Results of large-scale cyclic triaxial tests on the mixed material of Rudbar dam core

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ABSTRACT
Earth dams may be subjected to low-period, high frequency loads such those from earthquakes. Clayey gravel soils often used in the core of such dams are difficult to test due to the large size of the gravel particles since representative samples of such soils contain grains that exceed the maximum size allowable in conventional triaxial apparatus. Due to the higher cost of large-scale triaxial apparatus and the substantially longer time required for saturation and testing of large samples, smaller samples are often tested to determine the behavior of such soils. In the current study, results from large-scale triaxial tests on the mixed material of the core of an earth are presented. Saturation of each sample took more than one month, and was followed by consolidation and undrained cyclic loading. Degradation of shear modulus and change in damping ratio obtained from conventional size triaxial tests and resonant column tests on the same material are also compared with results of large scale tests.

RÉSUMÉ
Les barrages terrestres peuvent être soumis à des charges à haute fréquence de basse période, telles que celles causées par les tremblements de terre. Les sols de gravier argileux souvent utilisés dans le cœur de tels barrages sont difficiles à tester en raison de la grande taille des particules de gravier puisque des échantillons représentatifs de ces sols contiennent des grains qui dépassent la taille maximale autorisée dans un appareil triaxial conventionnel. En raison du coût plus élevé de l'appareil triaxial à grande échelle et du temps considérablement plus long requis pour la saturation et l'essai de grands échantillons, des échantillons plus petits sont souvent testés pour déterminer le comportement de ces sols. Dans la présente étude, les résultats d'essais triaxiaux à grande échelle sur le matériau mélangé du noyau d'une terre sont présentés. La saturation de chaque échantillon a pris plus d'un mois et a été suivie d'une consolidation et d'une charge cyclique non drainée. La dégradation du module de cisaillement et la variation du taux d'amortissement obtenues à partir des essais triaxiaux de taille conventionnelle et des essais de colonne résonnante sur le même matériau sont également comparées aux résultats des essais à grande échelle.

1 INTRODUCTION

The core of the non-homogeneous Rudbar Lorestan earth dam is constructed using mixed clayey gravel. Height of this dam is 155 meters from the rock foundation, and its crest is 185 meters long. The dam is built on the Rudbar river, a branch of the East Dez river, and is located about 100 km south of the city of Aligoodarz in the Lorestan province west of Iran.

Clayey gravel used in the dam core can help reduce leakage and pore pressure, and provide favourable mechanical properties in order to maintain stability during different loading conditions especially those due to earthquakes. Clayey gravel soils are composed of clay particles mixed with granular material, and their mechanical behaviour is something between clay and granular soils. The effects of each of these two components on the behaviour of the mixed soil depend on the behaviour of each component. Granular soils have high permeability and shear resistance under monotonic loading, but the probability of their liquefaction under quick monotonic loading and cyclic loading is high. Clayey soils have lower resistance and permeability and higher compressibility, while their potential for liquefaction is relatively smaller. Moreover, the possibility of cracking, and the probability of arching in dry mixed soils are lower than those of cohesive and granular soils, respectively (Sassanian et al., 2009).

Since gravelly soils often cannot be tested in regular size triaxial apparatuses, and considering the importance of studying dam behaviour under seismic loads, large size cyclic triaxial tests were conducted on the mixed material of the Rudbar dam core, and their behaviour under seismic loading was investigated. Results obtained from regular size triaxial tests and resonant column tests on the same material are also presented and compared with those obtained from large-scale test.

2 THE MATERIAL TESTED

Grain size distribution curve of the clayey gravel material tested is shown in Figure 1, and it consisted of 68% coarse aggregate (angular limestone gravel and sand) and 32% clayey fines with a plasticity index of approximately 17. Considering the sample diameter of 200 mm, maximum size of the soil grain was limited to 38 mm, which is less than one-fifth of the sample diameter. Maximum dry density and optimum moisture content of the soil were 23 kN/m^3 and 6.5%, respectively, as obtained from the modified AASHTO compaction test (ASTM D1557). Test samples were prepared at optimum moisture content and were compacted to 95 percent relative compaction (i.e. unit weight of 21.85 kN/m3).

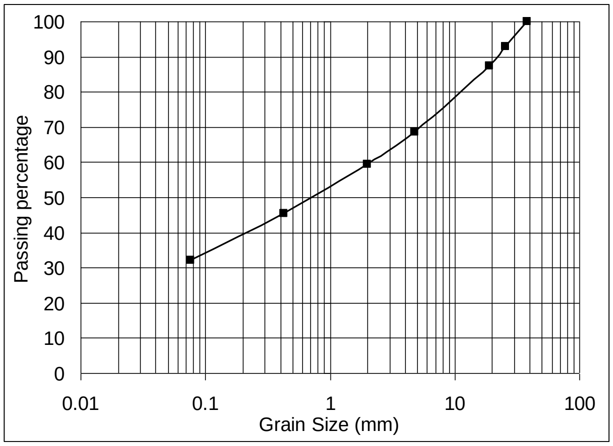


Figure 1. Grain size distribution of the soil tested

3 TEST EQUIPMENT

The experiments were conducted using an advanced, large-scale cyclic triaxial testing apparatus manufactured by SEIKEN, Japan available in the soil mechanics laboratory of the Department of Geotechnics of the Building and Housing Research Centre (BHRC) in Tehran, Iran. The device is able to perform static and dynamic tests on 200 and 300 mm diameter soil, and 50 mm diameter soft rock samples. The device is comprised of various parts including triaxial chamber, vertical loading frame, hydraulic and pneumatic pumps, electronic digital control unit, and data acquisition computer system. The servo-hydraulic system of the device is capable of applying up to 300±200 kN of vertical load. A loading system with a capacity of 500 kN is used to measure the vertical load applied to the sample. The maximum confining pressure applicable in the triaxial chamber used for soil samples is 2 MPa, and for the triaxial chamber used for rock samples is 20 MPa, and sensors having these capacities are used to measure these pressures. Changes in sample length are measured using a100 mm stroke LVDT placed on top of the triaxial chamber, and two 25 mm stroke magnetic sensors installed inside the triaxial chamber, above the sample. Pore water pressure in the soil sample is measured using a 1 MPa capacity sensor. Changes in sample volume are measured by a sensor with a capacity of 4 litres.

Tests can be conducted in both stress controlled and strain controlled conditions, and any desired stress path in the triaxial plane can be applied to the sample. Cyclic load waveform can be sinusoidal, rectangular or triangular, and frequencies up to 100 Hz can be applied. Figure 2 shows the large-diameter cyclic triaxial testing apparatus.

Samples of the same material but with 70 mm diameter and smaller maximum size particle (12 mm) were also tested using regular size conventional triaxial testing apparatus, equipped with resonant column testing shown in Figure 3. These cyclic triaxial and resonant column tests were performed at the International Institute for Earthquake Engineering and Seismology (IIEES) in Tehran, Iran.

Figure 2. Large-diameter cyclic triaxial device in the Soil Dynamics Laboratory of BHRC

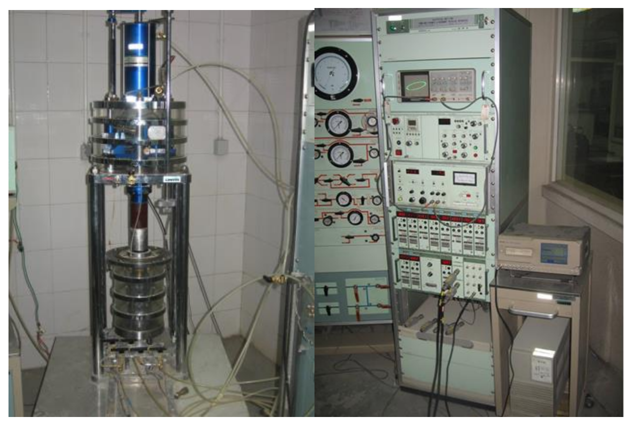


Figure 3. Regular size cyclic triaxial testing device equipped with resonant column testing facilities

4 SAMPLE PREPARATION AND TESTING

4.1 Sample preparation

Large-scale cyclic triaxial tests were carried out on four samples of the core mixed material of Rudbar Lorestan dam. Cylindrical reconstituted samples having 200 mm diameter and 400 mm height were tested in the soil mechanics laboratory of the Department of Geotechnics of the BHRC. All samples were reconstituted in the laboratory, and the required grain size distribution was achieved by mixing ratios of different grain sizes according to the desired gradation curve. Sample compaction was carried out according to the ASTM D1557 standard using an electric compactor until a dry density of 21.85 kN/m3, corresponding to 95 percent relative compaction was achieved.

Figure 4. Large-scale sample before saturation

4.2 Sample saturation and consolidation

Saturation of large-scale samples is one of the most important stages of testing such samples. Purpose of the saturation phase is the filling of all voids in the sample by water without inducing any pre-stresses or volume changes in the sample. For this purpose, a vacuum pressure of 80 to 90 kPa was first applied to the sample for at least 24 hours in order to evacuate its void air. De-air water was then introduced under very low head into the sample from its bottom such that all the voids are filled slowly with water. When the water flow was established along the sample and reached its top, the vacuum was removed and the sample was placed under pressure from its top and down in order to dissolve any small air bubbles in the water. Using this procedure, all the samples were saturated such that a target B value of 95% was achieved. Despite the significant percentage of coarse material compared to the fine content that existed in the samples (68% coarse-grained versus 32% fine-grained), the saturation time needed to reach the desired B value was more than a month for all the samples tested.

After the saturation stage was completed, and while the sample drainage valves were kept close and the back pressure under which the sample had reached saturation was kept unchanged, the cell pressure was increased until the difference between the cell pressure and back pressure was equal to the desired effective consolidation pressure. The sample drainage valves were then opened and the sample was left to consolidate over time. At this stage, the changes in volume were initially recorded every two seconds, and then every 1.5 minutes using the computerized data acquisition system (Taban, 2014). A plot showing the variation of volume change with time during consolidation of one of the samples is shown in Figure 5.

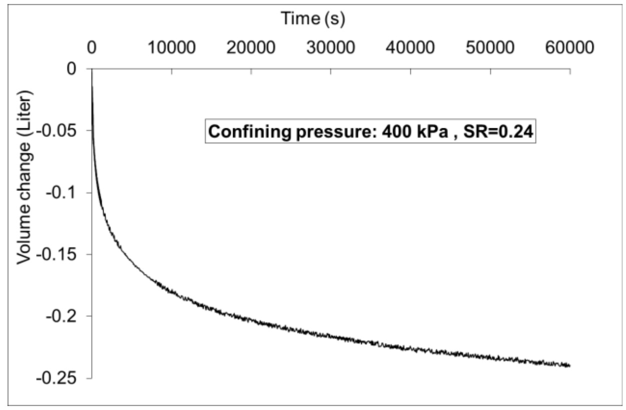


Figure 5. Volume change with time during saturation

4.3 Application of the cyclic load

Following completion of the sample consolidation, cyclic loading was initiated by cyclically changing the deviatoric stress, using the procedure described in the ASTM D5311 standard for application of cyclic loading in triaxial tests.

Samples consolidated under confining pressures varying from 200 to 400 kPa (2 to 4 kg/cm^2) were subjected to cyclic stresses in which stress ratio (SR), defined as half of the ratio of cyclic deviatoric stress to confining pressure, varied from 0.18 to 0.30 as shown in Table 1. In the two tests with confining pressures of 200 kPa, the SR remained constant throughout the test, while in the two tests with confining pressure of 400 kPa, the SR was higher at the beginning of loading, and it gradually decreased afterwards. A one cycle per second (1 Hz) frequency and a total of 500 cycles were used in all the tests.

Table 1. Combinations of confining pressures and stress ratios used in the cyclic triaxial tests

Test No.	confining pressure (kPa)	Stress Ratio (SR)
1	200	0.18
2	200	0.30
3	400	0.24-0.12
4	400	0.30-0.12

4.4 Regular size triaxial and resonant column tests

Regular size triaxial tests were carried out using the standard procedure as described in the ASTM D5311 standard and the resonant column tests were conducted according to the ASTM D4015-92 (2000) standard. Confinig pressures of 200, 400, 600 and 800 kPa were used to examine effects of confining pressures on the soil dynamic properties.

5 TEST RESULTS

5.1 Stress-strain behaviour of large-size samples

Figure 6 shows variations of deviatoric stress with axial strain for the test on large-size sample under confining pressure of 200 kPa and stress ratio of 0.3. Cyclic stress-strain loops are shown for each 100 cycle separately in order to allow examination and comparison of the behaviours in various stages of loading. As shown in the figure, as the number of cycles increase, the stress-strain loop moves to the right (i.e. the shear strain increases) and becomes more inclined from the vertical, indicating a decrease in the shear modulus. Moreover, the first 100 cycles shown in Figure 6(a) cause larger strain compared to the subsequent 100 cycles shown in Figures 6(b) to (e).

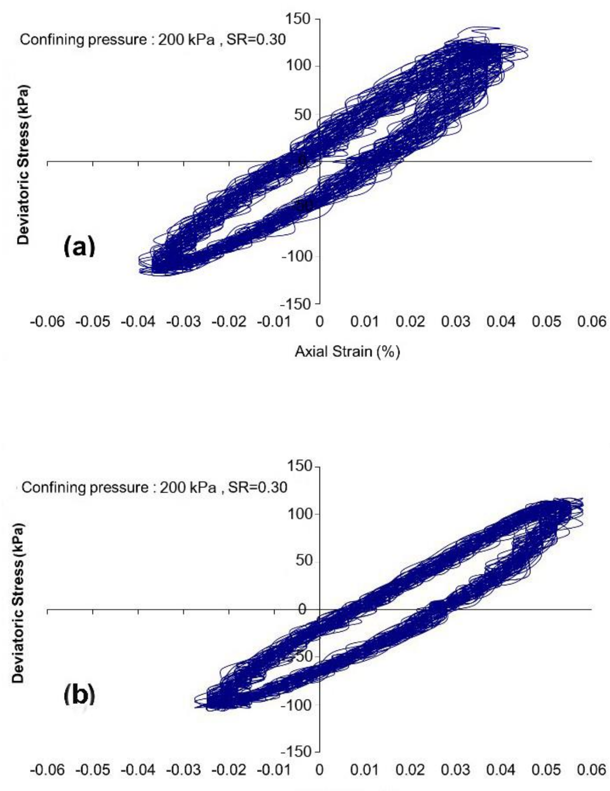


Figure 6. Stress-strain curves in test with 200 kPa confining pressure and SR= 0.3 in cycles: (a) 1 to 100 (b) 400 to 500

Similar behaviour is generally observed in Figure 7, in which a consolidation pressure of 400 kPa is used and a cyclic stress ratio of 0.24 is initially applied and then gradually decreased to 0.12, half of the initial value. However, the change in strain and increase in inclination of the stress-strain loop is significantly higher in the initial 100 cycles shown in Figure 7(a) compared to the subsequent 100 cycles shown in Figures 7(b) to (e). This may be attributed to the higher stress ratios applied in the initial cycles compared to the subsequent cycles, and shows the significant effect of cyclic stress ratio on the generation of shear strains and degradation of the shear modulus.

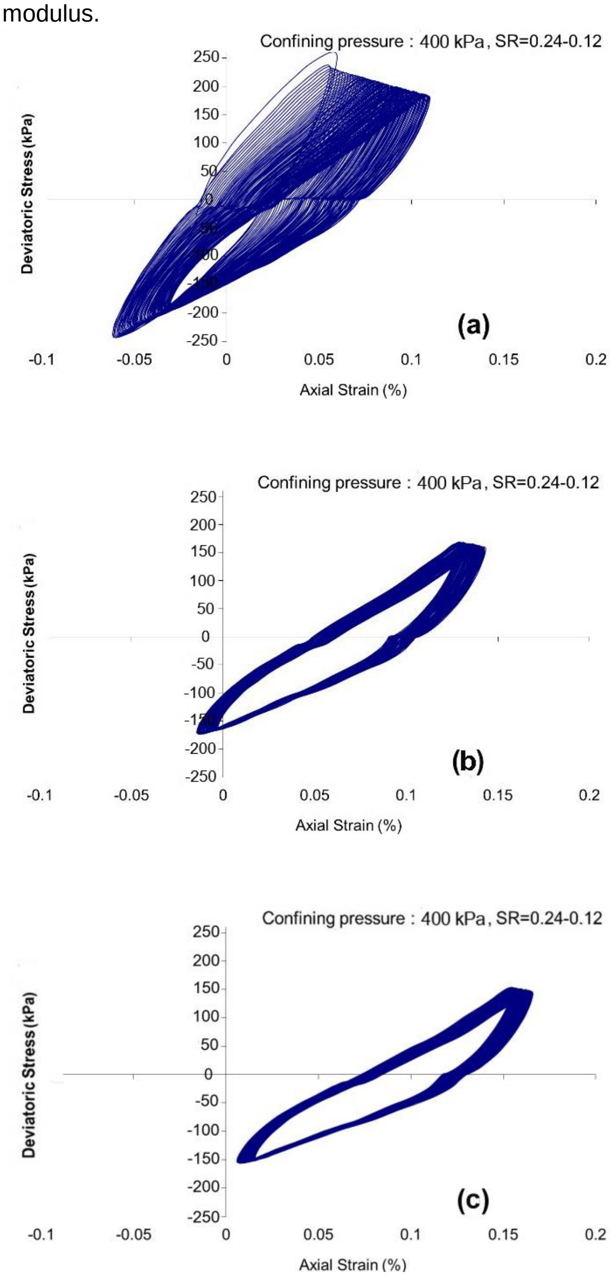
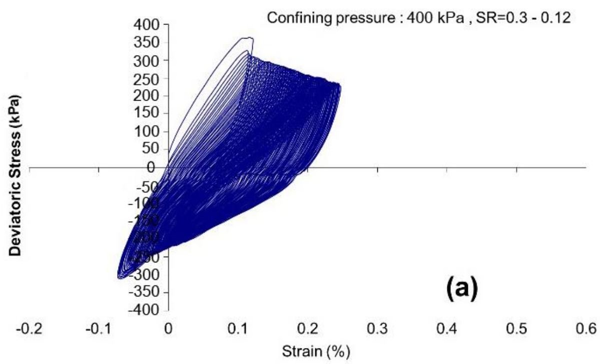


Figure 7. Stress-strain curves in test with confining stress of 400 kPa and SR= 0.24 to 0.12 in cycles: (a) 1 to 100 (b) 200 to 300 (c) 400 to 500.

Comparison of results shown in Figure 8, in which a confining pressure of 400 kPa and stress ratios ranging from 0.3 to 0.12 are applied, with those shown in Figure 7 indicate that although the same confining pressure was used in both tests, the increase in initial SR from 0.24 to 0.3 has resulted in a significant increase in the axial strain during the first 100 cycles. Based on Figure 7(a), the maximum axial strain has reached 0.11% after the first 100 cycles, whereas Figure 8(a) shows that more than double this value has occurred under the same number of cycles as a result of the increase in the initial stress ratio from 0.24 to 0.3. Moreover, higher shear strains are also generated in the second 100 cycles under higher stress ratio as can be noticed from Figure 8(b).



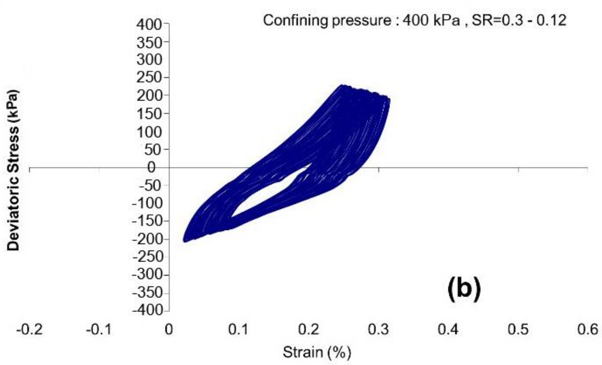
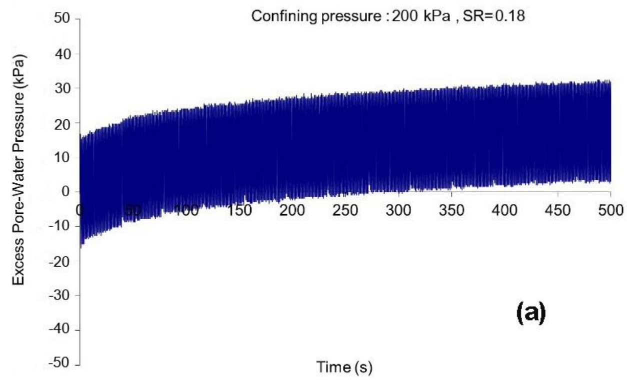


Figure 8. Stress-strain curves in test with confining stress of 400 kPa and SR= 0.3 to 0.12 in cycles: (a) 1 to 100, (b) 100 to 200

5.2 Pore pressures in large-scale samples

Figure 9 shows that during application of the cyclic loading to the two samples consolidated at 200 kPa confining pressure, the pore water pressure gradually increased, but the amount of increase is higher for the test with greater stress ratio. As the number of loading and unloading cycles increases, the pore water pressure gradually increases during the first approximately 100 cycles, after which the pore water pressure remains almost constant.



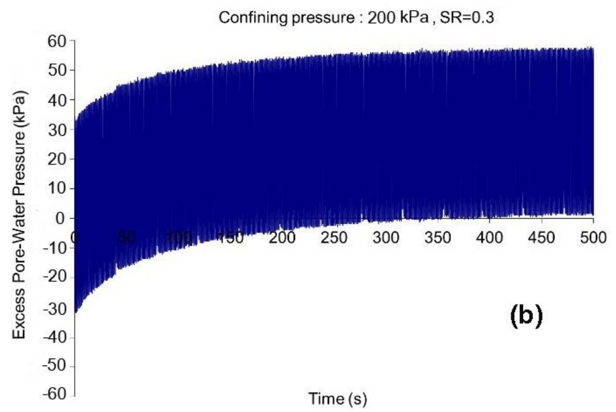


Figure 9. Increase in excess pore pressure with time for the sample with confining stress of 200 kPa and stress ratio of (a) 0.18 and (b) 0.3

Figure 10 shows similar results for the two samples consolidated at 400 kPa. However, while the total pore pressure increase is significantly higher than the samples consolidated to 200 kPa, fluctuations in the pore water pressure are relatively smaller than those consolidated to the lower pressure. In these tests, as in the previous tests, pore water pressure increased during the first approximately 100 cycles, after which the pore water pressure remained almost constant. In the test with lower stress ratio, pore pressure increased monotonically until the end of the test. However, in the test with higher stress ratio, excess pore pressure tended to decrease at later stages of the test, likely due to a tendency for material dilation at higher stress ratios, and due to reaching the stress ratio known as the phase transformation, at which soils start exhibiting dilative behavior after behaving contractively.

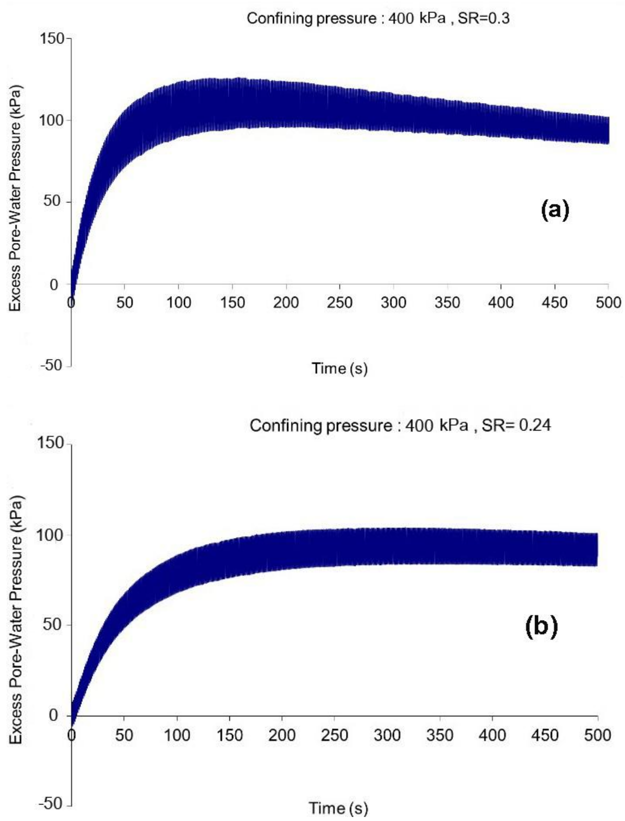


Figure 10. Increase in excess pore pressure in the samples consolidated at 400 kPa and (a) SR=0.3 and (b) SR=0.24

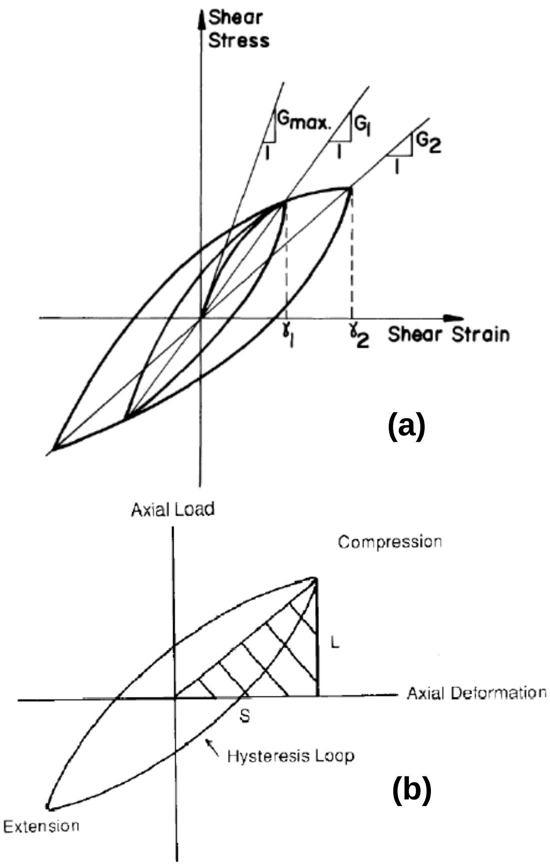


Figure 11. Determination of (a) shear moduli and (b) damping ratios from cyclic triaxial test results

5.3 Shear Moduli and damping ratios

To evaluate changes in shear moduli and damping ratios during cyclic tests, the secant shear moduli were determined as shown in Figure 11(a) and damping ratios using the following relationship:

$$D = \frac{A_L}{4\pi A_T} \times 100$$

in which AL is the area inside the stress-strain loop, and AT is the shaded area shown in Figure 11(b). An example showing the determination of shear modulus and damping ratio based on one of the stress-strain cycles measured in one of the cyclic triaxial tests is shown in Figure 12.

Using the aforementioned procedure, values of shear modulus were determined based on test results from 400 kPa confining pressure and stress ratios starting from 0.3 and 0.24 and variations of shear modulus with the number of cycles are shown in Figure 13. It can be seen that the shear modulus generally decreases with the increase in the number of cycles and reaches an almost constant value after the number of cycles reaches a certain value, approximately 200 cycles in this case. Moreover, at the higher cyclic stress ratio, decrease in the shear modulus with the number of cycles is somewhat higher.

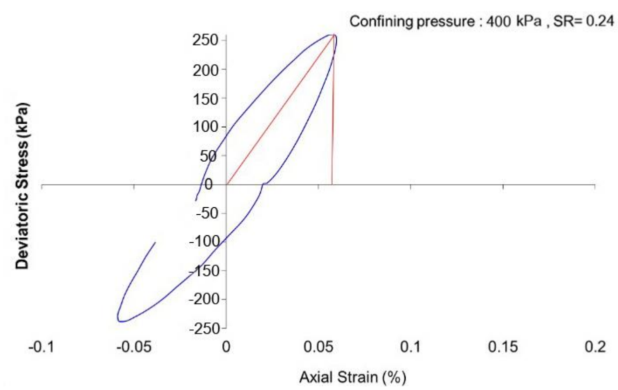


Figure 12. An example of determination of shear modulus and damping ratios from one of the stress-strain cycles recorded in a cyclic triaxial test

Degradation of shear moduli and changes in damping ratio with shear strain obtained from regular-size cyclic triaxial tests carried out at confining pressures of 200, 500, 600 and 800 kPa are shown in Figure 14. This figure shows that as the confining pressure increases, the degradation in shear moduli and the damping ratios decrease for the mixed clayey gravel soil tested in the current study.

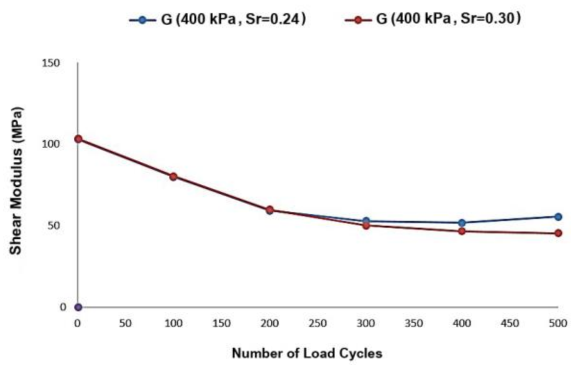


Figure 13. Variations of shear modulus with the number of cycles obtained from test results with 400 kPa confining pressure and SR= 0.3 and 0.24.

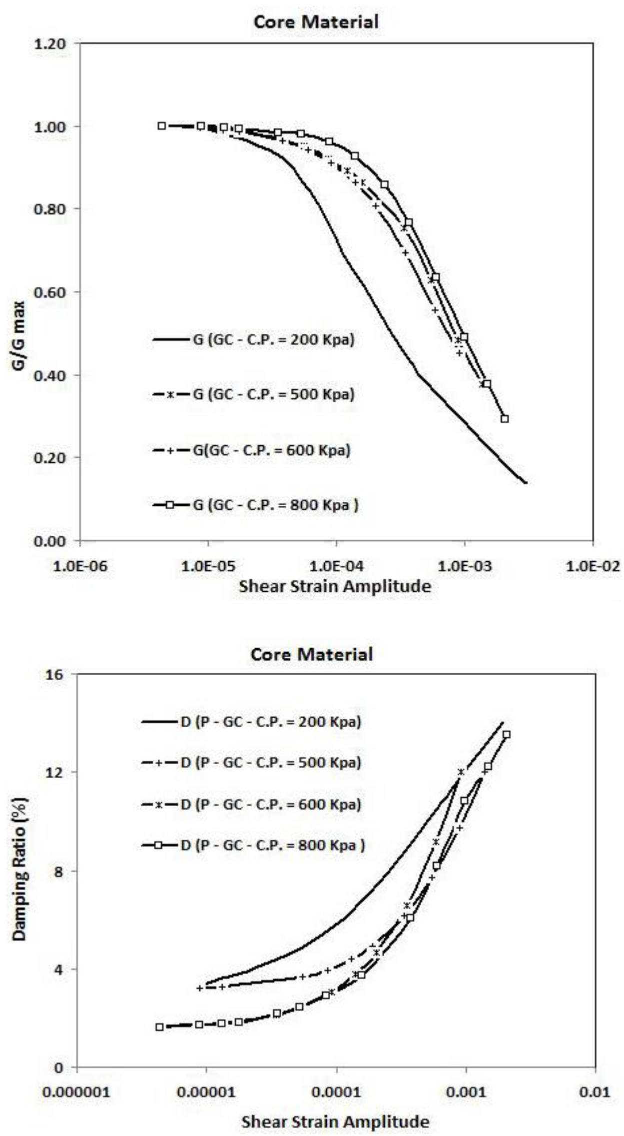


Figure 14. Shear moduli and damping ratios obtained from regular size cyclic triaxial tests

Since the lowest values of shear strain that can be measured accurately in cyclic triaxial tests are limited, resonant column tests were also conducted on the clayey gravel material of the dam core in order to obtain the extension of the shear modulus degradation and the damping ratio curves in the low-strain range below 10-4. Figure 15 shows these results together with the results obtained from regular size cyclic triaxial tests, and indicates that the results obtained from the two testing methods are generally consistent for the shear modulus degradation, but values of damping ratio obtained from resonant column tests are somewhat higher than those obtained from the regular-size cyclic triaxial tests.

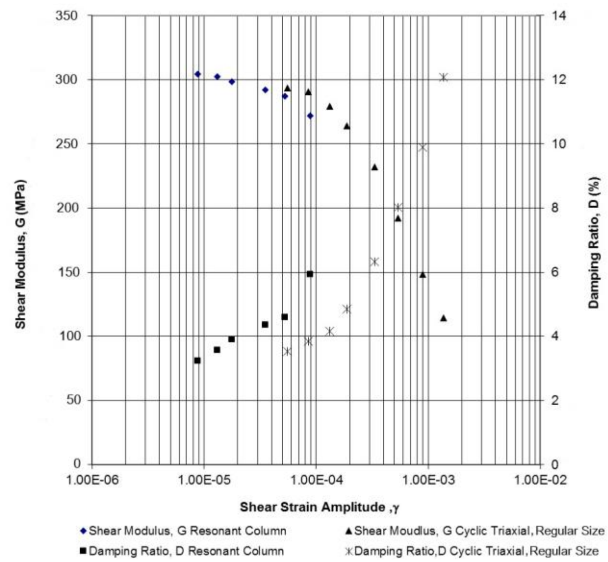


Figure 15. Shear moduli and damping ratios obtained from regular size cyclic triaxial and resonant column tests- C.P=500 kPa

Changes in shear modulus and damping ratio with shear strain are shown in Figures 15 and 16, respectively, for resonant column tests, regular-size cyclic triaxial tests, and large-size cyclic triaxial tests for each of the confining pressures of 200, 500 and 800 kPa separately for the mixed material of the Rudbar dam core. It may be noticed from these figures that while the results obtained from the three testing apparatus are generally consistent for the shear moduli, they exhibit some scatter in the case of damping ratios.

Degradation of shear moduli and changes in damping ratio with shear strain obtained from the three testing apparatus for tests with confining pressures of 200, 500 and 800 kPa are also shown in Figure 18. These results indicate that as the confining pressure increases, the degradation in shear moduli and the damping ratios decrease for the mixed clayey gravel soil tested in the current study, regardless of the testing apparatus used.

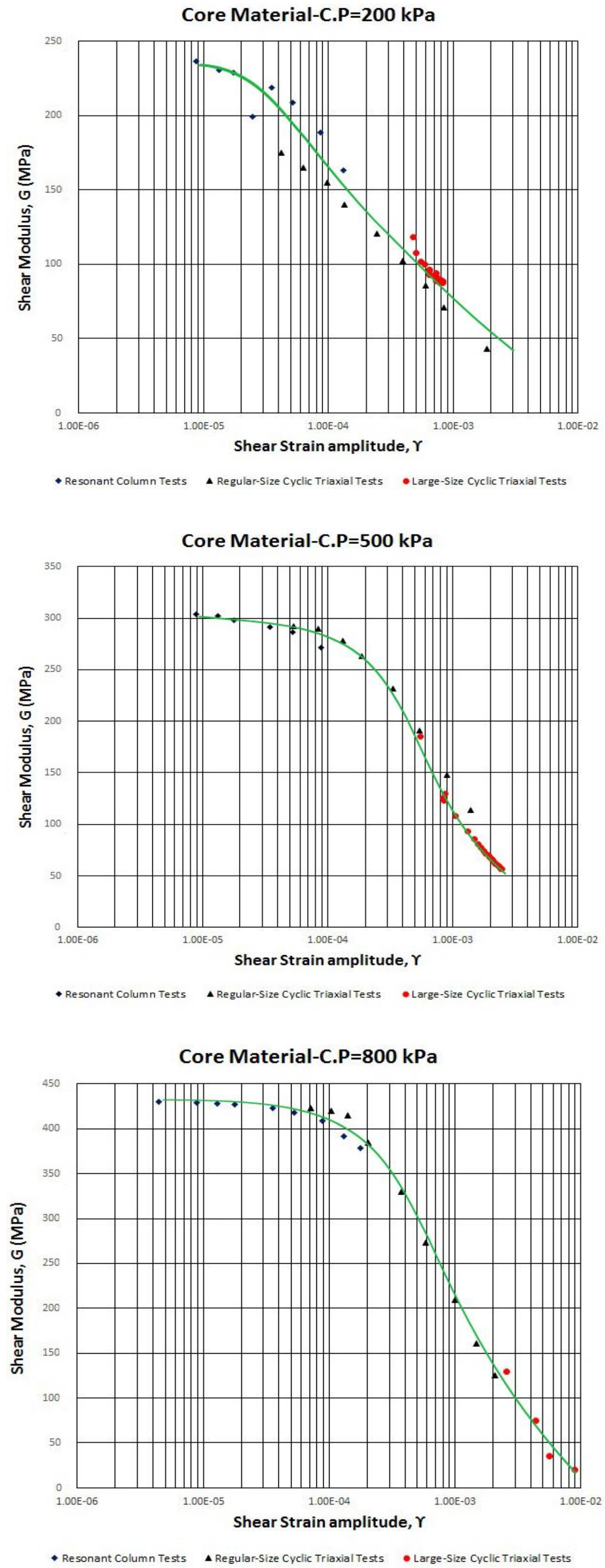


Figure 16. Shear moduli obtained from resonant column, regular-size and large size cyclic triaxial tests for confining pressures of 200, 500 and 800 kPa.

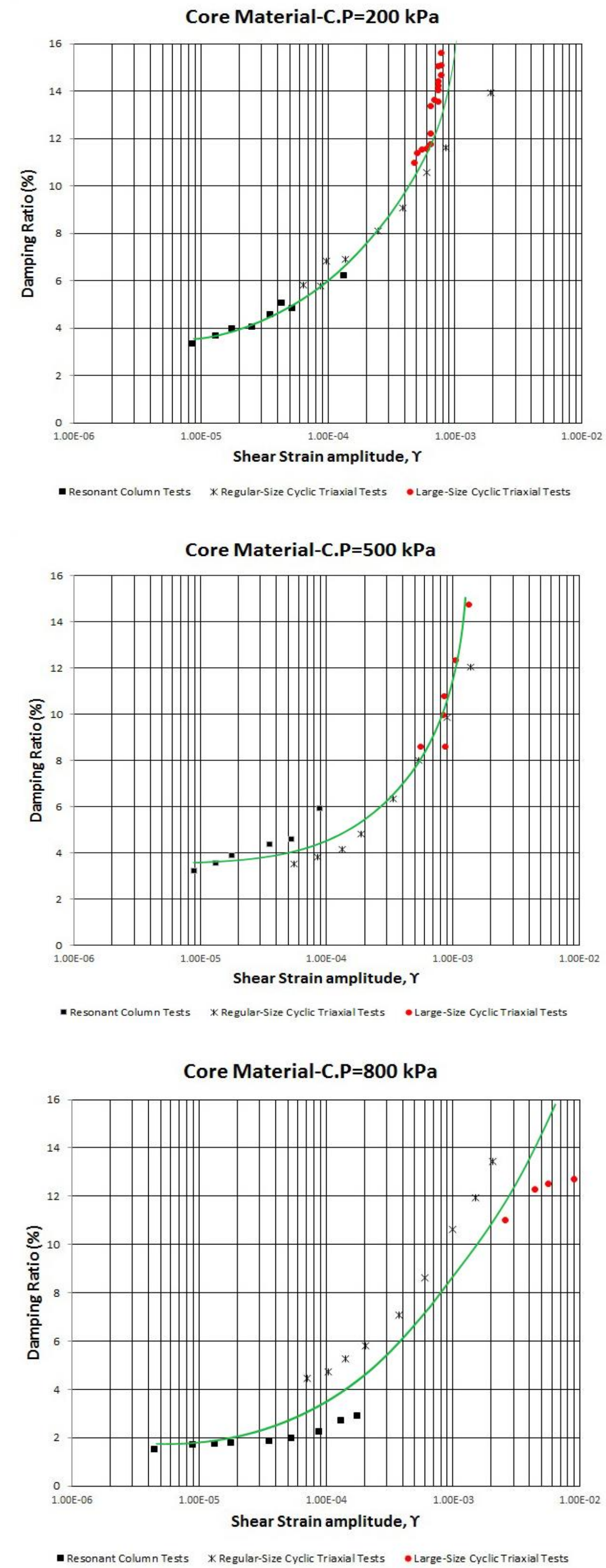
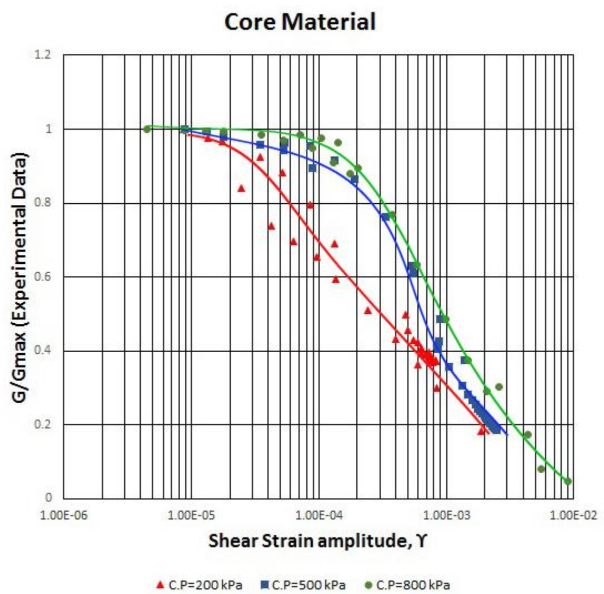


Figure 17. Damping ratios obtained from resonant column, regular-size and large size cyclic triaxial tests for confining pressures of 200, 500 and 800 kPa



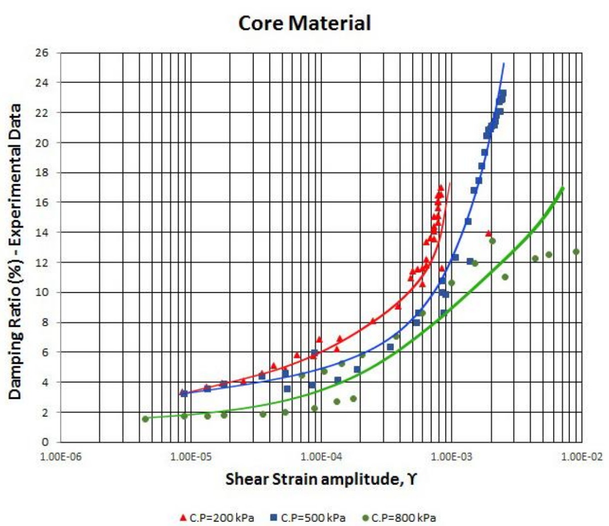


Figure 18. Effect of confining pressure on the shear modulus degradation and damping ratio as obtained from resonant column, regular-size, and large size cyclic triaxial tests

6 SUMMARY AND CONCLUSIONS

Results of large-scale and regular-size cyclic triaxial tests, and resonant column tests conducted on the clayey gravel core material of the Rudbar Dam examined during the current study lead to the following conclusions:

- Increase in the number of loading and unloading cycles, and the stress ratio applied to the soil leads to increase in shear strain and decrease in the shear modulus of the mixed material tested. These changes are larger at the initial loading cycles, but they become smaller or stop in subsequent cycles.
- Increase in the confining pressure applied to the mixed material increases its cyclic shear modulus, decreases degradation of its modulus with shear strain, and decreases its damping ratio
- Porewater pressure generated due to application of the loading and unloading cycles increases during the first approximately 100 cycles but it then becomes almost constant in subsequent cycles. In the test with higher cyclic stress ratio, a tendency for decrease in pore-water pressure is observed in subsequent cycles, which may be attributed to a tendency for dilation at higher stress ratios at which phase transformation (change from contractive to dilative behaviour) occurs in the mixed material tested.
- The material tested under cyclic loading did not exhibit liquefaction-type behaviour in which large shear strain and modulus degradation occurs after a certain state is reached. Instead, these changes occurred gradually and rate of these changes decreased in subsequent cycles. This behaviour is consistent with a cyclic mobility-type behaviour as expected to occur in the relatively dense material tested.
- Comparison of results obtained from resonant column tests with those from regular-size and large-size cyclic triaxial tests indicates that results obtained from the three testing methods are generally consistent for the shear modulus degradation with shear strain, but they show some scatter in the case of damping ratios.

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