Bulletin of Engineering Geology and the Environment (2021) 80:691–703
https://doi.org/10.1007/s10064-020-01893-w

ORIGINAL PAPER

Constant water content direct shear testing of compacted residual soils

Celal Emre Uyeturk[1] · Nejan Huvaj[1]

Received: 18 August 2019 / Accepted: 25 June 2020 / Published online: 10 August 2020

Abstract

Mechanical behavior of residual soils are studied by performing constant water content direct shear tests on reconstituted specimens using total stress analysis. The testing program involves initial degree of saturation (Sr) and applied normal stress as the control parameters. For three different soil samples, a total of 55 direct shear tests are conducted at Sr = 60%, 80%, and saturated conditions, in the normal stress range of 15–60 kPa. The results showed that shear strength increases with decreasing Sr and the relationship between shear strength and Sr is nonlinear. More dilative response is observed with decreasing Sr and decreasing applied normal stress. Maximum dilatancy and Sr relationship is nonlinear. At large displacements, samples prepared at different Sr levels showed similar ultimate shear resistances under similar applied normal stresses. It is concluded that Sr and applied normal stress dramatically influence the mechanical behavior of compacted residual soils studied.

Keywords Constant water content test · Direct shear test · Shear strength · Compacted soil · Unsaturated soil

Introduction

Compacted soils are extensively used in civil engineering applications, such as in earthfills, slopes, roads, and embankments. Therefore, the shear strength and shear-induced volumetric behavior of compacted soils are of significant importance. Extensive attention has been paid to the understanding of the behavior of compacted soils (Daniel and Benson 1990; Toll 1990, 2000; Oloo and Fredlund 1996; Delage et al. 1996; Vanapalli et al. 1996; Cokca et al. 2004; Cokca and Tilgen 2010, etc.).

According to Leroueil and Vaughan (1990), the engineering behavior of soils is significantly affected by the structure, the initial void ratio, and stress history. Ahmed et al. (1974) stated that, for a clay soil tested with unconfined compression tests, samples compacted dry of optimum moisture content (OMC) showed a brittle compressive failure at low strains which is attributed to the breakdown of numerous larger pores. On the other hand, compacted at the wet side of OMC, soils deformed to high strains. Delage et al. (1996) studied the microstructure of a compacted silt using scanning electron microscope and mercury intrusion pore size distribution measurements. They concluded that (1) for the soil compacted on the dry side of the OMC, a well-developed granular aggregate structure with inter-aggregate porosity was visible in porosimetry; (2) compacted at the OMC, a more massive structure with less obvious aggregates was observed, and the greater density obtained at OMC was attributed to lower resistance of the aggregates to deformation; (3) at the wet side of OMC, a structure where clay particles surround the silt grains was observed. Vanapalli et al. (1996) noted that regardless of having the same mineralogy, plasticity, and texture, soils compacted at different water contents and to different densities should be considered different soils. Vanapalli et al. (1996) further stated that the flow and water storage characteristics with changing suction are governed by the structure or aggregation which are resulted from compacted soils at different initial water contents.

Cokca et al. (2004) investigated the effects of compaction moisture content on the shear strength of Ankara clay with conventional direct shear test, and concluded that the friction angle decreases with increasing moisture content up to OMC and with decreasing suction. Due to aggregation of clayey particles, soil "packets" were reported to be formed, and this

✉ Nejan Huvaj
nejan@metu.edu.tr

Celal Emre Uyeturk
emreuyeturk@gmail.com

[1] Civil Engineering Department, Middle East Technical University, 06800 Ankara, Turkey

resulted in exhibition of post-softening behavior for the compacted soils, which is observed commonly in overconsolidated soils (Croney et al. 1958; Barden and Sides 1970; Collins and McGown 1974). Ng and Zhou (2005) performed a series of suction-controlled direct shear tests at different suction levels on compacted completely decomposed granite (CDG). They found that suction and soil density have a dramatic effect on the maximum dilatancy, where dilatancy can be defined as $\delta y/\delta h$, δy being the vertical displacement and δh being the horizontal displacement change. That is, upon increase in suction, the dilatancy increased. Furthermore, they noted that the maximum dilatancy and suction relationship is nonlinear. Thu et al. (2006) conducted constant water content triaxial compression tests on compacted silt and noted that at high initial matric suctions, the specimens exhibited a behavior similar to a highly overconsolidated soil: i.e. high dilation and post peak strain-softening behavior is observed at high initial matric suctions. Ng and Chiu (2003), based on triaxial tests on compacted loose unsaturated decomposed granitic soil, stated that the dilatancy of the unsaturated soil depends on the soil suction, the stress state, and the stress path.

Hossain and Yin (2010) investigated the shear strength and dilative characteristics of CDG soil via suction-controlled direct shear tests. The highest "dilation angle" ($\psi = \arctan\left(-\frac{\delta y}{\delta h}\right)$, where y is the vertical displacement and h is the shear displacement) was found for the test having the highest suction and lowest normal stress. Furthermore, Hossain and Yin (2010) stated that the tests having higher normal stress and lower suction values exhibited lower or zero dilation angles. Gallage and Uchimura (2016) conducted suction-controlled direct shear tests on compacted Edosaki and Chiba soils (fines content of 16.5% and 36%, respectively) to investigate the effects of drying and wetting on shear strength parameters at low suction (0–50 kPa). It was concluded that upon increasing normal stress, volume change becomes more contractive. Furthermore, the effect of suction on volumetric behavior of unsaturated soil reduces as the net confining stress increases. Gallage and Uchimura (2016) also noted that at higher suctions, shear stiffness was found to increase and specimens showed less contraction. They stated that at the residual state (achieved at 5–6 mm of shear displacement for their tests), no suction effects can be observed since pore-water paths could be discontinued at the shear plane and pore-water pressure at the shear plane will be equal to atmospheric pressure.

Overall, to evaluate the mechanical behavior of unsaturated soils, some researchers used modified triaxial test setup and others used modified direct shear test setup for controlling and/or measuring the pore-air and pore-water pressures. However, it is well known that suction-controlled tests are very time-consuming and suction-controlled direct shear and triaxial apparatuses are available only in limited number of research laboratories. As also noted by Oloo and Fredlund (1996), the necessary equipment for performing unsaturated tests may not be readily available in many geotechnical laboratories, and the investigation of unsaturated soil behavior is very difficult, costly, and time-consuming (Hossain and Yin 2010). This makes these types of tests not very accessible to practicing engineers and not very suitable for the case when a large number of tests is required to account for natural deposit heterogeneity. Thus, some researchers used conventional shear strength tests under constant water content conditions (Cokca and Tilgen 2010; Ahmadi-Naghadeh et al. 2013). As noted by Ng and Chiu (2003), although investigation of unsaturated soils are mostly focused on shear strength characteristics, stress-stain relationship, volume change, and dilatancy characteristics are also very important. In this study, a series of constant water content direct shear tests are performed on compacted residual soils to investigate the effects of normal stress and water content on the mechanical behavior (shear strength and volumetric behavior) in both unsaturated and saturated conditions. The results of this study add to and extend the knowledge on the behavior of compacted residual soils.

Soil characteristics

Three soil samples in disturbed condition were taken from three different already-failed rainfall-induced landslide areas in northern Turkey. The region is composed of basalts, andesitic lavas, and pyroclasts, and the soils are originated from decomposition of volcanic rocks (Gedik et al. 1992; Alan et al. 2019). Previous studies have noted that these materials contain halloysite mineral (Üyetürk 2019), which is commonly reported in volcanic residual soils (Wesley 2010). Grain size analysis, Atterberg limits, specific gravity, and miniature compaction tests were performed on these soils. Oven-drying of the soils before any of the tests was intentionally avoided since residual soils of volcanic origin are effected irreversibly upon drying (Wesley 1973, 2010; Huvaj and Uyeturk 2018). Figure 1 shows the grain size distribution curves of the samples. Index properties and compaction properties of the soils are summarized in Table 1. Dry density-water content relationship of the tested samples are obtained using Harvard compaction apparatus (Wilson 1970), and presented in Fig. 2.

Testing program

Specimen preparation

Air-dried samples were pulverized and sieved through 2-mm sieve (2-mm maximum particle size was determined based on the dimensions of the direct shear box). The direct shear boxes

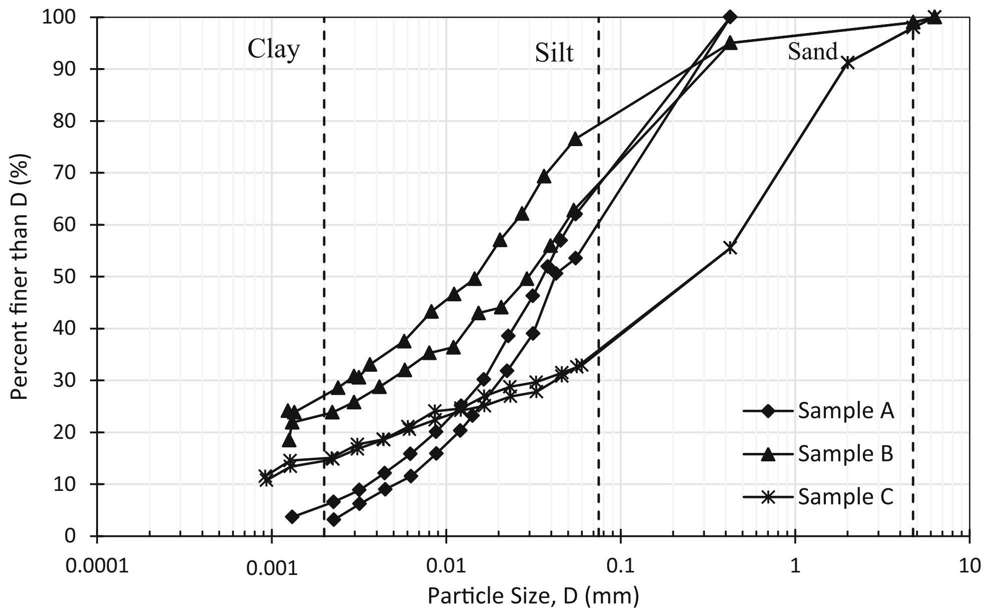


Fig. 1 Grain size distribution curves of the tested soils (two tests were performed on each sample)

used in this study were square box having a side length of 60-mm and circular box with 60-mm diameter. Specimen heights were 20 mm. Target dry density of the specimens (Table 2) were decided based on the measurements of the in situ dry density of the soils in the scarps of the landslide sites (Üyetürk 2019). Preparation water contents (see Table 3) were chosen to obtain preparation degree of saturation (Sr) as 60% and 80% for unsaturated tests, and as 90–95% for saturated tests when the soils are compacted. For sample C, specimens compacted at two different dry densities were tested, and they are denoted as C-1 and C-2, where C-1 has lower dry density as compared to C-2. Aiming the target dry density and Sr, required amount of water and air-dried soil was mixed thoroughly, sealed, and kept in humid room at least overnight (about 16 h) in order to obtain uniform water content distribution. Then, the soil-water mixture was placed into the shear box (Fig. 3a), and statically compressed in one layer to obtain the target bulk density (Fig. 3b, c). Compacting the soil in one layer was deemed suitable since the height-to-diameter ratio of the shear box is around 0.3, and thus, obtaining non-homogenous specimens was unlikely. Furthermore, some of the tests were duplicated to evaluate the repeatability of the specimen preparation method.

Direct shear test procedure

After the specimens were prepared, normal stress was applied to the specimens and the specimens were allowed to consolidate/compress under the applied normal stress. Direct shear tests conducted on compacted specimens were performed in both saturated and unsaturated conditions. Saturated conditions are assumed to be obtained by submerging the specimens under water in the shear box, as this method was used widely by other researchers as well (e.g., Cokca and Tilgen 2010; Picarelli et al. 2007). It is important to note that although specimens were submerged for saturation, this method does not guarantee "fully saturated" conditions, and there is no control of saturation of the specimen. For unsaturated tests, the main purpose was to satisfy constant water content conditions during the tests. Thus, the shear box was covered with moist towel throughout the testing procedure to avoid evaporation of water. Constant water content conditions were

Table 1 Properties of the soils used in this study

Sample name	Fines content[a] (%)	Clay size fraction[b] (%)	LL (%)	PI (%)	Specific gravity (G_S)	w_{opt} (%)	$\rho_{d,max}$ (g/cm^3)	USCS
A	61–67	2–6	40	0	2.81	≈ 30	1.60	ML
B	68–79	23–26	51	7	2.74	≈ 33	1.49	MH
C	35	14	54	15	2.77	≈ 24	1.36	SM[c]

[a] Particle diameter < 0.075 mm

[b] Particle diameter < 0.002 mm

[c] Fine portion of this sample is classified as OH (organic silt)

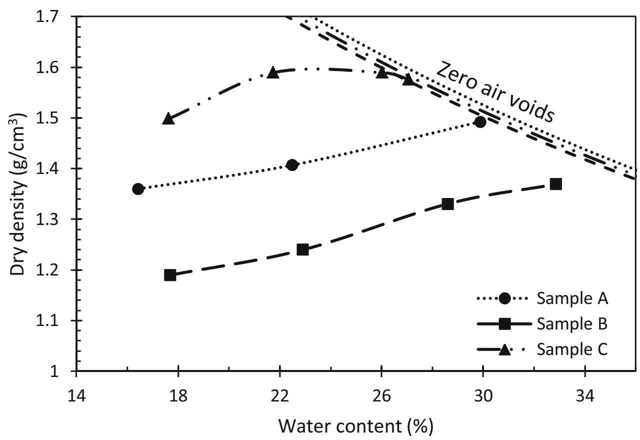


Fig. 2 Dry density-water content relationships of the samples

ensured by comparing the preparation water content and final water content of samples. The maximum difference between the starting and final water contents was found to be 1.5%; thus, it is assumed that constant water content conditions were satisfied for unsaturated direct shear tests. Preparation water contents of the samples are shown in Table 3.

Saturated direct shear tests were performed in consolidated-drained conditions based on ASTM D3080. The requirement of the drained condition is that the shear rate must be selected such that no excess pore pressure is induced. To satisfy that requirement, the shear rates were selected based on t_{50}, or t_{90}, which are the times required for 50% and 90% average degree of consolidation, respectively. The shear rates used in this study were determined as 0.024 and 0.037 mm/min. Unsaturated tests were also performed with the same shear rates used for saturated cases. Since the samples were taken from shallow landslide sites, low (15 to 60 kPa) normal stress range was used for the tests.

Test results and interpretation

Evaluating the results, the failure point is decided based on the shear stress-horizontal displacement plot, and the failure point is considered as the maximum shear stress the soil structure can carry. For example, if a peak shear resistance is observed, this value is considered the failure point; on the other hand, if no peak shear resistance is observed and the specimen reached to an ultimate shear resistance at large displacement, the maximum shear stress value (generally the last data point) is taken as the failure point. In the calculation of the shear stress and normal stress at failure, area correction was applied. Dilatancy values are calculated using $\psi = -\delta y/\delta h$, where δy is the vertical displacement and δh is the horizontal displacement change with respect to the initial values of (0,0). In the plots, "−" values indicate dilation (volume increase) and "+" values indicate contraction (volume decrease).

In the figures, saturated tests are denoted as "S-Y," and unsaturated tests are denoted as "UX-Y," where X indicates the preparation degree of saturation (Sr) in percent and Y indicates the initial applied normal stress.

For the unsaturated tests, interpretation of the shear strength requires two stress state variables: net normal stress and matric suction (Fredlund and Morgenstern 1977). In this study, since suction measurements were not performed, specimen preparation degree of saturation (Sr) is used rather than matric suction. The behavior of the specimens during shearing are discussed using stress state variables: (1) normal stress, (2) preparation degree of saturation in the following sections. Nevertheless, Sr and matric suction can be used interchangeably while interpreting the results. For example, increase in Sr implies decrease in matric suction when all else are constant.

For three different soil samples, at Sr = 60%, 80%, and saturated conditions, in the normal stress range of 15–60 kPa (for each shear strength envelope, at least 4 different normal stresses are used), a total of 55 direct shear tests are conducted. The test results presented in the following sections show that some of the specimens showed a clear peak with dilatant behavior, whereas some others reached to ultimate strength at large displacements together with contraction in volume. All of the tests showed that the shear strength increases with increasing applied normal stress and decreasing Sr.

Effect of applied normal stress on shear strength and volumetric behavior

Shear stress, vertical displacement, and dilatancy versus horizontal displacement plots are shown in Figs. 4, 6, 8, and 10 for unsaturated tests, and Figs. 5, 7, 9, and 11 for saturated tests.

Observations for sample A

Increase in normal stress changed the shear resistance behavior of the specimens. Under the lowest normal stress of Sr =

Table. 2 Target dry densities of the specimens in direct shear tests

Sample name	A	B	C-1	C-2
Preparation dry density (g/cm^3)*	1.225 (1.220–1.229)	1.260 (1.251–1.270)	1.421 (1.409–1.433)	1.580 (1.565–1.594)

*Achieved range in the tests is reported in parenthesis

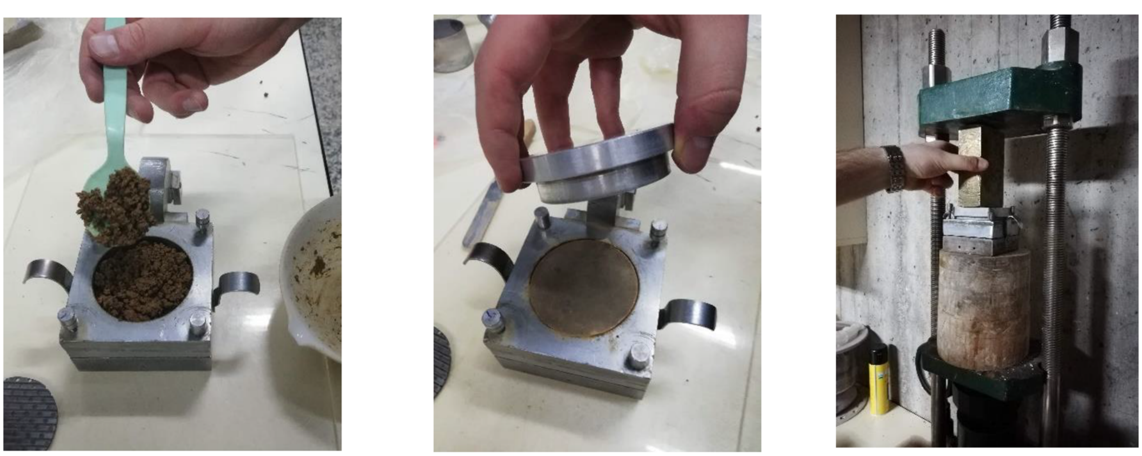
Table 3 Direct shear test water contents and obtained shear strength parameters

Sample	Degree of saturation (%)	Preparation water content (%)	Optimum moisture content (%)	Failure cohesion (kPa)	Ultimate cohesion (kPa)	Failure internal friction angle (°)	Ultimate internal friction angle (°)
A	60%	27.3	~30	15.9	7.4	33.3	38.0
	80%	37.0		12.1	8.4	32.2	34.9
	Saturated	40.8		0.3	0.4	34.3	34.3
B	60%	25.9	~33	28.5	5.9	35.3	41.3
	80%	34.2		20.6	7.0	31.7	39.5
	Saturated	40.1		4.5	0.8	31.5	33.7
C-1	60%	21.3	~24	32.2	1.9	27.5	44.2
	80%	28.2		11.6	10.6	33.1	33.7
	Saturated	32.1		3.0	−0.3 (0)	35.4	35.0
C-2	60%	16.7	~24	44.6	10.0	51.6	37.6
	80%	21.9		29.2	5.8	44.3	40.4
	Saturated	25.4		14.3	4.4	34.4	31.0

80% (Fig. 4(a)) and saturated cases (Fig. 5(a)), after the maximum shear resistance is reached, there is no decrease from that value as horizontal displacements continued. As the normal stress increased, specimens started to exhibit strain-hardening behavior. For Sr = 60% case (Fig. 4(a)), strain softening is observed under the lowest normal stress. Upon increase in normal stress, specimens started to exhibit no strain softening, and under the highest normal stress, specimens showed strain-hardening response. Considering the volumetric response during shearing, as the normal stress decreased, specimens dilated more. That is, the most dilatant specimens at different saturation states are the ones under lowest normal stress (Figs. 4(a) and 5(a)). Moreover, the maximum dilatancy decreased (i.e., specimens dilated less) upon increase in normal stress (Figs. 4(c) and 5(c), 12a).

Observations for sample B

Volumetric change behavior similar to sample A is also observed for sample B (Fig. 6(b, c)) for Sr = 60% and Sr = 80% cases. That is, as the normal stress increased, most of the specimens dilated less. Also, for saturated case of sample B, although S 31.9-kPa test contracted more with respect to S 15.7-kPa test, the lowest contraction amounts are observed for the highest normal stress case (Fig. 7(b)). Furthermore, as the normal stress increased, the maximum dilatancy decreased for Sr = 60% and Sr = 80% cases (Fig. 6(c)). Considering the lowest normal stress test for Sr = 60% and Sr = 80% cases, peak shear resistance is observed during shearing (Fig. 6(a)). It is also observed that as the normal stress increased (i) for the Sr = 60% case, the loss of shear resistance is decreased; and (ii) for the Sr = 80% case, specimens



(a) (b) (c)

Fig. 3 Reconstituted specimen preparation for direct shear tests: **a** soil placement, **b** static compaction by hand, **c** compaction by hydraulic piston

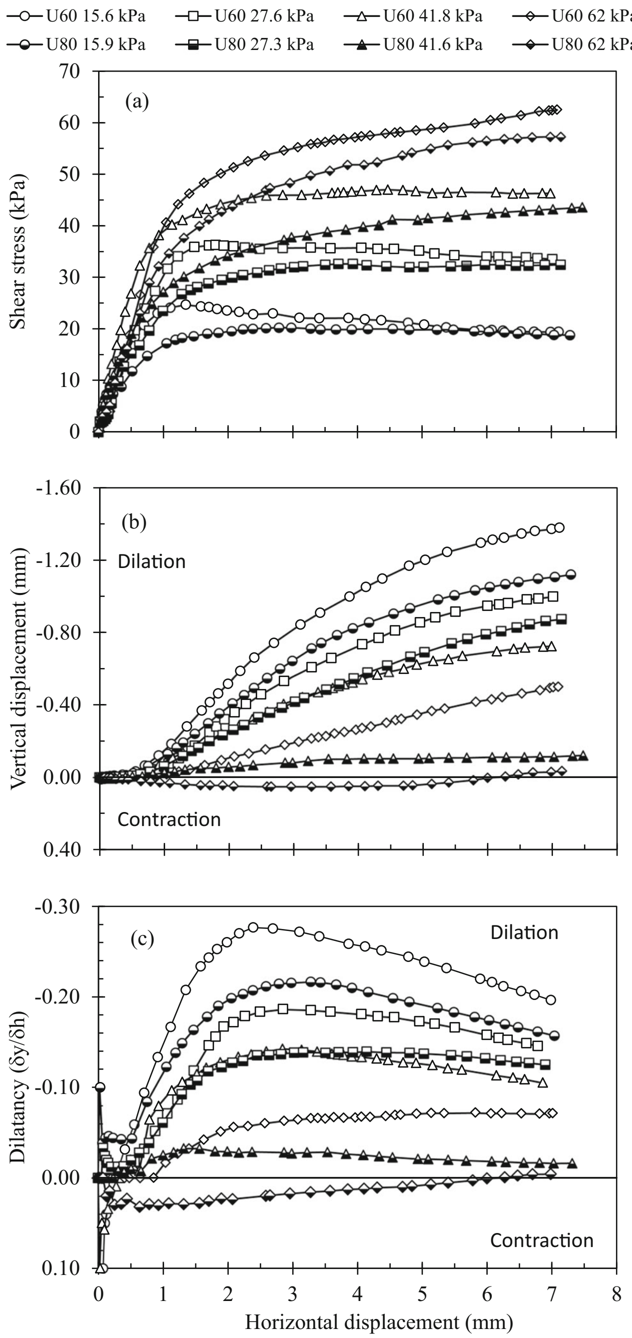


Fig. 4 Direct shear test results for unsaturated specimens of sample A: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement

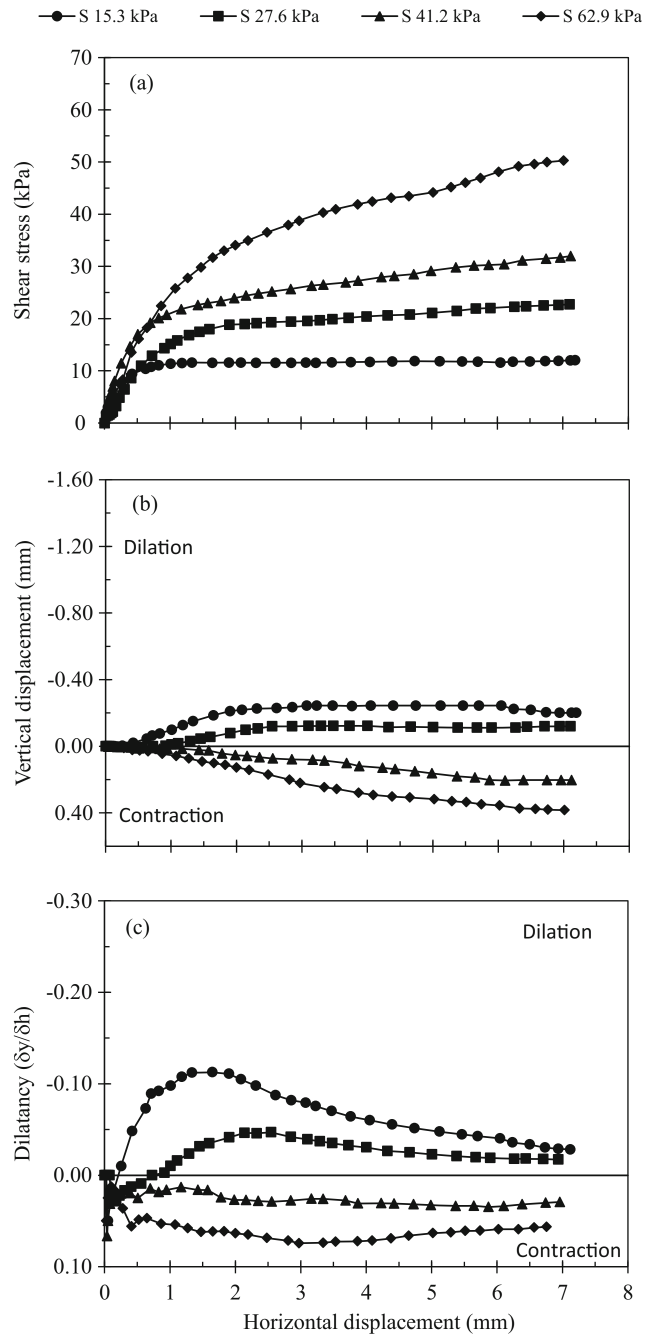


Fig. 5 Direct shear test results for saturated specimens of sample A: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement

started to exhibit strain hardening rather than strain softening (Fig. 6(b)). Similar observation is made for saturated tests (Fig. 7(a)). That is, increase in normal stress resulted in strain-hardening behavior.

Observations for samples C-1 and C-2

Increase in normal stress resulted in strain-hardening behavior for all the specimens of sample C-1 (Figs. 8(a) and 9(a)). Focusing on the Sr = 60% tests, the specimen sheared under lowest normal stress exhibit a clear peak shear resistance followed by an abrupt loss of shear resistance (Fig. 8(a)). It can be seen in Fig. 8(a) that upon increase in normal stress, the difference between peak and ultimate shear stresses decreased. Also, upon increase in normal stress, the rate of decrease of shear resistance decreased (i.e., the rate of shear resistance loss is decreased). Furthermore, for the highest applied normal stress, no resistance loss is observed but rather the specimen

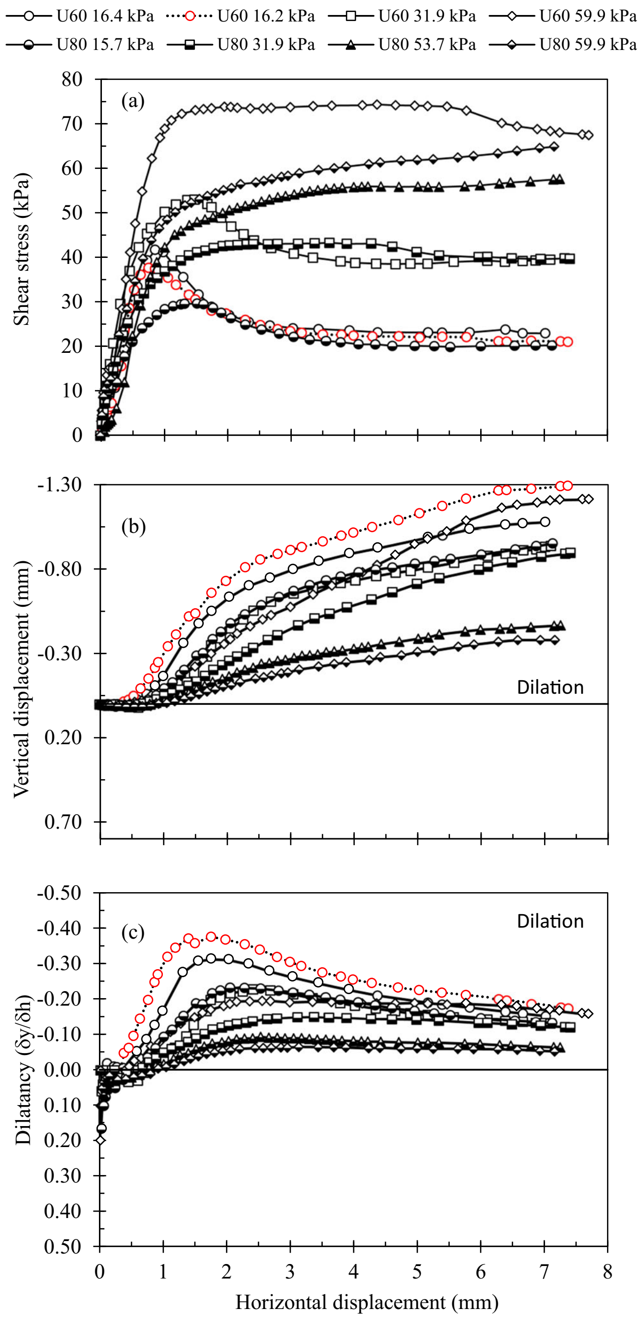


Fig. 6 Direct shear test results for unsaturated specimens of sample B: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement

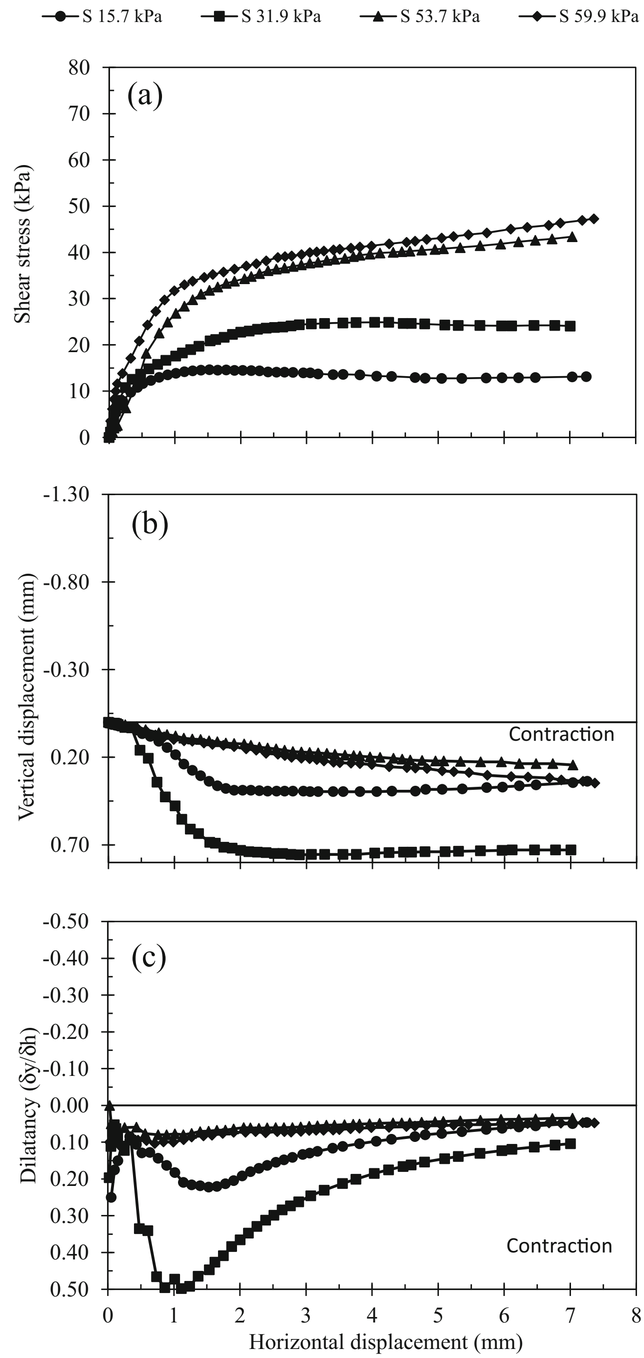


Fig. 7 Direct shear test results for saturated specimens of sample B: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement

exhibits strain-hardening behavior. Maximum dilatancy increased with decrease in normal stress (Figs. 8(c), 9(c), and 12c).

Specimens of sample C-2, which were compacted at higher dry density as compared to C-1, all exhibited dilation. Nevertheless, similar trends are also observed for sample C-2 (Figs. 10(c) and 11(c)).

Ahmed et al. (1974) noted that, a clay soil compacted at the dry side of OMC exhibited a brittle failure at low axial strains, whereas the same soil compacted at the wet side of OMC showed a ductile behavior in unconfined compression tests. In the current study, in constant water content direct shear tests on samples compacted at the dry and wet sides of OMC, it can be concluded that such generalizations cannot be made, because of the significant effect of normal stress (i.e., confining stress) on the brittle/ductile response, which may not have been

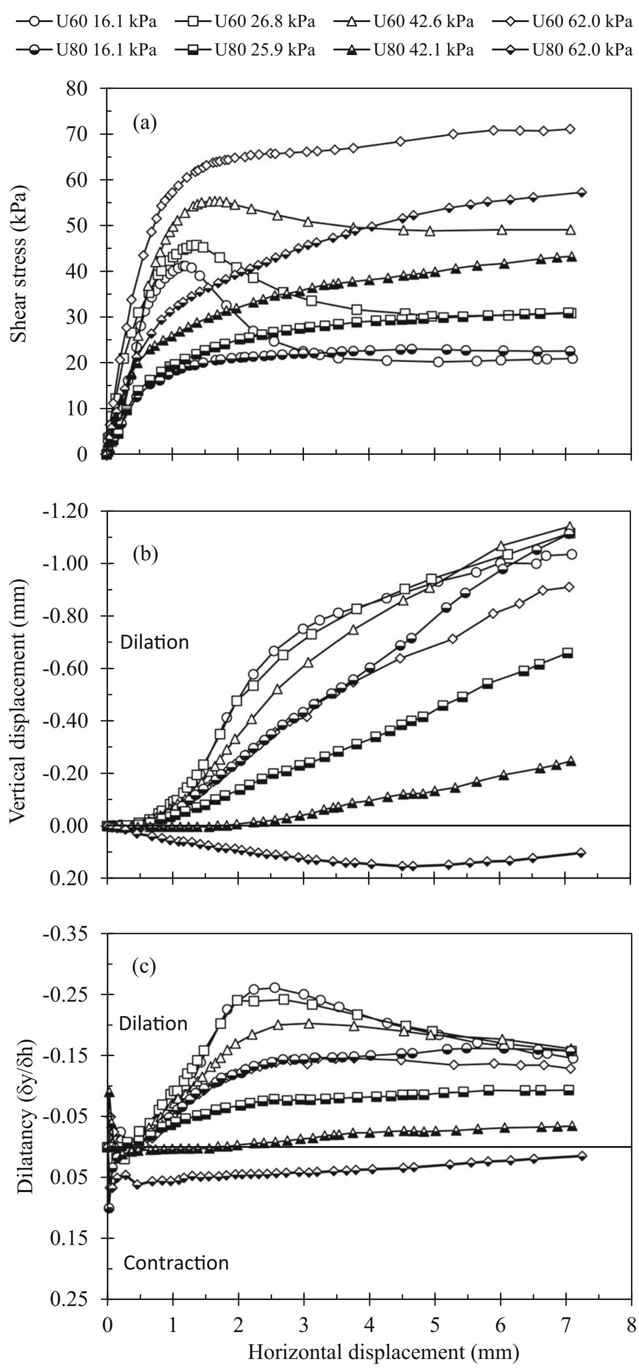


Fig. 8 Direct shear test results for unsaturated specimens of sample C-1: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement, for lower dry density case

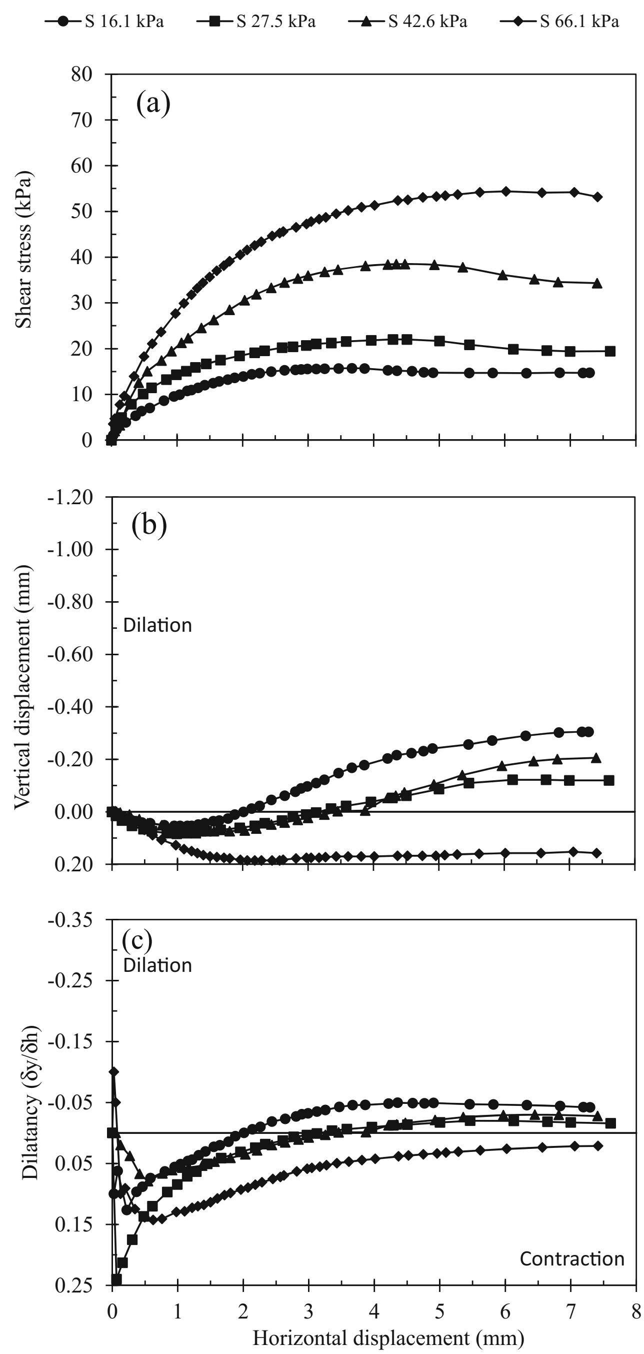


Fig. 9 Direct shear test results for saturated specimens of sample C-1: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement, for lower dry density case

observed in unconfined compression tests. For example, in sample C-1, U60 tests compacted at the dry side of OMC (Fig. 8(a)) showed that, not all of the samples compacted at the dry side of optimum showed a brittle behavior; and as normal stress level increased, more ductile behavior is observed on samples compacted at the dry side of OMC. Another example is sample C-2; saturated (S) tests compacted at the wet side of OMC (Fig. 11(a)) where they showed brittle response. Therefore, the brittle/ductile response is significantly influenced by the normal stress level and initial density state as well as the compaction water content being on the dry/wet side of OMC (or Sr).

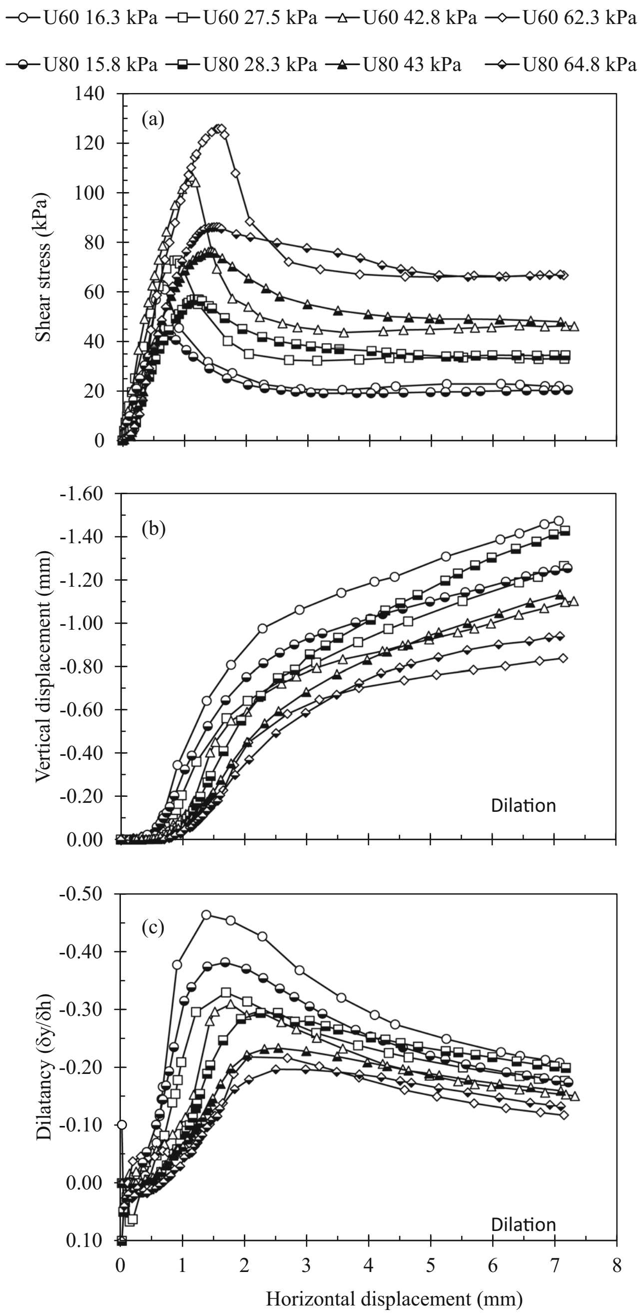


Fig. 10 Direct shear test results for unsaturated specimens of sample C-2: (a) shear stress-horizontal displacement, (b) Vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement, for higher dry density case

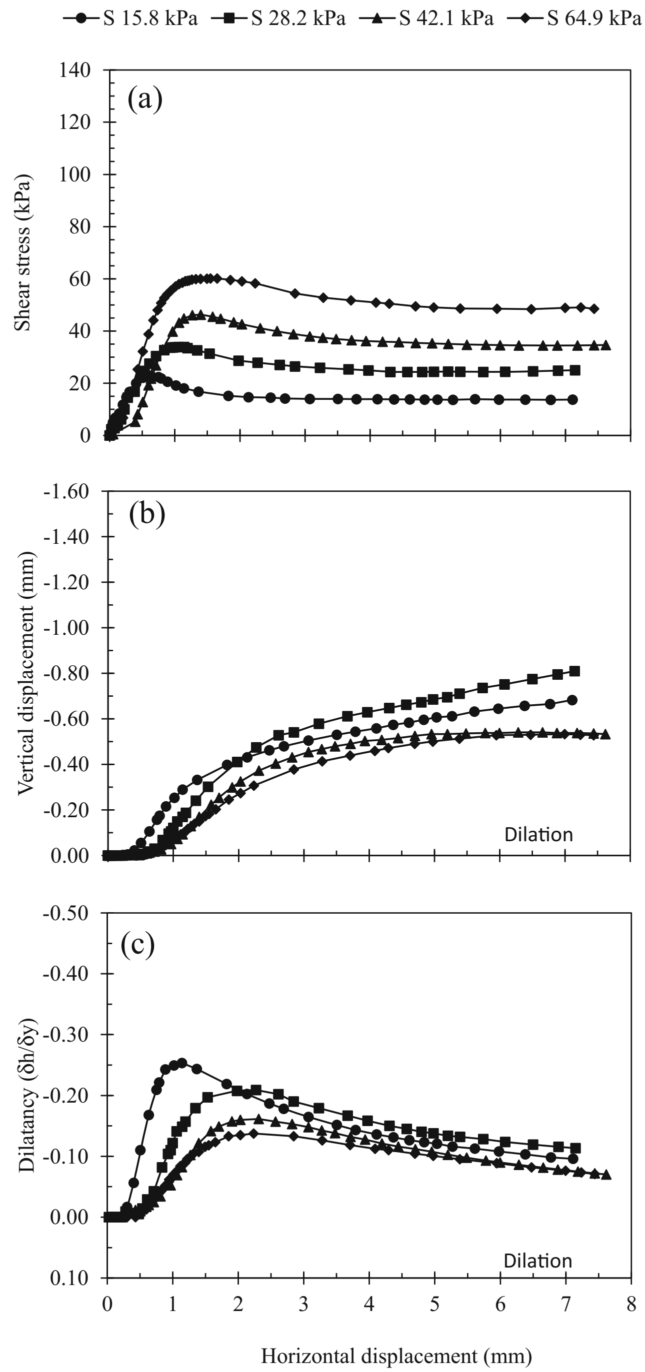


Fig. 11 Direct shear test results for saturated specimens of sample C-2: (a) shear stress-horizontal displacement, (b) vertical displacement-horizontal displacement, (c) dilatancy-horizontal displacement, for higher dry density case

Effect of preparation degree of saturation on shear strength and volumetric response

A clear peak shear resistance is generally observed only for some of the unsaturated specimens and saturated cases of sample C-2 (which is attributed to high dry density). For sample C-2 (Figs. 10(a) and 11(a)), for unsaturated tests, a rather abrupt decrease in shear resistance, after reaching the peak resistance, is observed. As Sr changes from 60 to 80% to saturated, the decrease in shear resistance with horizontal displacement gets more gradual (i.e., the difference between peak and ultimate shear stresses increased and more brittle behavior is observed as Sr decreases).

The shear strength and initial stiffness (that can be obtained from shear stress-horizontal displacement graphs) both

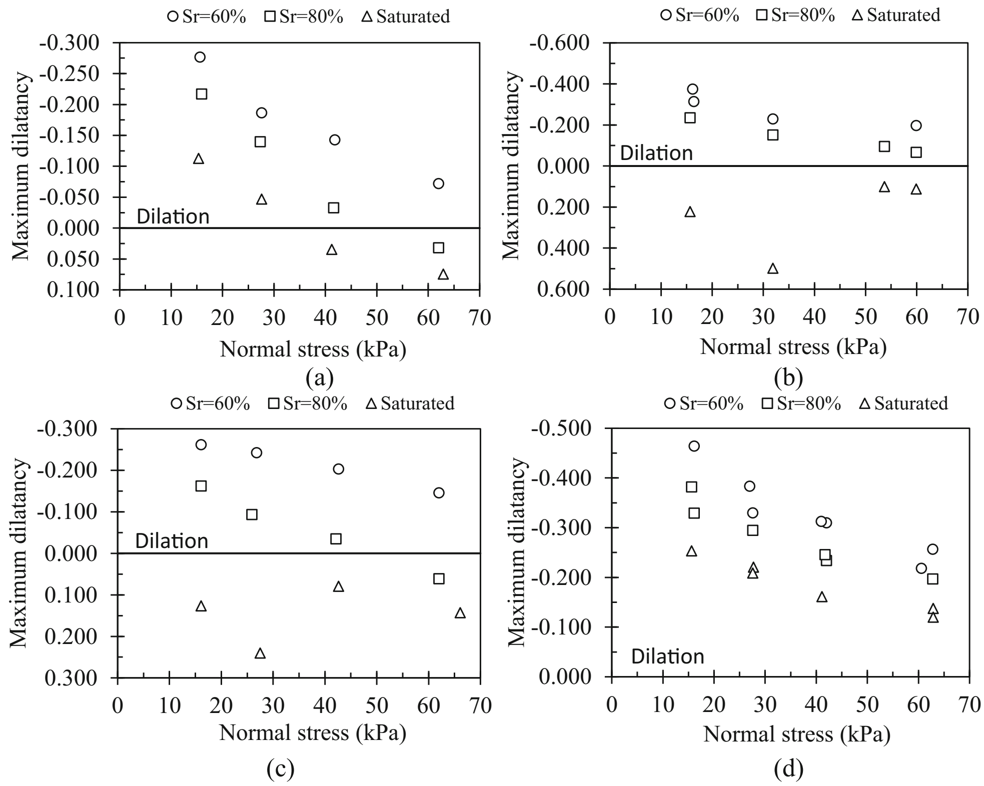


Fig. 12 Maximum dilation and normal stress relationships for different Sr values for **a** sample A, **b** sample B, **c** sample C-1, and **d** sample C-2

increase with decreasing Sr (which implies increasing matric suction) at a given normal stress, for all samples. Furthermore, stiffness increased with increasing normal stress at a given Sr value. Gallage and Uchimura (2016), based on suction-controlled direct shear tests on two different compacted soils, also concluded that at higher suctions (lower Sr), the stiffness increases.

Direct shear tests conducted on the specimens having the same initial dry density but different compaction water contents (either on the dry or wet side of OMC) showed that at large displacements, on the order of 6–7-mm (ultimate condition) shear stress values of different degree of saturation cases at similar applied normal stress approach to each other, i.e., when the samples are sheared under similar normal stresses, similar ultimate shear resistance is obtained independent of the Sr (or independent of being compacted at the dry or wet side of OMC, or independent of the suction level) (Figs. 4(a), 5(a), 6(a), 7(a), 8(a), 9(a), and 10(a)).

These results are supported by suction-controlled direct shear tests of Gallage and Uchimura (2016), who stated that at the residual state (achieved at 5–6 mm of shear displacement for their tests), no suction effects can be observed since pore-water paths could be discontinued at the shear plane and pore-water pressure at the shear plane will be equal to atmospheric pressure, which results in disappearance of suction effects.

Most of the unsaturated tests exhibited volume increase, whereas most of the saturated tests showed volume decrease (Figs. 4(b), 5(b), 6(b), 7(b), 8(b), 9(b), and 10(b)) (except sample C-2, having high dry density, where all of the specimens dilated). Furthermore, generally, as the Sr decreased (matric suction increased), the specimens became more dilative (or less contractive) under similar applied normal stress (Figs. 4(b), 5(b), 6(b), 7(b), 8(b), 9(b), 10(b), and 11(b)). That is, generally, the most dilative response is observed for the sample having the lowest Sr, under the same normal stress.

At the same Sr (at similar suction level), as normal stress is increased, volume change becomes more contractive (Figs. 4(b), 5(b), 6(b), 7(b), 8(b), 9(b), 10(b), and 11(b)). The same conclusion was reported by Gallage and Uchimura (2016) via suction-controlled direct shear tests on two different compacted soils.

Maximum dilatancy-normal stress relationships for different Sr cases, for samples A, B, C-1, and C-2, are shown in Fig. 12a–d, respectively. Maximum dilatancy-Sr relationships for different normal stress levels cases, for sample A and sample C-1, are shown in Fig. 13. It is observed that Sr influenced the maximum dilatancy of specimens (Figs. 12 and 13). That is, as Sr decreased (as suction increased), maximum dilatancy increased (Figs. 4(c), 5(c), 6(c), 7(c), 8(c), 9(c), 10(c), 12, and 13). Similar conclusion was also stated by Ng and Zhou (2005) based on suction-controlled direct shear tests, where they reported that suction and soil density have a dramatic effect on the maximum dilatancy; upon increase in suction, the dilatancy increased, and that the maximum dilatancy and suction relationship is nonlinear. Figure 13 also shows that

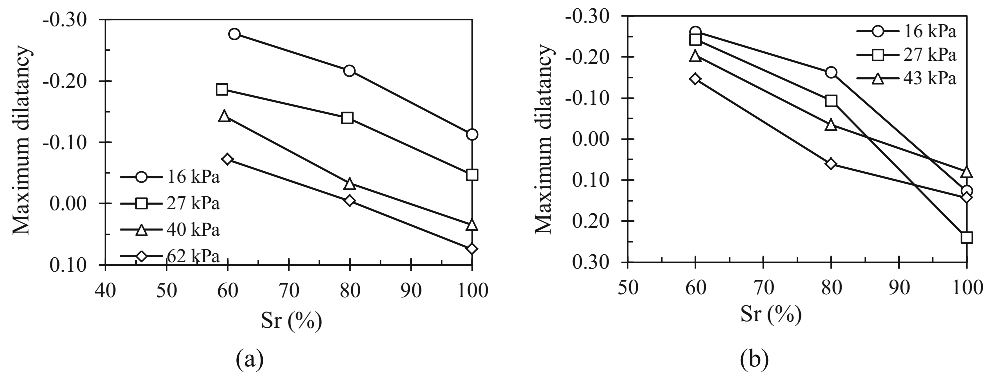


Fig. 13 Maximum dilatancy and degree of saturation relationship **a** for sample A and **b** for sample C-1, (normal stresses are approximate average values for all tests)

maximum dilatancy and Sr (matric suction) relationship is nonlinear, based on constant water content direct shear tests in this study. Ng and Chiu (2003), based on triaxial tests, also reported that dilatancy of the unsaturated soil depends on the soil suction, the stress state, and the stress path. It can be seen in Fig. 12 that the maximum dilatancy is found for the test having the lowest Sr (highest suction) and lowest normal stress. This conclusion was also supported by Hossain and Yin (2010) using suction-controlled direct shear tests. Hossain and Yin (2010) further stated that the tests having higher normal stress and lower suction (higher Sr) values exhibited lower or zero dilatancy.

Effect of Sr on cohesion and friction angle

Some of the samples showed a clear peak and then a clear drop in shear resistance; however, some of them reached to a maximum value at large horizontal displacement without a clear peak. Therefore, instead of "peak" shear strength envelope, "failure" term is used to refer to the maximum shear strength. Cohesion and friction angles obtained from the tests are shown in Table 3. In Table 3, the optimum moisture contents of the samples are also presented.

The cohesion intercepts decreased upon increase in Sr (and from the dry side of OMC to the wet side of OMC), and approached to zero for saturated case, except for sample C-2. For C-2 sample, although the cohesion decreased as the Sr increased, 14.3 kPa of cohesion was still available, which is attributed to high compaction dry density (Table 3).

The friction angles decreased upon increase in Sr (and from the dry side of OMC to the wet side of OMC) for fine-grained samples (A and B). For sample A, failure friction angle value decreased from 33.3° at the dry side of OMC to 32.2° at the wet side of OMC. Similarly for sample B, friction angle value decreased from 35.3° at the dry side of OMC to 31.7° at the wet side of OMC. This was also noted by Cokca et al. (2004), where they stated that the friction angle decreases with increasing moisture content up to OMC and with decreasing suction.

As the horizontal displacements are continued and ultimate strength condition is reached, cohesion values decreased dramatically and friction angles increased as compared to the maximum values (at the same Sr) (except for high dry density specimens of sample C-2) (Table 3). For sample C-2, both the cohesion and the friction angle decreased at the ultimate condition.

In order to demonstrate the relation between the peak shear strength and Sr, shear strength is calculated for certain normal stress values (e.g., 20, 40, 60 kPa) using the friction angle and cohesion intercepts of the failure envelopes given in Table 3. Such relations can be seen for samples B and C-1 (Fig. 14), and for other samples similar plots can be seen in Üyetürk (2019). Figure 14 demonstrates that at a given normal stress, shear strength increases with decreasing Sr (i.e., increasing matric suction), and the relationship between shear strength

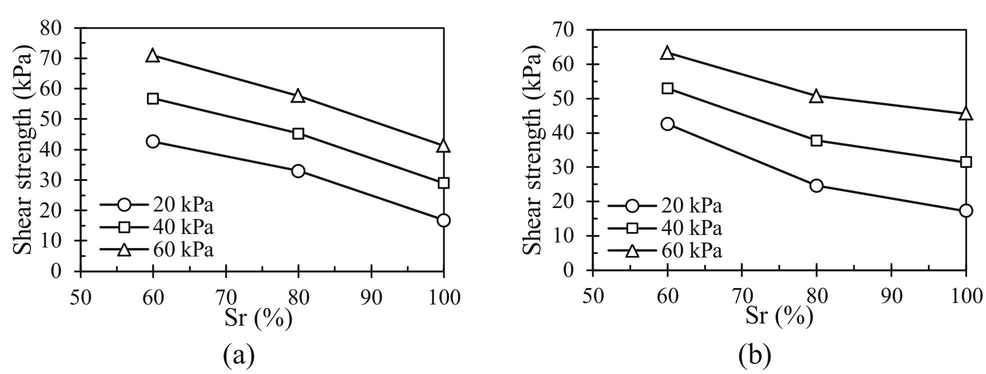


Fig. 14 Shear strength versus Sr relationships **a** for sample B, and **b** for sample C-1

and Sr is nonlinear. Nonlinearity of this relationship was also noted by Escario and Juca (1989) and Jotisankasa and Mairaing (2010).

Concluding remarks

Suction-controlled tests are very time-consuming and suction-controlled direct shear and triaxial apparatuses are available only in limited number of research laboratories. This makes these types of tests not very accessible to practicing engineers and not very suitable for the case when a large number of tests is required to account for natural deposit heterogeneity. In this study, the mechanical behavior of residual soils of volcanic origin are studied by performing constant water content direct shear tests on statically compacted specimens, and the following conclusions are drawn:

- Degree of saturation and normal stress play a key role in mechanical behavior (shear strength and volumetric response) of the soils studied. For example, whether soil exhibits strain-hardening (ductile) or strain-softening (brittle) behavior is controlled by the degree of saturation (or the compaction water content being on the dry/wet side of OMC) and normal stress.
- Shear strength increases with increasing normal stress and decreasing Sr. At low Sr and low normal stress, peak shear resistance is observed. At a given Sr, upon increase in normal stress, strain-hardening behavior is observed rather than strain softening for some samples. Also, under constant normal stress, upon increase in Sr, soils exhibit strain-hardening (ductile) behavior. Specimens tend to become brittle as the Sr and normal stress decreased.
- The volumetric response during shearing changes based on normal stress and Sr. Most of the unsaturated tests exhibited volume increase, whereas most of the saturated tests showed volume decrease. Generally, the dilatancy increased with decreasing Sr and normal stress, i.e., soils become less dilative (or more contractive) upon increase in Sr under the same normal stress. As Sr decreased (as matric suction increased), maximum dilatancy increased. Maximum dilatancy and Sr (matric suction) relationship is nonlinear.
- As for the shear strength envelopes, the failure cohesion values found to increase with decreasing Sr. For fine-grained soils, failure friction angles are found to decrease as Sr increased (from dry side of OMC to wet side of OMC). The relationship between shear strength and Sr is nonlinear.
- For unsaturated tests, at large displacements (ultimate condition) under similar normal stress, similar shear resistances are obtained, which seem to be independent of the Sr. In other words, when the samples are sheared under the same normal stresses, the same ultimate shear resistance is obtained independent of the suction level (or independent of being compacted at the dry or wet side of OMC).
- Ultimate cohesion values calculated at large displacement (ultimate condition) are always smaller than failure (peak) cohesion values. Ultimate friction angles, on the other hand, are mostly found to be greater than the friction angles at the failure (except for sample C-2, having high dry density).

The results of this study could be useful in modeling soil behavior in rainfall-triggered landslide initiation and runout, wetting induced volumetric behavior of embankments and static liquefaction in unsaturated soils.

Acknowledgments The authors would like to thank Assoc. Prof. Dr. Nabi Kartal Toker at the METU Civil Engineering Department for his valuable discussions.

References

Ahmadi-Naghadeh R, Toker NK, Ahmadi-Adli M (2013) Water content controlled instead of suction controlled strength tests. Life Sci J 10(1):2023–2030

Ahmed S, Lovell CW, Diamond S (1974) Pore sizes and strength of a compacted clay: technical paper. Publication FHWA/IN/JHRP-74/02. Joint Highway Research Project, Indiana Department of Transportation and Purdue University, West Lafayette, Indiana. https://doi.org/10.5703/1288284314552

Alan İ, Balcı V, Keskin H, Altun İ, Böke N, Demirbağ H, Arman S, Elibol H, Soyakil M, Kop A, Hanilçi N (2019) Tectonostratigraphic characteristics of the area between Çayeli (Rize) and İspir (Erzurum). Maden Tetkik ve Arama Dergisi 158:1–57

ASTM (2011) Standard test method for direct shear test of soils under consolidated drained conditions, D3080 / D3080M-11. ASTM International, West Conshohocken

Barden L, Sides GR (1970) Engineering behavior and structure of compacted clay. J Soil Mech Found Div

Cokca E, Tilgen HP (2010) Shear strength-suction relationship of compacted Ankara clay. Appl Clay Sci 49(4):400–404

Cokca E, Tilgen HP (2010) Shear strength-suction relationship of compacted Ankara clay. Applied Clay Science 49(2010):400–404

Cokca E, Erol O, Armangil F (2004) Effects of compaction moisture content on the shear strength of an unsaturated clay. Geotech Geol Eng 22(2):285

Collins KT, McGown A (1974) The form and function of microfabric features in a variety of natural soils. Geotechnique 24(2):223–254

Croney D, Coleman JD, Black WPM (1958) Studies of the movement and distribution of water in soil in relation to highway design and performance. HRB spec.Washington D.C. Report 40:226–250

Daniel DE, Benson CH (1990) Water content-density criteria for compacted soil liners. J Geotech Eng 116(12):1811–1830

Delage P, Audiguier M, Cui YJ, Howat MD (1996) Microstructure of a compacted silt. Can Geotech J 33(1):150–158

Escario V, Juca F (1989) Strength and deformation of partly saturated soils. Proc. of 12th International Conference Soil Mech and Found. Eng., v.1 p. 43–46

Fredlund DG, Morgenstern NR (1977) Stress state variables for unsaturated soils. J Geotech Geoenviron 103(5):447–466

Gallage C, Uchimura T (2016) Direct shear testing on unsaturated silty soils to investigate the effects of drying and wetting on shear strength parameters at low suction. J Geotech Geoenviron 142(3): 04015081

Gedik A, Ercan T, Korkmaz S, Karataş S (1992) Rize-Fındıklı Çamlıhemşin arasında (Doğu Karadeniz) yer alan magmatik kayaçların petrolojisi ve Doğu Pontidlerdeki bölgesel yayılımları. Geological Bulletin of Turkey 35:15–38 (**in Turkish**)

Hossain MA, Yin JH (2010) Shear strength and dilative characteristics of an unsaturated compacted completely decomposed granite soil. Can Geotech J 47(10):1112–1126

Huvaj N, Uyeturk E (2018) Effects of drying on Atterberg limits of pyroclastic soils of Northern Turkey. Appl Clay Sci 162:46–56

Jotisankasa A, Mairaing W (2010) Suction-monitored direct shear testing of residual soils from landslide-prone areas. J Geotech Geoenviron 136(3):533–537

Leroueil S, Vaughan PR (1990) The general and congruent effects of structure in natural soils and weak rocks. Géotechnique 40(3): 467–488

Ng CW, Chiu AC (2003) Laboratory study of loose saturated and unsaturated decomposed granitic soil. J Geotech Geoenviron 129(6):550–559

Ng CWW, Zhou RZB (2005) Effects of soil suction on dilatancy of an unsaturated soil. In proceedings of the international conference on soil mechanics and geotechnical engineering (Vol. 16, No. 2). AA Balkema Publishers, p. 559

Oloo SY, Fredlund DG (1996) A method for determination of φb for statically compacted soils. Can Geotech J 33(2):272–280

Picarelli L, Evangelista A, Rolandi G, Paone A, Nicotera MV, Olivares L, Santolo AS, Lampitiello S, Rolandi M (2007) Mechanical properties of pyroclastic soils in Campania region. Charact Eng Prop Nat Soils 3–4:2331–2383. https://doi.org/10.1201/NOE0415426916.ch18

Thu TM, Rahardjo H, Leong EC (2006) Shear strength and pore-water pressure characteristics during constant water content triaxial tests. J Geotech Geoenviron 132(3):411–419

Toll DG (1990) A framework for unsaturated soil behaviour. Géotechnique 40(1):31–44

Toll DG (2000) The influence of fabric on the shear behaviour of unsaturated compacted soils. In Advances in unsaturated geotechnics (pp. 222–234)

Üyetürk CE (2019) Geotechnical characterization of soils prone to rainfall-induced landslides in Rize (northern Turkey). Master of Science dissertation in civil engineering, Middle East Technical University

Vanapalli SK, Fredlund DG, Pufahl DE (1996) The relationship between the soil-water characteristic curve and the unsaturated shear strength of a compacted glacial till. Geotech Test J 19(3):259–268

Wesley LD (1973) Some basic engineering properties of halloysite and allophane clays in Java, Indonesia. Geotechnique 23(4):471–494

Wesley LD (2010) Geotechnical engineering in residual soils. John Wiley & Sons

Wilson SD (1970) Suggested method of test for moisture-density relations of soils using Harvard compaction apparatus. In Special Procedures for Testing Soil and Rock for Engineering Purposes: Fifth Edition. ASTM International