14 Aug 2008, 4:30pm - 6:00pm

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INVESTIGATION ON THE LIQUEFACTION OF A CLAYEY-SANDY SOIL DURING CHANGUREH EARTHQUAKE

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ABSTRACT

An intense earthquake (MW = 6.4) occurred in western Iran, about 225 km west of Tehran at 7:28 local time, June 22, 2002. Surface soil in this area is mostly clay; however, clear traces of sand boiling, softening of soil, and consequent deformations were observed particularly in Hessar village. Some soil samples were prepared throughout an excavated pit from a depth of 2 m, the depth of the liquefied layer. The preliminary tests showed that the soil has a liquid limit of 38, a plasticity index of 18, and a < No. 200 fraction of 44%. These index characteristics would indicate a nonliquefiable soil according to the commonly used criteria. Analysis of cyclic triaxial test data suggests that the clayey sand deposit likely developed high residual excess pore pressures and significant shear strains during the earthquake and thus likely contributed to the observed lateral deformations. In this paper, different cases of observed liquefaction and consequent geotechnical phenomena are presented. Moreover, the results of laboratory tests on reconstituted samples are presented to prove how a soil with 44% of clay content could be liquefied.

INTRODUCTION

On June 22, 2002 at 7:28 local time, an intense earthquake hit a sparsely populated region of Qazvin province in northwest Iran, causing significant damage and casualties to small mountainous town and villages in the area. The earthquake - named the Changureh (Avaj) earthquake- was felt as far as Tehran, which is approximately 225 km east of the epicenter. Though the moderate moment magnitude of 6.4 (ERI, University of Tokyo) – 6.5 (USGS) calculated for this earthquake was not surprisingly large as contrasted with those major earthquakes that ever occurred in the country, seriously ravaged villages were found along east-west oriented valley in the west of Abegarm, and 261 people were reportedly killed and 1,300 injured. In addition, over 25000 people are estimated to be homeless as a result of this earthquake. Figure 1 shows the map of the region with the epicenter and major faults located.

LIQUEFACTION

Hesar, about 5km north of Abdarreh, lies in the middle of a valley of about 15km wide extending in NW-SE direction. Liquefaction took place in a flat area of a little grassy land about 2 km northwest of Hesar (Fig.2a). Figure 2b shows the traces of liquefied sand. The bunch of cracks appearing across the area seems to be winding along a small river trace, which is dried up in hot weather. Soils along the crack at Point A are covered 5-10 cm thick with fine sand (about 0.5 mm in diameter, Fig.2a), while gravels were found at Point B (Fig.3). The flat mass of gravel stopped there was cut upright (Fig. 4) to discuss its possible sedimentation process. The bottom part was full of middle size grains of about 3-5 mm diameter, which was then covered thick with finer sand. And lastly, a number of bigger grains (5-15 mm) were found all over the sand mass. This fact suggests that the liquefied matter was forcibly spouted twice (Konagai and Ghalandarzadeh, 2003)

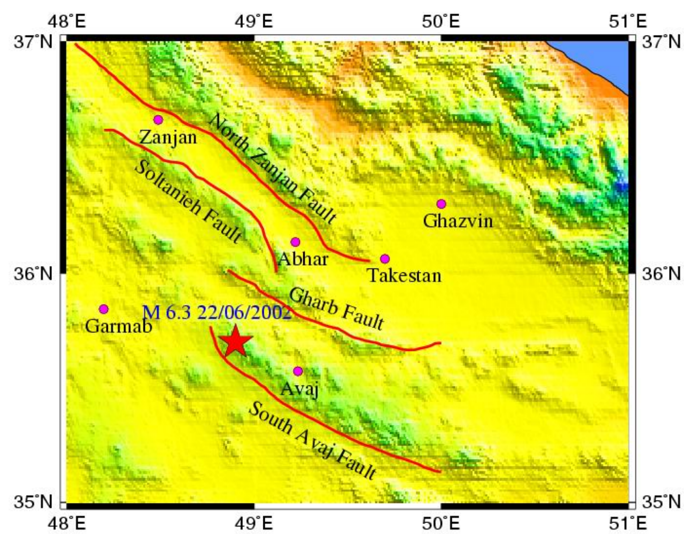
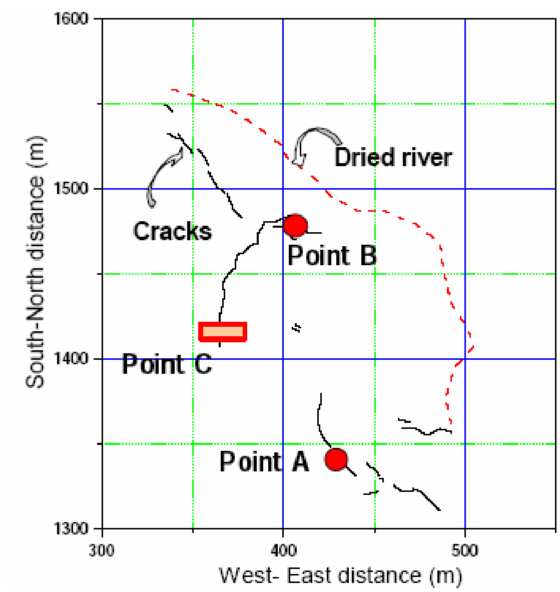


Figure 1. Site map showing the epicenter of the earthquake and faults around the affected area (Mansoori et al, 2002)

A trench was excavated at Point C, where the surface soil was covered thick with fine sand (0.5 mm in diameter). Several cracks were found almost upright on a stiff clay wall of the trench through which liquefied sands forced their ways up (Fig.5). There were several cracks appearing upright on the clay wall of the trench, through which liquefied sands forced their ways up. The clay layer was 2m thick lying over the completely wet sand layer.

(a)



(b)

Figure 2. (a) Liquefied area at point A , (b) Crack mat at the liquefied area, Hesar

Figure 3. Liquefied gravel at point B

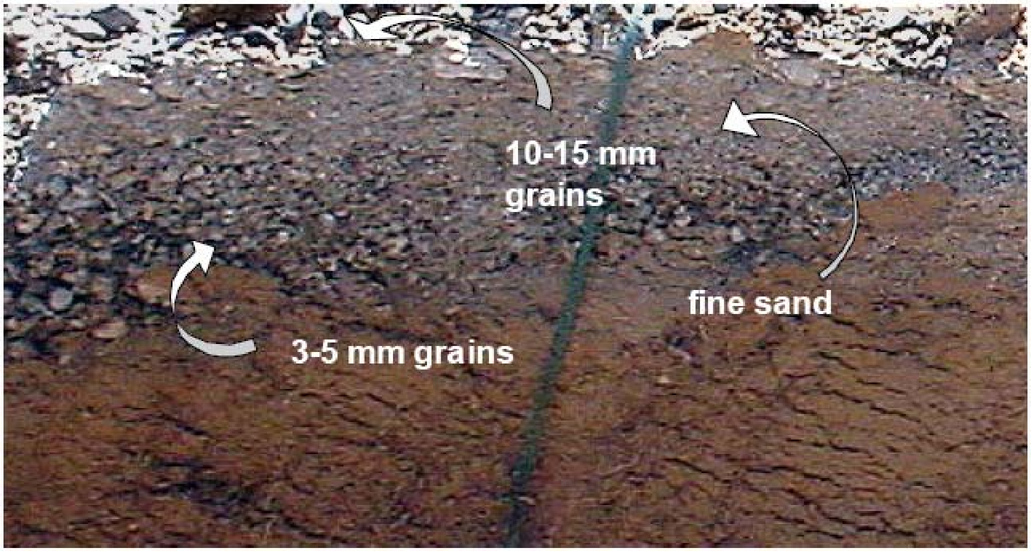


Figure 4. Cross section of flat mass of liquefied matters at point B

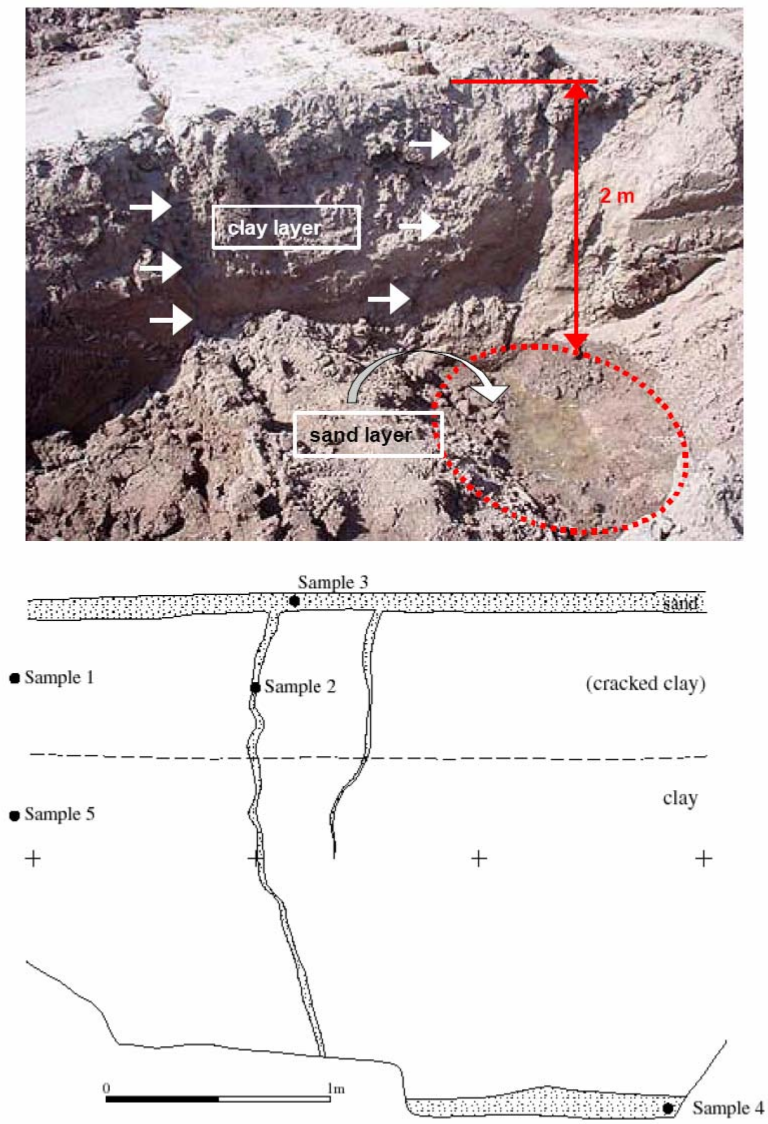


Figure 5. Liquefied sand channels through clay layer at point C

DAMAGE TO EARTH DAM AND NEARBY GROUND

A small size earth dam, located at northwest of Abdarreh village experienced the strong ground motion caused by Changureh earthquake (Fig. 6). The height of dam is approximately 10 meter. The body of this dam consists of clayey soil with no core section constructed inside. The upstream side is covered by rock fill material. Approximate cross section and layout of the dam is shown in Figure 7.
As can be seen in figure 6 the dam reservoir was not full at the time of earthquake. Therefore, the body of dam was not saturated. In contrast, the ground at upstream and downstream sides seems to be wet and probably saturated due to the certain level of water in the reservoir. This condition can well explain the damage pattern of the mentioned dam and adjacent ground. The instabilities appeared in the form of some longitudinal cracks indicating failures of upstream and downstream slopes of the dam. Some minor cracks which cross the body at both end of the dam were also appeared. Figure 7 shows the distribution of these cracks. Figures 7 to 13 show the cracks at both sides of dam and at its two ends.

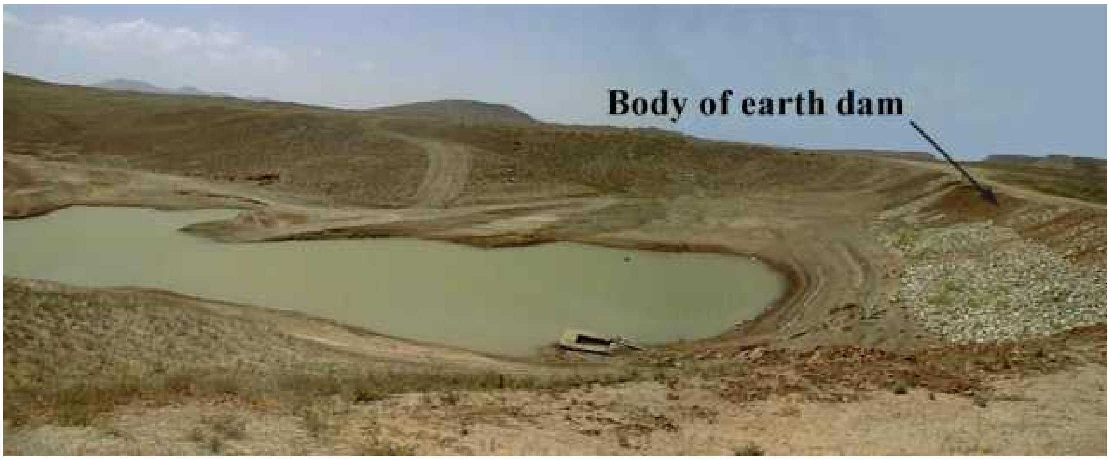


Figure 6. General view of the small earth dam located northwest of Abdarreh village

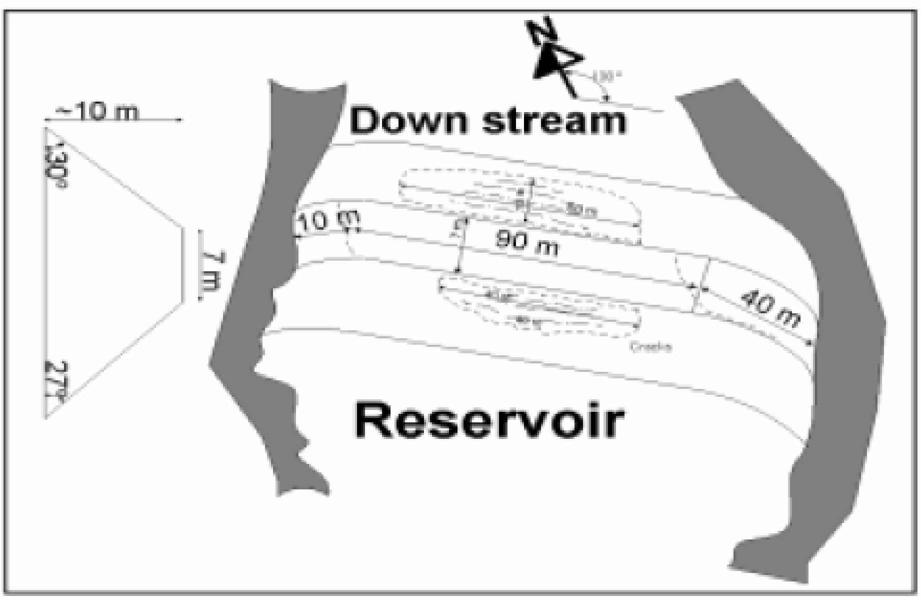


Figure 7. Layout of the earth dam and the crack distribution on its body

By measuring and summation of the width of all longitudinal cracks the total permanent displacement of both upstream and downstream slopes were estimated to be 56 cm and 87 cm, respectively. These values represent the deformation of the displaced ground, whereas the horizontal permanent displacements were 10 cm and 30 cm for upstream and down stream slopes as shown in figure 8. The size of dislocated zone at downstream side is almost 60 m in length and 10 m in width (Fig.7). To estimate the maximum acceleration at the position of the dam a simple analysis based on Seed-Makdisi method is conducted. By measurement of internal friction angle of almost 30o and cohesion of ~25 KPa, using remolded soil samples obtained from the body of dam and selecting an average shear wave velocity of 400 m/sec, the maximum acceleration is approximated to be 0.16g at the ground level and 0.58g at the crest of dam which looks to be reasonable.

Two other types of failures are observed at two different points in the riverbank of both upstream and downstream sides. The large deformation caused by earthquake at the left bank of river in the downstream part is shown in Figure 12. A kind of slumping is occurred probably due to softening of the soil caused by reduction of effective stress. The evidence for lateral movement is the clear displacement of trees, which were located at the feet of the bank as is shown in Figure 13. Figure 14 illustrates another failure, somehow similar to that explained above, at the left river bank in upstream side. As it can be seen in that the soil was wet and probably saturated. The cause for the lateral deformation could be the softening of soil due to reduction of effective stress.

(a) *(b)*

Figure 8. Instabilities at (a) upstream slope (b)down stream slope

Figure 9. Instabilities at upstream slope from a closer view

Figure 10. Cracks crossing the body of dam at its left abutment

Figure 11. Cracks crossing the body of dam at its left abutment

Figure 12. Slumping of the left bank of river at upstream side

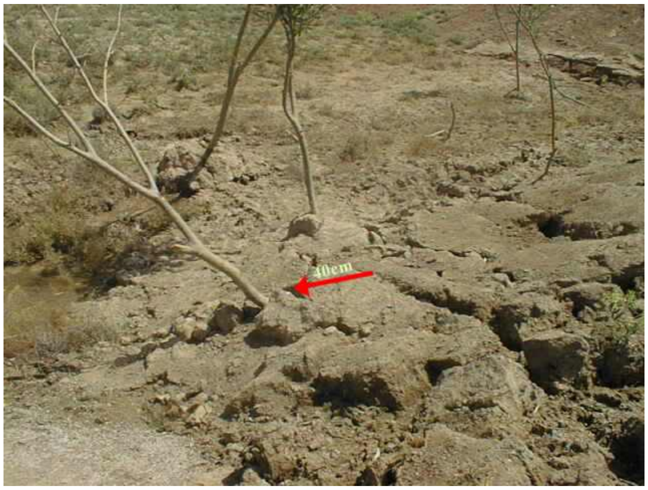


Figure 13. Movement of trees indicate lateral displacement of trees

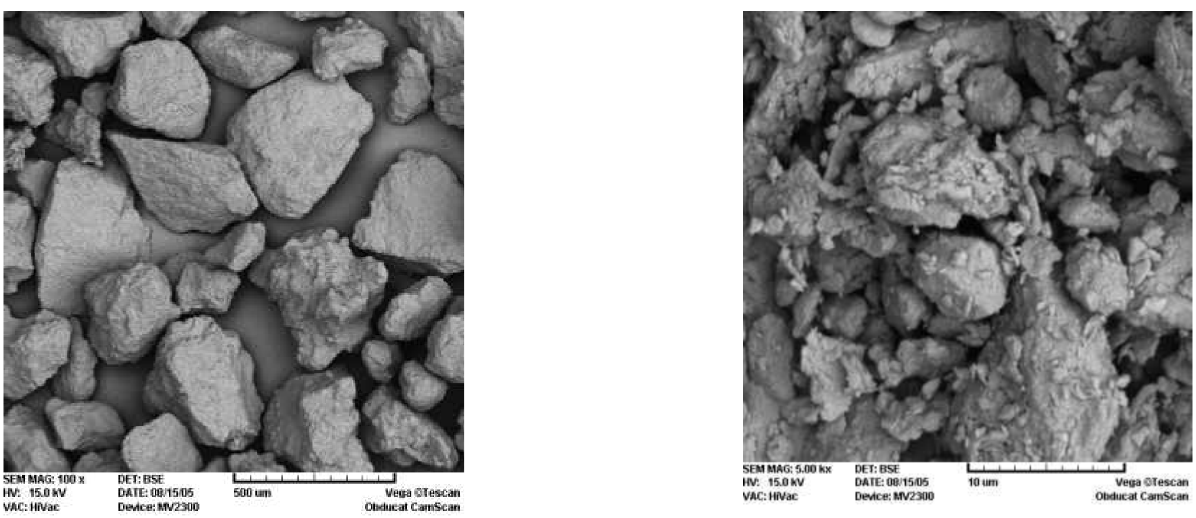
Figure 14. Lateral movement of left bank at upstream

GRAIN SIZE DISTRIBUTION AND INDEX TESTS

Laboratory tests on samples from the liquefied clayey sand included gradation, index test and undrained cyclic triaxial tests in compression-extension mode. An amount of soil (sample 4 in figure 5) was sent to the laboratory for further investigations. The preliminary tests showed that the soil has a fine content (<0.075 mm) fraction of 44% mixed with subangular sand grains. Index tests revealed that the fines have a liquid limit of 38 and a plasticity index of 18. Figures 15a and 15b show the SEM image of clean sand and fines, respectively. Figure 16 presents the grain size distribution curve of clean sand and natural soil.

CYCLIC TRIAXIAL TESTS

Due to some limitations in the field, it was not possible to take undisturbed sample from the site. Therefore, all cyclic triaxial tests were performed on reconstituted samples. Specimens were prepared by placing dried soil in a funnel that had a tube attached to the spout. The tube was placed in the bottom of a split mold and was then slowly raised along the centerline of the specimen. The diameter of specimen was measured after a slight vacuum was applied and the mold was removed. Both the diameter and height of the specimen was measured to the nearest 0.01 mm. The diameter was determined at three locations (top, middle and bottom) to ensure the specimen's quality which had a diameter of 38 mm and a height of 76 mm.



(a) *(b)*

Figure 15. SEM image of (a) clean sand (b) fines of Changureh soil

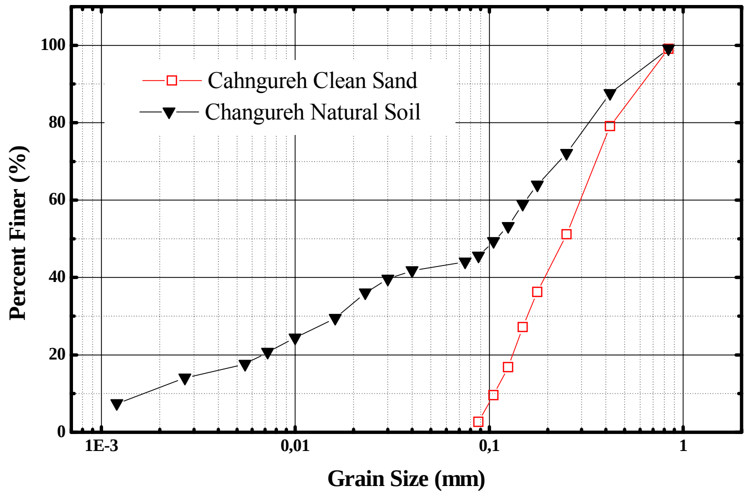


Figure 16. Grain size distribution of clean sand and natural soil of Changureh

Saturation was performed by purging the dry specimen with Co_2 for approximately 30 minutes. De-aired water was then introduced into the specimen from the bottom drain line. B value measurements were made before cyclic loading and a B value greater that 0.96 was achieved in all cases, indicating satisfactory saturation. The saturated samples were consolidated isotropically with an effective confining stress of 200 kPa. Six isotropically consolidated, undrained tiaxial tests were performed on reconstituted samples. Undrained cyclic tests were run using uniform cyclic loads in compression-extension mode at a frequency of 0.05 Hz for three tests, and 0.01 Hz for the other ones. Because of the inability of the water pressures to equilibrate throughout the sample and measurement system, such low frequencies were selected. A loading rate of 0.01 Hz was used in order to investigate the effect of the loading rate on pore water pressure and cyclic strength. The characteristics of tests as well as the test conditions and results for all tests are summarized in Table 1.

Table 1. Test conditions and results for cyclic triaxial tests

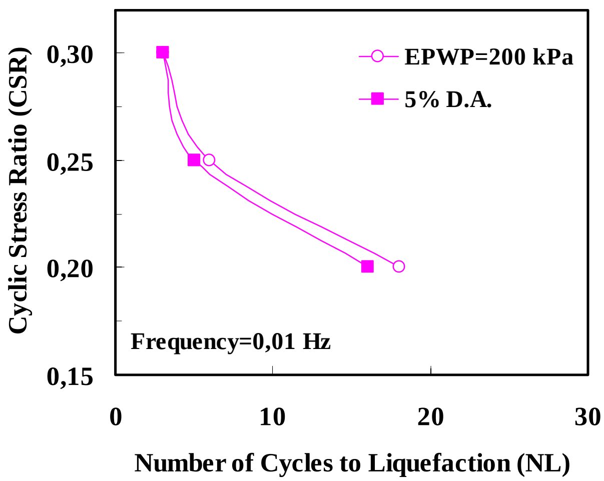
Test No.	CSR	Frequency	e	N_L (ε = 5%)	N_L (P'=0)
1	0.20	0.05	0.70	148	154
2	0.25	0.05	0.67	57	63
3	0.30	0.05	0.68	7	9
4	0.20	0.01	0.72	16	17
5	0.25	0.01	0.71	5	6
6	0.30	0.01	0.69	3	3

RESULTS AND DISCUSSION

The cyclic stress ratio versus the number of cycles to initial liquefaction ($u=\sigma_3$) and 5% double amplitude axial strain for both frequencies is plotted in Figure 17. The cyclic stress ratio was defined as CSR=$\Delta\sigma_d/2\sigma_{3c}'$, where $\Delta\sigma_d$ is the deviatoric load and σ_{3c}' is the isotropic consolidation stress. It is clearly seen that the failure defined by 5% axial strain occurs at relatively smaller cycles than those required to develop a state of initial liquefaction (EPWP=200kPa) for both frequencies. Therefore, it seems that criterion of 5% double-amplitude axial strain is more convenient than initial liquefaction to define liquefaction of soils with high fine contents in cyclic triaxial tests, as reliable pore pressure measurements are difficult in cyclic testing with relatively low permeabilities as stated by El Horsi et al. (1984) and Das et al (1999). Figure 18 presents the SEM image of the specimen. It is clearly seen that most of the sand grains have been fully confined by fine particles; hence, the specimen can be taken as a fine dominated one. It can be concluded that in the fine dominated soils, it is better not to rely on pore pressure measurements, because: 1) The conventional cyclic triaxial test apparatuses measure the water pressure at the top and bottom of the specimen; whereas, it is known that the water pressure cannot equilibrate throughout the sample and measurement system. 2) The specimen becomes more ductile when fine particles have separated sand grains. Therefore, cyclic failure in the form of cyclic deformation can be more likely to occur. The same behavior was observed for sand with high kaolin content (Ghahremani, 2005)

EFFECT OF FREQUENCY OF LOADING

The effect of frequency of loading on the cyclic resistance of mixture is shown in figure 19. It is demonstrated that by decreasing the frequency of loading, resulted cyclic strength is also decreased. This is in line with findings of Zergoun and Zaid (1994) who demonstrated that pore pressure measurements on clay samples can only reliably be obtained in slow cyclic tests and found that slow cyclic test resulted in lower cyclic strengths that fast ones.



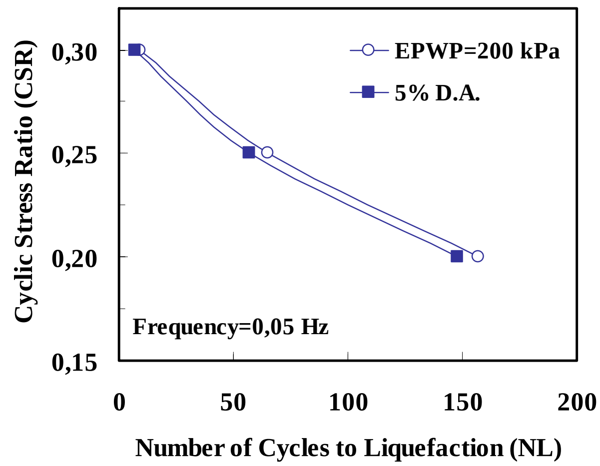


Figure 17. Comparison between number of cycles to initial liquefaction and number of cycles to 5% D.A. for (a) frequency of 0.01 Hz (b) frequency of 0.05 Hz

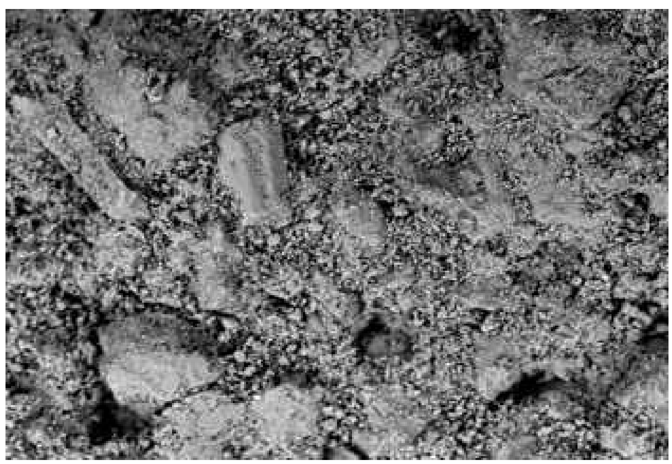


Figure 18. SEM image of natural soil of Changureh

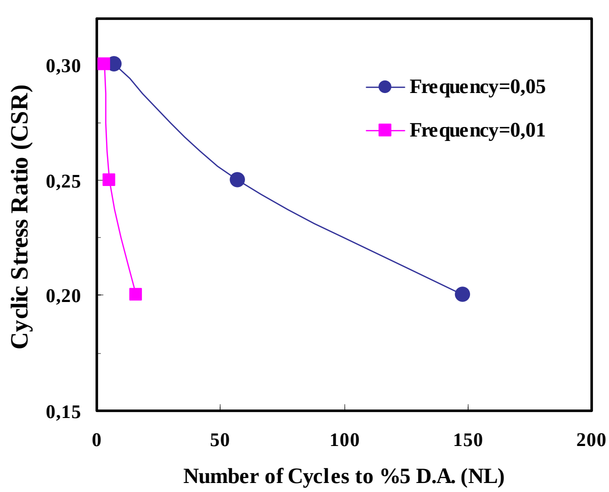


Figure 19. Effect of frequency of loading on cyclic resistance of mixture

CONCLUSION

The paper presented the reconnaissance report of Changureh earthquake including some geotechnical hazards such as liquefaction. The main focus of the paper was on the liquefaction of a clayey sandy soil which was assumed to be nonliquefiable according to commonly used criteria. It contained 44% plastic fines and yet liquefied during a moderate earthquake. The results of laboratory tests were in line with observed behavior in the field and suggest that the clayey sand deposit likely developed high residual excess pore pressures and significant shear strains during the earthquake and thus likely contributed to the observed deformations. Moreover, it was found that when dealing with soils with high fine content, it is better to define liquefaction criterion as 5% double amplitude axial strain rather than state of initial liquefaction ($u=\sigma_3$). The effect of frequency of loading on cyclic resistance of soil was also studied. The results revealed that by decreasing the loading rate, the cyclic resistance is also decreased.

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