International Journal of Engineering & Applied Sciences (IJEAS)
Vol.4, Issue 1(2012)52-66

VOLUME CHANGE AND SHEAR BEHAVIOR OF COMPACTED CLAY-SAND/GRAVEL MIXTURES

H. Soltani-Jigheh[1], K. Jafari[2]
[1]Dept. of Civil Engineering, Azarbaijan University of Tarbiyat Moallem, 35 kms from Tabriz-Maragheh Motorway, P.O.Box: 53714-161, Tabriz, Iran, e-mail: hsoltani@azaruniv.edu
[2]BSc student, Department of Civil Engineering, Azarbaijan University of Tarbiyat Moallem, Tabriz, Iran, e-mail: k.jafari.n@gmail.com

Abstract

Compacted clayey soils are used as hydraulic barriers in earth structures, such as core of earth fill dams, landfill liners, and etc. These soils have some defects from technical points of view. Mixing of these soils with granular materials (e.g. sand and gravel) is one of the soil improvement methods. A number of shrinkage, direct shear and triaxial tests are carried out on the mixtures of clay and sand/gravel. The results reveal that the values of shrinkage and swelling in the mixed clayey soils (in comparison with pure clay) decrease due to adding aggregates. Moreover, the inclusion of sand/gravel to the clays beyond a threshold value improves their shear strength and the values of shear strength and shrinkage depend on the size of grains added to the clay. Results of triaxial tests also indicate that the behavior of mixed specimens are changed from contractive to dilative with increasing of aggregates in the mixtures.

Keywords: *Mixed clayey soils; shrinkage; shear strength; laboratory tests*

1. Introduction

In some geotechnical engineering projects, such as core of earth fill dams, landfill liners, and etc, to achieve lower values of hydraulic conductivity it requires to compact clayey soils at wet of optimum water content. Shear strength of clayey soils in general is relatively low and when they subject to seasonal drying, loss of water occurs due to desiccation that alters their properties, including reduction in soil plasticity, possible cracking, and increasing of hydraulic conductivity.

Therefore, some modifications to their properties are often required to satisfy the design criteria. Mixing of clayey soils with granular materials (e.g. sand and gravel) is one of the methods to reduce the swell and shrinkage potential as well as improvement strength. These soils with dominant part of clay are called *mixed clayey soils* [1, 2].

Desiccation induced cracking of compacted soil barriers have been studied by Daniel and Wu [3] and Albrecht and Benson [4]. Kleppe and Olson [5] reported an investigation on the desiccation of two highly plastic clays and sand/bentonite mixtures. A number of studies have been done on shrinkage characteristics of compacted soils and associated cracking problems [6, 7].

In addition, several researchers have investigated the effect of granular material on the mechanical properties of mixed clayey soils [8-12]; Nakase et al., 1978; Shakoor and Cook, 1990; Shelley and Daniel, 1993; Howell et al., 1997). Vallejo and Mawby [13] carried out direct shear tests on mixtures of Ottawa sand-kaolin clay and found that shear strength of the mixtures depends upon their sand contents (w_s). They also reported results of the tests conducted by other researchers on aggregate-clay mixtures. They concluded that for the mixtures with granular material contents between 56% and 76%, shear strength of the mixture equals the granular frictional resistance plus the clay's shear strength. For the granular material contents < 56% and > 76%, the mixture's shear strength equals the clay's shear strength and the granular frictional resistance, respectively. The above conclusion is in general agreement with the findings of Vallejo and Zhou [14]. Jafari and Shafiee [15] carried out consolidated undrained compression triaxial tests on mixtures of kaolin and sand/gravel and observed that the angle of shearing resistance at the critical state increases as the aggregate contents in the mixture increases. They also observed that, during undrained loading, higher pore water pressures are induced in the mixtures with higher aggregate contents.

Soroush and Soltani-Jigheh [2] studied behavior of compacted clay-sand and clay-gravel mixtures by conducting undrained triaxial tests. They suggested critical sand/gravel contents, below which the shear strength and secant deformation modulus, of the mixed soil (as compared to those of the pure clay) remain almost unchanged, and beyond which they increase considerably. Also the results revealed that adding sand/gravel to the clay increases pore water pressures during monotonic shearing. The clay-gravel mixtures, as opposed to the clay-sand mixtures, showed a slightly higher strength and lower pore water pressure during shearing.

The review of previous studies reveals that the effects of granular material content, grain size and clay type on the behavior of mixed clayey soils have not studied, precisely. In addition, the effects of granular material content and size of grains on the volume change characteristics of these soils are unknown. For this purpose, a number of shrinkage, direct shear and triaxial tests are carried out on the specimens made of pure clay and clay-granular material mixtures. To prepare soil mixtures two different type of clay are used and varies amount of aggregates with different grain size are added to the clay. Finally, based on the results of the tests, the swelling and shrinkage properties and shear behavior of specimens are evaluated.

2. Testing Program

2.1. Materials

A number of soil specimens, made of clay-granular material mixtures (with different amount of clay and sand/gravel) and two pure clayey soils, were tested. The granular part of the mixtures comprised sub-rounded aggregates was retrieved from the borrow pit in vicinity of Tabriz city in east Azarbaijan province of Iran. In order to evaluate the effect of grain sizes on the behavior of soil mixtures, the granular soil retrieved from a barrow pit washed and sieved, and the grains retained between two frequent sieves selected as granular part of mixtures. These sieved materials were named as G, S1, and S2 with average grain size of 5.54, 2.805, and 1.44, respectively. Specific gravities of these materials respectively are 2.61, 2.63, and 2.66. Major part of cohesive materials obtained from the Azarshahr clay deposits near Azerbaijan University of Tarbiat Moallem area. This cohesive soil has index properties of LL=33%, PI=12%, and G_s=2.698 and denoted with AC.

To investigate the effect of clay plasticity on the characteristics of soil mixtures, an artificial clay with index properties of LL=48%, PI=22%, and G_s=2.64 (characterized by ACB) was used. Both of the clayey soils were categorized as CL [16]. The grading curves of used materials are shown in Figure 1.

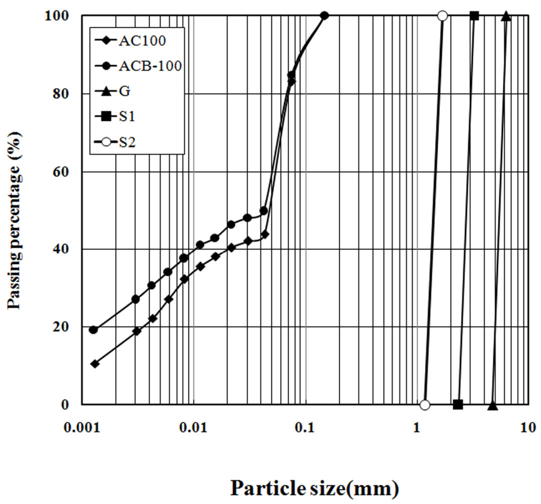

Figure 1. Grain size distribution curves of used materials.

2.2. Preparation of soil mixtures and specimens

Soil mixtures were prepared by mixing different amounts of the clay and sand/gravel (by weight) (i.e. 0%, 20%, 40% and 60% sand/gravel). Test specimens must be prepared so to model closely the prototype conditions. Therefore, standard Proctor compaction tests were performed on the specimens to obtain maximum dry density (γ_{dmax}) and optimum water content (ω_{opt}) [17]. Name of the specimens as well as their specifications are listed in Table 1. G, S1, and S2 of specimen's name show the granular material type used in the mixtures and AC and ACB stand for the type of clay matrix. The numbers denote the clay percentage in weight; e.g., the specimen made of 40% AC clay and 60% S2 sand was named as S2-AC40.

Table 1. Specification of tested specimens.

Specimen designation	Clay type	Clay (%)	Sand/gravel, w_g (%)	γ_{dmax} (kN/m^3)	ω_{opt} (%)
AC100	AC	100	0	16.5	19.5
G-AC80	AC	80	20	17.9	18.0
G-AC60	AC	60	40	18.5	13.6
G-AC40	AC	40	60	20.3	12.5
S1-AC80	AC	80	20	17.9	15.8
S1-AC60	AC	60	40	19.4	12.75
S1-AC40	AC	40	60	20.1	12.5
S2-AC80	AC	80	20	17.5	16.5
S2-AC60	AC	60	40	18.5	15.1
S2-AC40	AC	40	60	18.9	14.9
ACB100	ACB	100	0	14.24	23.5
G-ACB80	ACB	80	20	17.20	20.0
G-ACB60	ACB	60	40	17.8	16.5
G-ACB40	ACB	40	60	18.75	13.7
S1-ACB80	ACB	80	20	16.5	18.75
S1-ACB60	ACB	60	40	17.9	13.5
S1-ACB40	ACB	40	60	18.5	11.5
S2-ACB60	ACB	60	40	17.3	16.5
S2-ACB40	ACB	40	60	18.1	13.5

2.3. Shrinkage tests

The shrinkage tests were conducted on the some specimens. The test method was essentially similar to the method that described in Briaud et al. [18] and Indrawan et al. [6]. The specimens were compacted in a cylindrical Plexiglass mold of 90 mm in diameter and 100 mm in height with density of $0.95\gamma_{dmax}$ and moisture of ω_{opt} +1%. It should be noted that in order to attain homogenous specimens, materials needed for each layer were mixed with water separately and kept isolated in a plastic bag for about 24 hours before compaction. The surface of each layer was scored after compaction for better binding with the next layer. Also, in order to prevent the soil particle migration downward during the test processes, filter papers were placed in the bottom and top of the soil specimens. The similar specimen preparation method was followed for the direct shear and triaxial tests which will be introduced in next sections.

For soil saturation and subsequently downward drainage during dry process, the holes were provided within the bottom plate of cylindrical mold. After specimen preparation, it placed in a container filled with water for saturation. During saturation, for preventing any disturbance in the soil specimen due to water pressure, the height of water in container increased gradually until it reached to the level of the soil surface. In addition, a surcharge was applied from the upper surface of soil specimen to cause a pressure of 5 kPa. A schematic view of soil specimen within the cylindrical Plexiglas as well as container has shown in Figure 2.

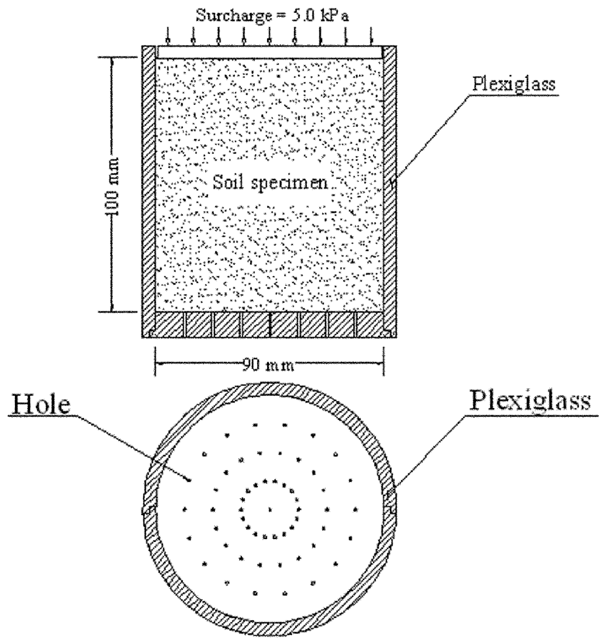

Figure 2. A schematic view of test specimen and its additional facilities.

During saturation process, a probable swelling of soil was measured using a vernier capilar and its final value is recorded. Moreover, to sure from completely saturation of specimen, it weighted occasionally. Soil volume-weight relations back calculated the degree of saturation.

Just as the specimen saturated, it was extracted from the water container to drainage and also to dry freely in air. Air-drying of specimens in the laboratory is assumed to simulate field conditions. During desiccation process the weight and volume of the specimen was measured periodically. An average of at least five measurements of the heights, diameters and lengths along the direction of specimen height were made using a vernier caliper to measuring reduced volume of specimen. Results from the measurements were used to compute the volumetric shrinkage of the specimen. To obtain an accurate computation of soil specimen volume change, at least 10 sets of measurements of volume were performed for each specimen. Finally, the specimen was placed in oven to completely drying at constant temperature of 105 ± 5 $^{\circ}$C to obtain final water content. Using the final water content and previous changes in the weights of specimens, the water contents corresponding to each measurement back calculated. The associated void ratio was calculated by measured volume changes. The dry densities (γ_d) and initial void ratios of the specimens before testing are presented in Table 1.

2.4. Shear tests

A number of direct shear tests conducted on the specimens prepared as the same as the shrinkage test method [19]. The size of specimens was 60 mm × 60 mm. Direct shear tests were done at three normal stresses of 120, 180, and 240 kPa with shearing rate of 1.0 mm/min. In addition, consolidated undrained (CU) triaixal test carried out on the specimens of AC clay-sand mixtures [20]. Diameter and height specimens were 38 mm and 76 mm, respectively. The specimens were consolidated at effective confining stress of 50, 100 and 500 kPa and then sheared with rate of 0.045 mm/min.

3. Results

3.1. Compaction Tests

The impact of granular material content on the compaction curves of specimens is indicated in Figure 3 (a, b and c) for the G, S1 and S2 mixtures, respectively. In these figures, the solid and dotted lines are respectively corresponded for mixtures of AC and ACB clays. Also, maximum dry density (γ_{dmax}) and optimum water content (ω_{opt}) for each specimen are presented in Table 1.

As obvious, the inclusion of sand/gravel to the clay influenced both the optimum water content and the maximum dry density, as with an increase in sand/gravel content maximum dry density increases and the values of optimum water content decreases.

3.2. Shrinkage Tests

Shrinkage behavior of soil specimens are evaluated in the form of variations of void ratio (*e*) versus water content, as typically presented in Figure 4. It is obvious from the figure that the void ratios of the specimens decrease due to reduction in water content until it reaches a minimum value, at which no future volume change is observed. The water content at the beginning of no volume change condition is shrinkage limit and can calculate from the e-wG_s curve for each specimen. Variations of final shrinkage and swelling percentages of specimens in term of granular material content (w_g) are presented in Figure 5. The results show that with increase of w_g the rate of reduction in void ratio (or shrinkage) decreases, as while it reaches approximately to zero for the specimens of w_g = 60%; i.e. the inclusion of grains to clay causes the shrinkage value diminishes to zero and thereby there is not any cracking at the surface of the specimens. According to Figure 5a, the swelling values of the specimens decrease with an increase in w_g. From this figure, it can conclude that the role of grains in reduction of shrinkage and swelling are more markedly in ACB-clay mixtures relative to the AC-clay mixtures.

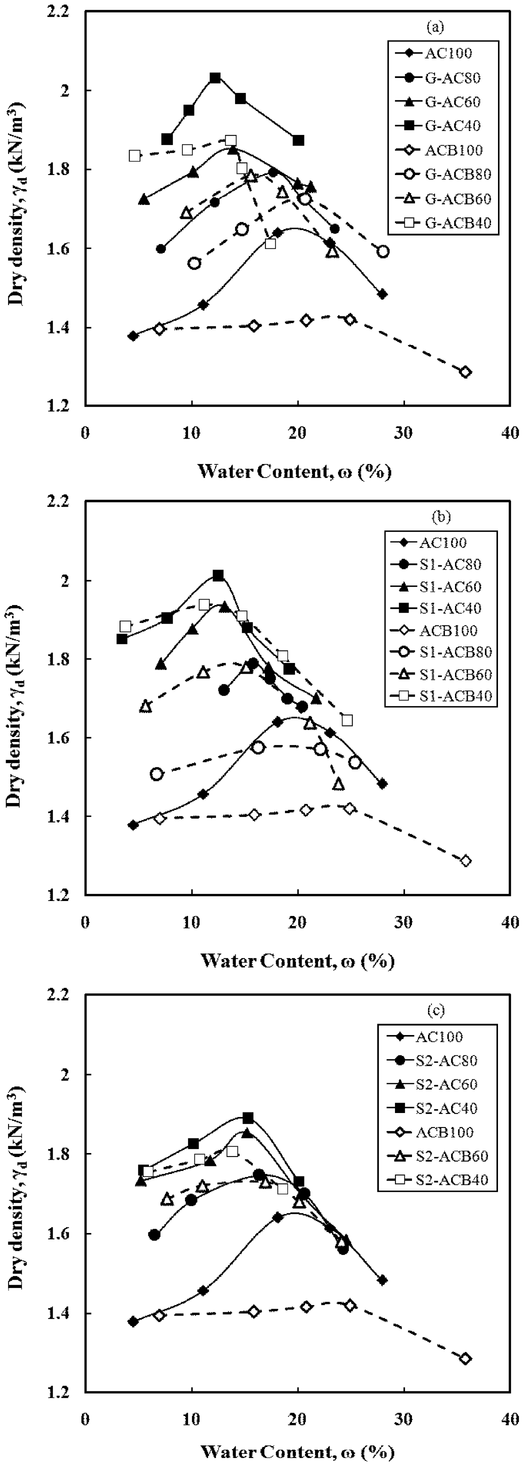

Figure 3. Compaction curves of AC and ACB mixtures with: a) G, b) S1, c) S2.

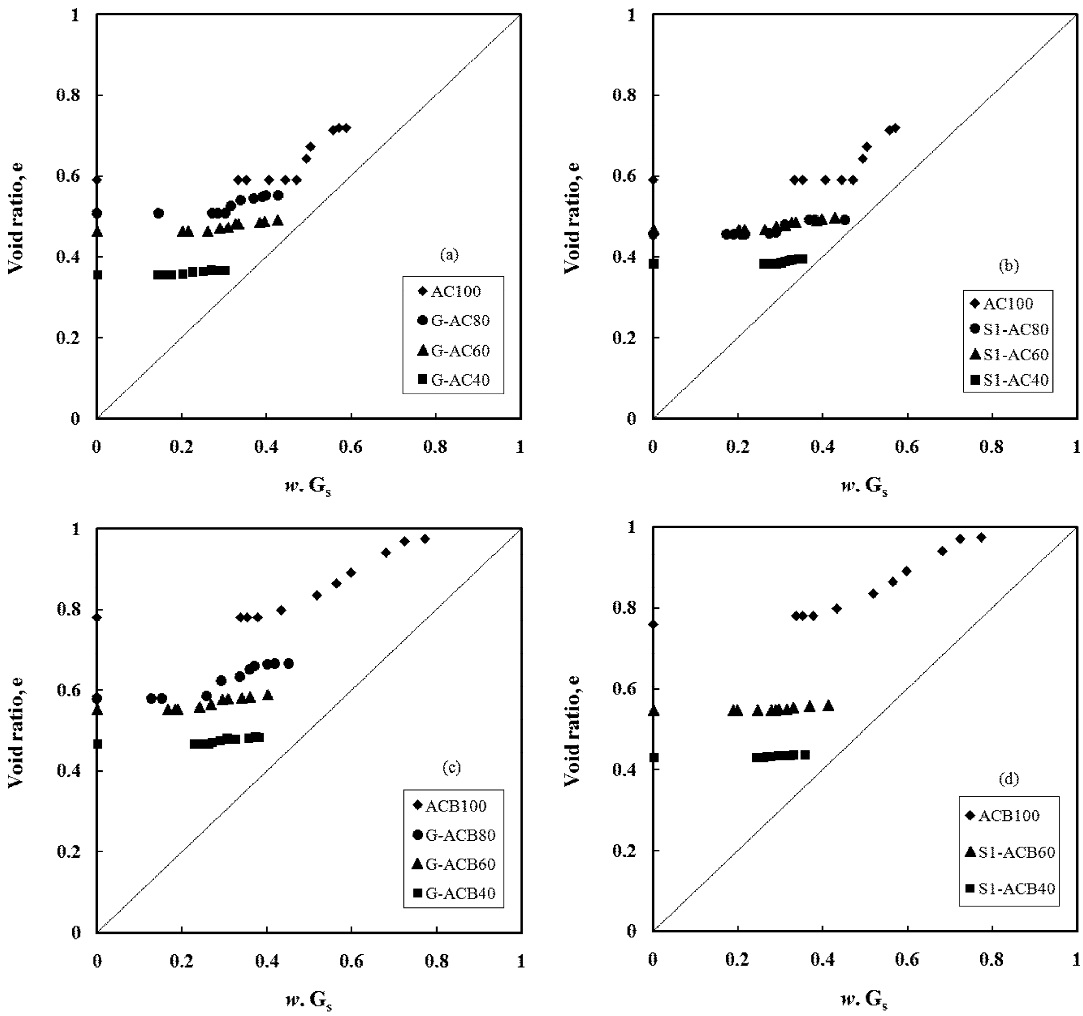

Figure 4. Void ratio versus water content: a) G-AC, b) S1-AC, c) G-ACB, and d) S1-ACB, mixtures.

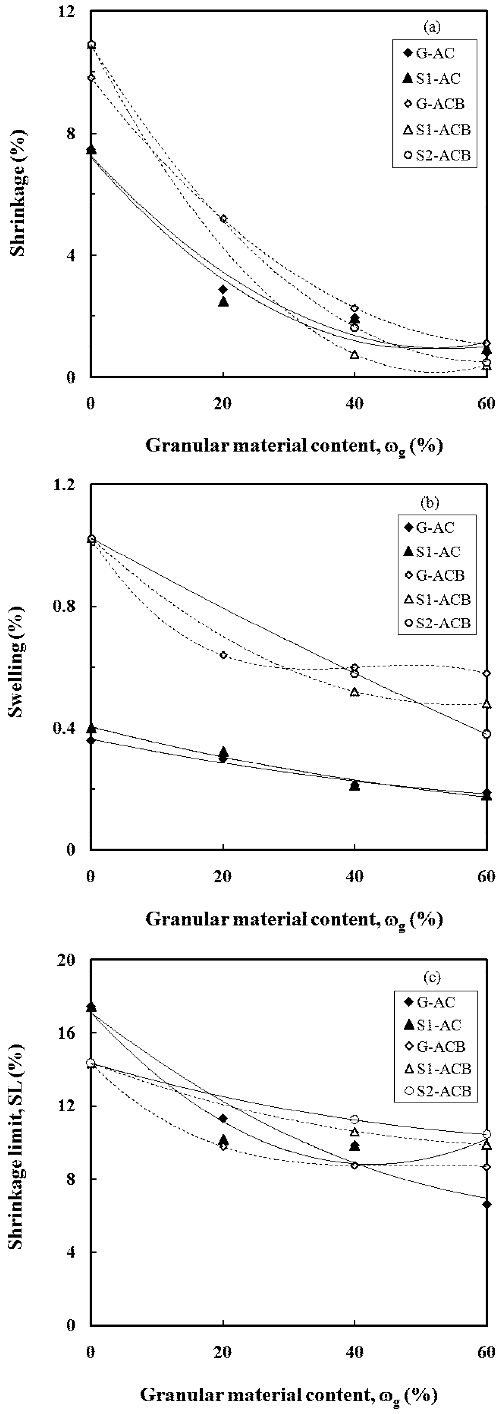

Figure 5. Effect of granular material on the values of: a) shrinkage strain, b) swelling percent, and c) shrinkage limit.

3.3. Direct Shear Tests

Variations of normalized shear strength versus granular material content are shown in Figure 6. This figure indicates that when w_g is less than a specified value, normalized shear strength is approximately ≤ 1.0; i.e. the shear strength of clay – granular material mixtures are lower than or equal to the associated value of pure clay. For w_g more than the specified value, irrespective of S2-ACB mixtures, shear strength of mixtures are considerably more than the associated value of pure clay and the rate of increasing depends on the types of granular material and clay. As depicted in Figure 7, the similar trend was observed for variations of friction angle (ϕ) versus granular material content, without any exceptions. The value of specified w_g (in general between 20%-50%) depends on the size of grains and clay plasticity. The effect of grain sizes on the values of friction angle is clear in Figure 7, as the values of ϕ increases with increasing of grain sizes. If the size of grains is very small and amount of them is high, the aggregates not only do not increase the shear strength, but also decrease it. The results show that the role of grains in improvement of shear strength is dominant in ACB-clay; i.e. when the fine-grained part of mixtures has higher plastic, adding of grains to clay matrix can improve its strength effectively.

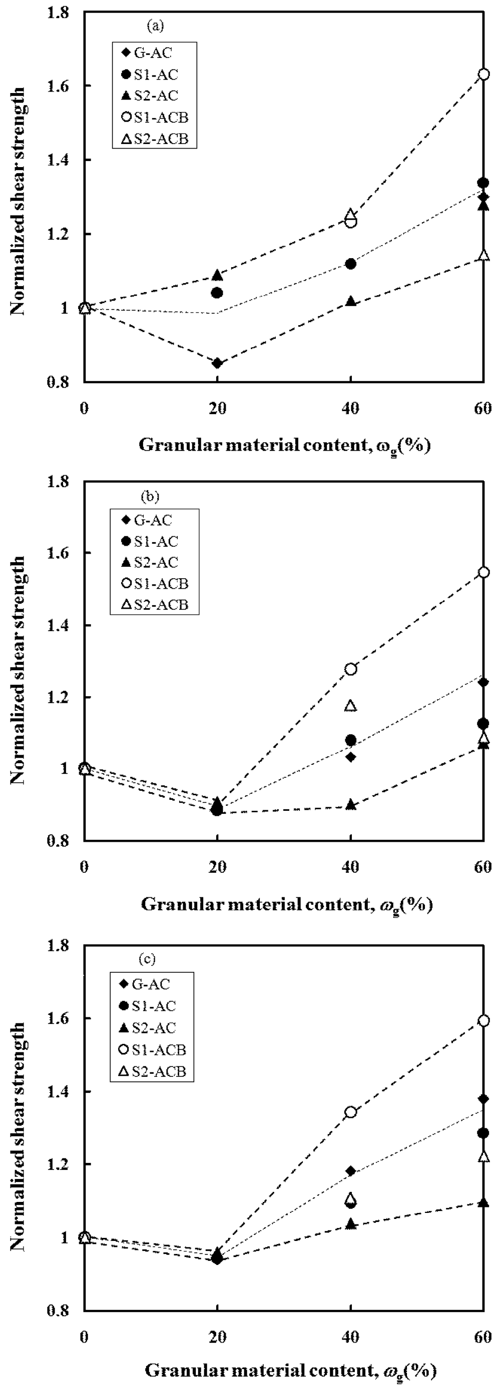

Figure 6. Normalized shear strength versus granular material content at normal stress of: a) 120kPa, b) 180kPa, and c) 240kPa.

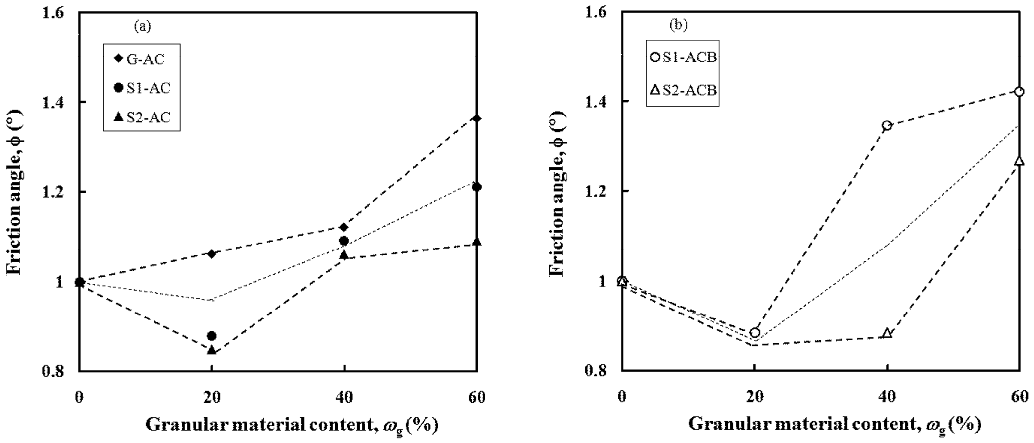

Figure 7. Variations of friction angle versus granular material content for mixed specimens of: a) AC clay, b) ACB clay.

3.4. Triaxial Tests

Typical results of the undrained tests on the AC100, S1-AC80, S1-AC60 and S1-AC40 specimens are presented in Figures 8 and 9, respectively for effective consolidation stresses (σ'_c) of 50 and 500kPa. The figures show variations of deviatoric stress (q or $\Delta\sigma_d$) versus axial strain (ε_a), Δu versus ε_a, and q versus mean normal effective stress (p′), respectively. In this section, the effects of granular material contents on the stress-strain behavior and pore water pressure generation as well as effective stress paths of the specimens are evaluated. Review of Figures 8a and 9a reveals that the behavior of S1-AC40 specimens is more different from the others and the shear strength of S1-AC40 is more than the associated values of AC100, S1-AC80 and S1-AC60 specimens.

Figures 10 and 11 present variations of undrained shear strength and secant deformation modulus of the specimens, normalized to their associated values of the pure clay [i.e., $S_{u(mix)}/S_{u(AC100)}$ and $E_{50(mix)}/E_{50(AC100)}$, respectively], versus the specimens' granular material contents (w_g). It should be noted that in these figures, the solid bullets represent the S1-AC specimens and the hollow bullets denote the S2-AC specimens. Figure 10 suggests obviously critical values for w_g (hereafter named as $(w_g)_{cr}$) below which $S_{u(mix)}/S_{u(AC100)}$ remain more or less unchanged and beyond which they increase abruptly. This critical value depends on the size of grains and w_g = 40 % and 20 % can be suggested as critical granular material contents with respect to S1-AC and S2-AC mixtures. The conclusion generally is in consistence with the results published in literature (Valejo and Mawby, 2000; Jafari and Shafiee, 2004; Soroush and Soltani-Jigheh, 2009).

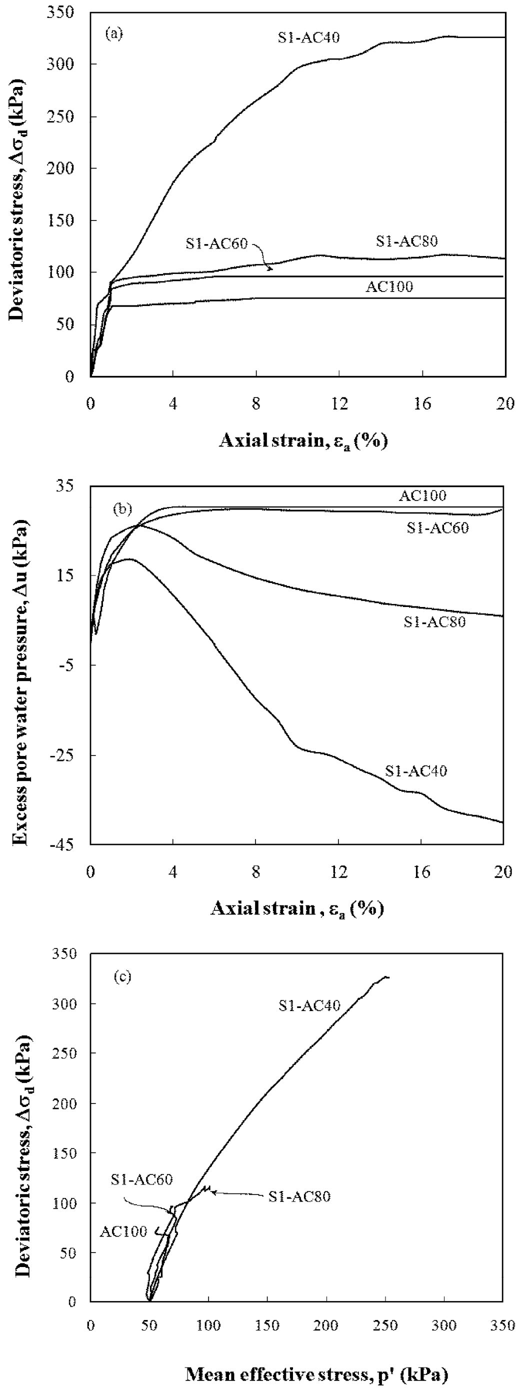

Figure 8. Results of the tests on S1-AC mixtures at σ'_c=50 kPa: a) $\Delta\sigma_d$ vursus ε_a, b) Δu vursus ε_a and c) q - p′.

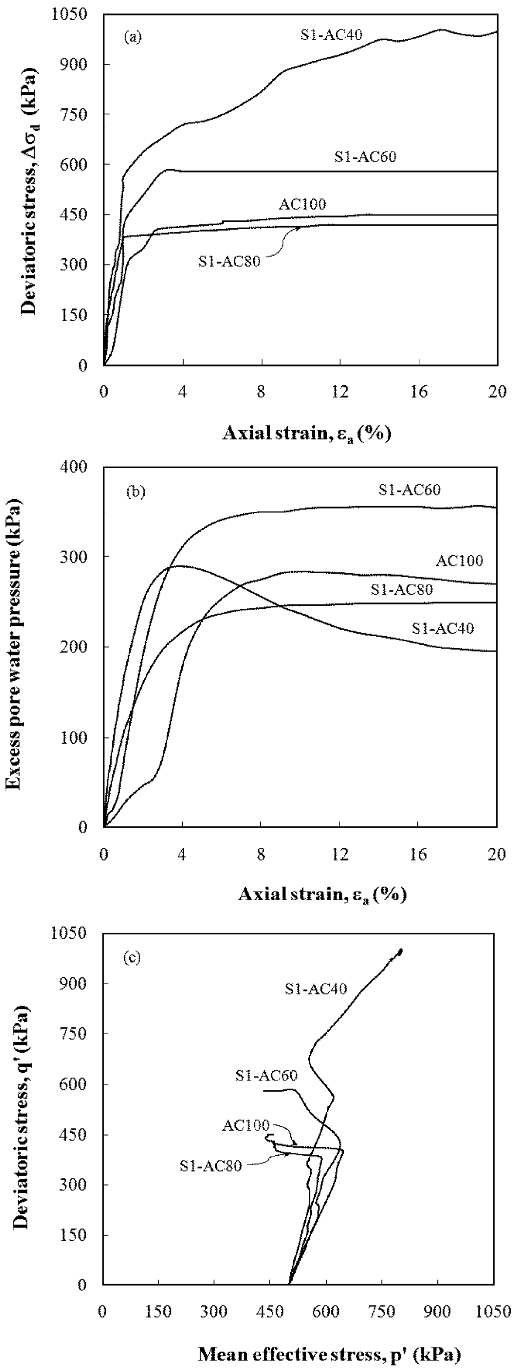

Figure 9. Results of the tests on S1-AC mixtures at σ'_c=500 kPa: a) $\Delta\sigma_d$ vursus ε_a, b) Δu vursus ε_a and c) q - p′.

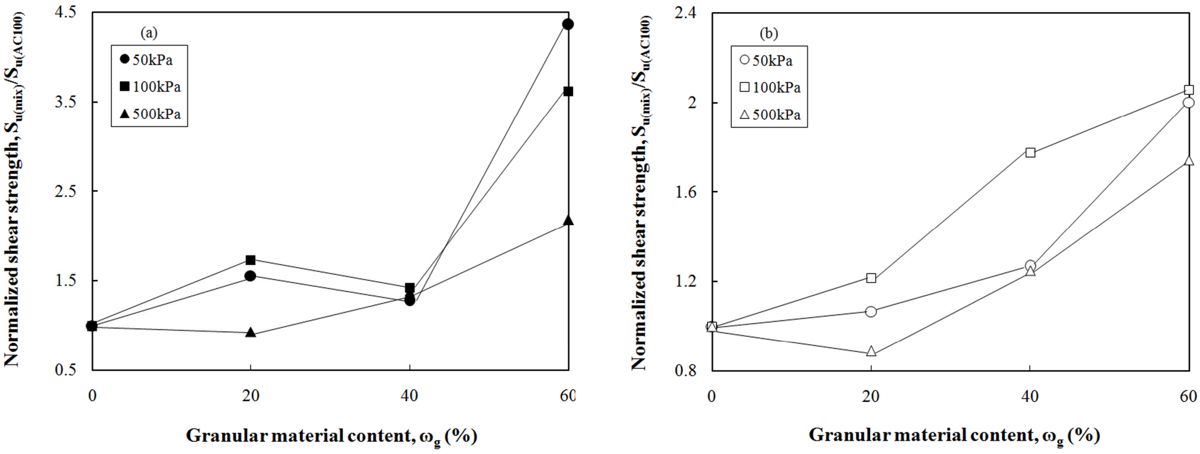

Figure 10. Variations of shear strength versus sand content: a) S1-AC mixtures, b) S2-AC mixtures.

Figure 11 shows that in general the values of $E_{50(mix)}/E_{50(AC100)}$ increase with increasing w_g from 0% to 40%, then their values decrease with increase of w_g from 40% to 60%. The reduction in E_{50} can be related to changing soil structure from floating structure (in S1-AC60 and S2-AC60) to contact one (in S1-AC40 and S2-AC40). The results are inconsistence with those reported by Soroush and Soltani-Jigheh (2009). They reported that E_{50} of mixed specimens increases with an increase in aggregates within the mixtures.

Excess pore water pressure variations (Δu) (displayed in Figures 8b and 9b) indicate that in the pure clay specimens and mixed specimens with low content of sand Δu increases due to shearing, but within the specimens of high content sand Δu increases initially with strain and then decreases; in other word the behavior of these specimens are dilative.

Figures 8c and 9c compare typical effective stress paths during shearing of the pure clay and clay-sand specimens in the q:p' plane. It appears that for the tests with a given σ'_c, behavior of specimens with low w_g is contractive. In contrast, behavior of specimens with high w_g is dilative. Dilative behavior, also, is more pronounced in the tests with low consolidation pressures (σ'_c).

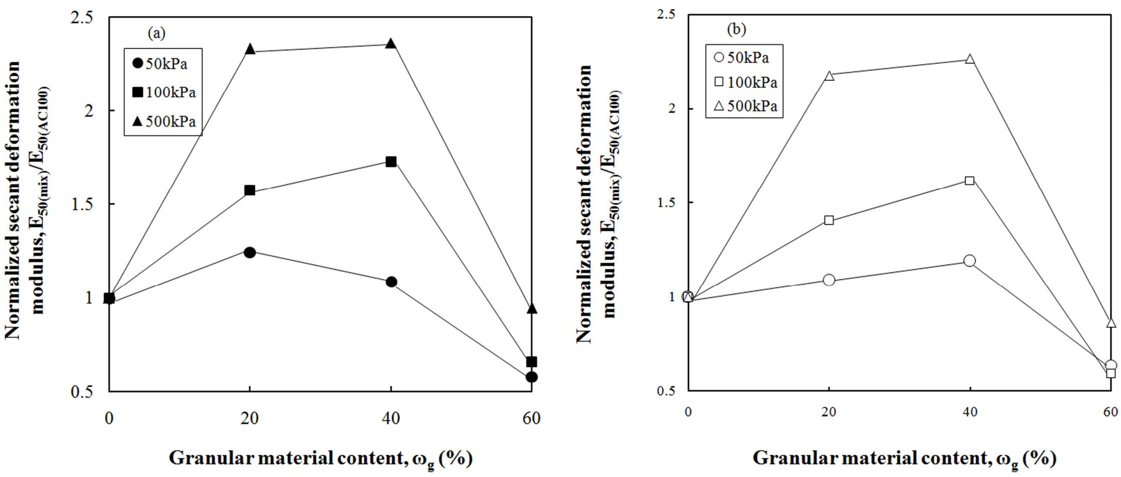

Figure 11. Variations of deformation modulus versus sand content: a) S1-AC mixtures, b) S2-AC mixtures.

4. Conclusions

To recognize the behavior of mixed clayey soils, a number of shrinkage, direct shear and triaxial tests were carried out on the specimens consist of pure clay and clay-granular material mixtures. The most important conclusions of the tests are following:

- Inclusion of grains to clay reduces the values of swelling and shrinkage; as while for the specimens of w_g = 60% the volume change due to saturation and desiccation approaches to zero, and potential of cracking diminishes in the specimens.
- Based on the direct shear tests, adding of grains to clay matrix up to a threshold value is approximately ineffective on shear strength, and even for the specimens of w_g = 20%, the shear strength of mixtures decreases as compared with pure clay. The similar results obtained from triaxial tests. This trend verified for friction angle.
- Results of direct shear tests suggest a threshold values for granular materials which are dependent on the grain sizes and clay plasticity. Also, to improve the properties of mixed clayey soils the size of added grains should not be very fine; at least it must be larger than coarse sand.
- The values of deformation modulus increase with increasing w_g from 0% to 40%, then their values decrease with increase of w_g from 40% to 60%. The reduction can be related to changing soil structure from floating structure to contact one.
- Effective stress paths revealed that when the amount of sand within the mixtures is much more, the behavior of mixture changes from contractive to dilative one.

Acknowledgments

This work has been supported by a grant/research fund number 401.508 from Azarbaijan University of Tarbiat Moallem. Appreciations are expressed for their supports.

References

[1] Soltani-Jigheh, H. and Soroush, A., Behavior of mixed clayey soils under monotonic loading, International Journal of Science and Technology Amirkabir, 18, 21-29, 2007.

[2] Soroush, A. and Soltani-Jigheh, H., Pre- and Post-cyclic Behavior of Mixed Clayey Soils, Canadian Geotechnical Journal, 46, No. 2, 115-1128, 2009.

[3] Daniel, D.E. and Wu, Y.K., Compacted clay liners and covers for arid sites, Journal of Geotechnical Engineering, 119 (2), 223–237, 1993.

[4] Albrecht, B.A. and Benson, C.H., Effect of desiccation on compacted natural clays, Journal of Geotechnical and Geoenvironmental Engineering, 127 (1), 67–75, 2001.

[5] Kleppe, J.H. and Olson, R.E., Desiccation cracking of soil barriers, Hydraulic Barriers in Soil and Rock, 874, 263-275, 1985.

[6] Indrawan, I.G.B., Rahardjo, H. and Leong, E.C., Effects of coarse-grained materials on properties of residual soil, Engineering Geology, 82, 154-164, 2006.

[7] Krisdani, H., Rahardjo, H. and Leong, E.C., Effects of different drying rates on shrinkage characteristics of a residual soil and soil mixtures, Engineering Geology, 102, 31-37, 2008.

[8] Holtz, W.G. and Willard, M., Triaxial shear characteristics of clayey gravel soils, Soil Mechanics and Foundation Engineering, 82, 143-149, 1956.

[9] Nakase, A., Nakanodo, H. and Kusakabe, O., Influence of soil type on pore pressure response to cyclic loading, In Proceeding of 5th Japan Earthquake Engineering Symposium, 593-600, 1978.

[10] Shakoor, A. and Cook, B.D., The effect of stone content, size and shape on engineering properties of compacted silty clay, Bulletin of the Association of Engineering Geologists, 27, 245-253, 1990.
[11] Shelly, T.L. and Daniel, D.E., Effect of gravel on hydraulic conductivity of compacted soil liners, Journal of Geotechnical Engineering, 119, 54-68, 1993.
[12] Howell, J.L., Shackelford, C.D., Amer, N.H. and Stern, R.T., Compaction of sand-processed clay soil mixtures, Geotechnical Testing Journal, 20 (4), 443–458, 1997.
[13] Vallejo, L.E., and Mawby, R., Void ratio Influence on the shear strength of granular material clay mixtures, Engineering Geology, 58, 125-136, 2000.
[14] Vallejo, L.E., and Zhou, Y., The mechanical properties of simulated soil-rock mixtures, In Proceeding of the 13th International Conference on Soil Mechanic and Foundation Engineering, New Dehli, India, A.A. Balkema, Rotterdam, The Netherlands, 1, 365-368, 1994.
[15] Jafari, M.K. and Shafiee, A., Mechanical behavior of compacted composite clays, Canadian Geotechnical Journal, 41 (6), 1152-1167, 2004.
[16] ASTM D 2487, Standard practice for classification of soils for engineering purposes (Unified Soil Classification System), Annual Book of ASTM Standards, 04.08, 2000.
[17] ASTM D 698, Standard test method for laboratory compaction characteristics of soil using standard effort, Annual Book of ASTM Standards, 04.08, 2000.
[18] Briaud, J.L., Zhange, X. and Moon, S., Shrink test-water content method for shrink and swell predictions, Journal of Geotechnical and Geoenvironmental Engineering, 129, 590-600, 2003.
[19] ASTM D 3080, Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions), Annual Book of ASTM Standards, 04.08, 2000.
[20] ASTM D 4767, Standard test methods for Consolidated Undrained Triaxial Compression Test for Cohesive Soils, Annual Book of ASTM Standards, 04.08, 2000.