negligible errors found in some part of data ranges. This little error was due to unit weight changes with soil layers and depth, because the authors used total and effective stresses in the calculation while software used layers unit weight and depth for computation. Moreover the software does not draw the FS more than 5 but we can achieve these values by referring the program`s summary report.

By completing above mentioned steps for liquefaction analysis in the dam for almost all upstream and downstream boreholes in liquefiable region, there is inconsiderable difference between 2 methods. Table 6 summarizes FS results for the first three depths on each borehole:

RESULTS AND DISCUSSION

Liquefaction control and its review, play an important role before construction and during service life of a dam, because this issue can cause other events in the dam, like increasing pore water pressure, soil boiling... Denser and less saturated soils are not susceptible to liquefaction under low shear stresses and liquefaction phenomenon is also possible on clays and fine-grained soils.

The authors investigated liquefaction triggering safety factor and it`s damaging effects in case of earthquake in an embankment dam on Aharchay watershed located on cold climate and fault zone with different magnitudes and accelerations derived from the past seismic activities at adjacent faults. Therefore liquefaction of the foundation and body of a dam under various earthquakes has been reported as one of the major causes of harm. In order to control the validity and accuracy of results, the authors checked it by a computer program; almost same results from both methods were observed. Among these results some points had low safety factors. They can be referred to 201,208,209 and 212 boreholes only in one or maximum two points under a severe possible earthquake with high magnitude and acceleration. Authors believe that these results are impractical because field studies, which done by them, pointed out there are some nonliquefiable points and zones near studied boreholes at the same depth and regions.

Liquefaction primarily might be limited to loose area in the structure near to the embankment/foundation interface [12]. As the result of study shows liquefaction is also likely to occur at the tailings retained by the dam. An estimate of total deformation resulting from each seismic event will require a post liquefaction evaluation which was not included in these analyses.

Earthquake magnitude, M, should be used for estimation of earthquake size in liquefaction resistance calculations. No general corrections are recommended about adjust earthquake magnitude to account for differences in duration due to source mechanism or geographic region.

REFERENCES

1. Yegian, M.K., V.G. Ghahraman, A.A. Nogole Sadat and H. Daraie, 1995. Liquefaction During the 1990 Manjil, Iran. Earthquake, II Case History Analyses, Bultin of the Seismological Society of America, 85(1): 83-92.
2. Seed, H.B. and I.M. Idriss, 1971. Simplified Procedure for Evaluating soil Liquefaction Potential. Journal of the Soil Mechanics and Foundations Division, ASCE, 97(9): 1249-1273.
3. Toprak, S., 2003. Liquefaction Potential Index: Field Assessment", Journal of Geotechnical and Geoenvironmental Engineering. ASCE, 129(4): 315-322.
4. Haeri, S.M. and A. Hamidi, 2005. Steady State and Liquefaction Characteristics of Gravely Sands. Geotechnical and Geological Engineering, Springer Netherlands, 23(2): 141-156.
5. Hyde, Adrian F.L., T. Higuchi and K. Yasuhara, 2006. Liquefaction, Cyclic Mobility and Failure of Silt. Journal of Geotechnical and Geoenvironmetal Engineering, ASCE, 132(6): 716-735.
6. Marcuson, W.F., M.E. Hynes and A.G. Franklin, 2007. Seismic Design and Analysis of Embankment Dams: The State of Practice. In the Proceedings of 4th Civil Engineering Conference in the Asian Region, pp: 2-17.
7. Huang, M., X. Yu and Y. Huang, 2010. Soil Dynamics and Earthquake Engineering. In the Proceedings of the GeoShanghai International Conference, Shanghai, China, pp: 328-329.
8. Chwdhury, I. and S.P. Dasgupta, 2008. Dynamics of Structure and Foundation: a unified approach. CRC Press, pp: 50-155.

9. Boulanger, R.W. and I.M. Idriss, 2004. State Normalization of Penetration Resistance and the Effect of Overburden Stress on Liquefaction Resistance. In the Proceedings of 11th ICSDEE and 3rd ICEGE, Berkeley, pp: 484-491.
10. NCEER Summary Report, 2004 Liquefaction, Workshop Participants, USA.
11. Seed, H.B., K. Tokimatsu and L.F. Harder, 1984. The Influence of SPT Procedures in Evaluating Soil Liquefaction Resistance, Report:UCB/EERC-84-15, University of Berkeley, Berkeley, California.
12. Boulanger, R.W. and I.M. Idriss, 2004. Semi-empirical Procedures for Evaluating Liquefaction Potential During Earthquakes. In the Proceedings of 11th ICSDEE and 3rd ICEGE, Berkeley, pp: 32-56.
13. Bray, J.D. and R.B. Sancio, 2006. Assessment of the Liquefaction Susceptibility of Fine-grained soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 132(9): 1165-1177.
14. Phoon, K.K., 2008. Reliability Based Design in Geotechnical Engineering. Taylor and Francis UK Press, pp: 50-102.
15. Boulanger, R.W. and I.M. Idriss, 2005. New Criteria for Distinguishing between Silts and Clays Those are Susceptible to Liquefaction Versus Cyclic Failure. In the proceedings of 25th USSD Conference, Salt Lake City, pp: 357-366.
16. Seed, H.B., K.O. Cetin, R.E.S. Moss, A.M. Kammerer, J.M. Pestana, M.F. Riemer, R.B. Sancio, J.D. Bray, R.E. Kayen and A. Faris, 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework., ASCE, SPL-2, Los Angeles, Geotechnical, California, pp: 1-45.
17. Iran`s Ministry of Energy, 2000. East Azarbaijan`s Regional Water Corporation, Reports of Ministry of Energy.
18. Xenaki, V.C. and G.A. Athanasopoulos, 2008. Dynamic Properties and Liquefaction Resistance of Two Soil Materials in an Earthfill dam-Laboratory test results. Soil Dynamics and Earthquake Engineering, Elsevier, 28(8): 605-620.
19. Elgamal, A., Z.H. Yanga and E. Parra, 2002. Computational Modeling of Cyclic Mobility and Post-liquefaction Site Response. Soil Dynamics and Earthquake Engineering, Elsevier, 22(4): 259-271.
20. Pagano, L., S. Sica and A. Desideri, 2006. Representativeness of Measurements in the Enterpretation of Earth Dam Behavior. Canadian Geotechnical Journal, 43(1): 87-99.
21. Adalier, K. and M.K. Sharp, 2004. Embankment Dam on Liquefiable Foundation-Dynamic Behavior and Densification Remediation. Journal of Geotechnical and Geoenvionmetal Engineering, 130(11): 1214-1224.