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Effect of grain size on the shear strength of unsaturated silty soils

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Abstract. In this study, shear strength behavior of fine-grained soils was investigated under unsaturated conditions. The samples in the unsaturated state were subjected to a net normal stress (σ-u_a) of 40 kPa and different matric suctions (u_a-u_w) of 50, 100 and 150 kPa. The matric suction values applied in the triaxial tests were selected according to the bubbling pressures determined from the SWC curves. The study was carried out on prepared re-constituted cylindrical samples by uniaxial consolidation of soil slurries. First, consolidated drained (CD) triaxial compression tests were performed on the saturated samples and the cohesion and angle of internal friction were determined. After that, drained triaxial compression tests under matric suctions were performed on the unsaturated samples. In order to obtain unsaturated test results, cohesion and internal friction angle values of saturated samples were used. The nonlinear surface representing the shear strength surface was approximated consisting of two planes (double planar surface). The reason for the nonlinear behavior of some soils is that the amount of sand content contained in it is relatively high and the bubbling pressure/permanent water content value is relatively low.

Keywords: unsaturated soil; shear strength; soil behavior; suction; failure

1. Introduction

Soils are generally considered as saturated in classical soil mechanics and this situation is taken as a conservative basis for the solution of engineering problems. Soils above groundwater level and in capillary saturated zone are called unsaturated soils. In other words, soils with a degree of saturation below 100-95% are called unsaturated soils. Contrary to the classical soil mechanics, most engineering problems are related to unsaturated soils with their voids filled with air and water (Fattah *et al.* 2013).

An important part of our world has arid or semi-arid climate conditions. In such regions, it is almost impossible to reach saturated soil condition. In arid and semi-arid regions, soils above groundwater have negative pore pressure (Hamid 2005). On the other hand, artificially prepared soil fills like earth dams and road foundations are usually compacted soils. The compacted soils are initially unsaturated and have negative pore water pressure (Tripathy *et al.* 2011).

A soil sample has two phases (solid and water) when saturated. There are 4 different phases in a soil sample at the unsaturated condition (solid, water, air and air–water interface). Phase 4, called the air-water interface (which is sometimes named contractile skin), is obtained by exposing the sample to an air pressure greater than the water pressure. The difference between air and water pressure (u_a-u_w) is called matric suction (Fredlund and Morgenstern 1977, Fredlund and Rahardjo 1993, Tilgen 2003).

Shear strength of a soil is the greatest shear stress that the soil can resist without failure. It is necessary to know the shear strength of soils to determine how the soil will behave under applied loads. To estimate the behavior of unsaturated soils, classical soil mechanics developed for saturated soils approach is insufficient. Even in regions where the water level is close to the surface, the foundations of the structure mostly settle on unsaturated soil layers (Özocak 2003). In this regard, it is important to accurately predict the shear strength parameters in the unsaturated state.

Shear strength of unsaturated soils has been investigated by various researchers by modifying different laboratory soil strength test equipment like direct shear test and traditional triaxial compression test (Fredlund *et al.* 1978, Fredlund and Vanapalli 2002, Fredlund and Rahardjo 1993, Zhou *et al.* 2014, Zhou and Xu 2015, Lin *et al.* 2018, Rasool and Aziz 2019). Fredlund *et al.* (1978) proposed the shear strength of unsaturated soils as given in Eq. (1), depending on the net normal stress (σ-u_a) and matric suction (u_a-u_w) variables.

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan\phi' + (u_a - u_w)_f \tan\phi^b \quad (1)$$

In this equation;

c' = intercept of the "extended" Mohr-Coulomb failure

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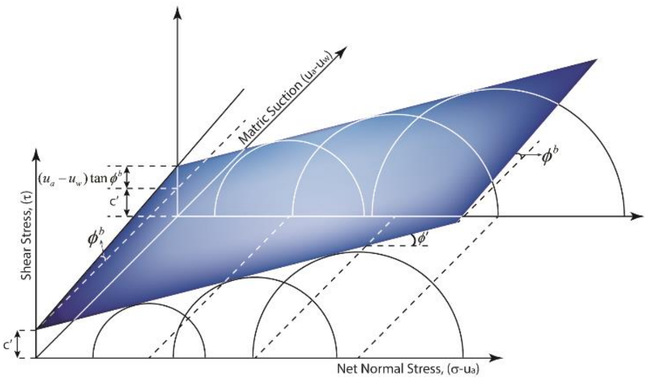

Fig. 1 Extended Mohr-Coulomb failure envelope

envelope on the shear stress axis where the net normal stress and the matric suction at failure are equal to zero (effective cohesion),

$(\sigma_f-u_a)_f$= net normal stress on the failure plane at failure,

$(u_a-u_w)_f$= matric suction on the failure plane at failure,

ϕ^b= angle indicating the rate of increase in shear strength relative to the matric suction,

ϕ'= angle of internal friction associated with the net normal stress state variable.

Soil suction of unsaturated soils (ψ) is composed of two components: osmotic and matric suction. The sum of matric (u_a-u_w) and osmotic suction (π) components is called total suction (Fredlund and Rahardjo 1993).

$$\psi = (u_a - u_w) + \pi \tag{2}$$

Technically matric suction (u_a-u_w) is defined as the difference between pore air pressure (u_a) and pore water pressure (u_w). Osmotic suction component (π) is formed by the fact that the ground water has more energy than the pure water due to the salts of the dissolved substances in the ground water. Edil *et al.* (1981), stated that the matric suction appears to be the fundamental suction component controlling the mechanical behavior during a desorption schedule. The mechanical behavior of unsaturated soils is significantly influenced by matric suction not total suction. Matric suction is highly dependent on the saturation of the sample, its density, grain size and distribution of the grain, in particular the geometry of the voids too (Özocak 2003).

While the only variable that affects the behavior of the samples in saturated state is effective stress, the variables that affect the sample behavior in the case of unsaturated soil are two: matric suction (u_a–u_w) and net normal stress (σ–u_a). These two variables are needed to interpret the behavior of unsaturated soils under applied loads. (Fredlund *et al.* 1978); Handoko *et al.* 2013). Thus, the two-dimensional failure criterion suggested by Mohr- coulomb can be shown as three-dimensional as suggested by Fredlund *et al.* (1978) for unsaturated soils (Fig. 1).

The axis translation technique is used widely to control or measure matric suction of unsaturated soils (Charles *et al.* 2007). There are two independent variables on the basis of axis translation technique. These variables are net normal stress (σ-u_a) and matric suction (u_a-u_w). (Coleman 1962, Frendlund and Morgenstern 1977). As is known, in unsaturated soil environments, air pressure is equal to atmospheric pressure and pore water pressure is negative value. With the help of the axis translation technique, positive air pressure, zero or positive pore water pressure can be applied in the laboratory environment and application and control of matric suction value of desired size is possible.

Sawangsuriya *et al.* (2009a) proposed a relationship describing the small-strain modulus behavior of unsaturated compacted soils. Sawangsuriya *et al.* (2009b) also proposed an empirical relationship relating normalized resilient modulus to matric suction for fine-grained soils. Gupta *et al.* (2007) said that since shear strength is related to deviator stress and the deviator stress can be described as a function of soil suction, one can assume that shear strength will follow some general trend as a function of soil suction.

Cohesion "c'" and slope angles "ϕ'" and "ϕ^b" are strength parameters used to correlate shear strength with stress state variables (Fredlund *et al.* 2012). The slope of the failure envelope in the matric suction axis of unsaturated soils is defined by ϕ^b, where parameter ϕ^b is the angle that indicates the rate of increase in shear strength due to the change in matric suction (Fredlund *et al.* 1978). In order to find the angle ϕ^b, which is one of the shear strength parameters of unsaturated soils, triaxial test results on unsaturated samples at different suction values is used.

Zhao and Zhang (2013) investigated the critical state characteristics by using a triaxial pressure plate to obtain the soil-water characteristic curve (SWCC). Axial translation technique was used for matric suction stresses greater than 10 kPa. Sudden water losses occur in these samples due to the fact that the coarse-grained soils have less water retention capabilities than fine-grained soils. Therefore, the SWCC of coarse grained soils can have two air entry values (AEV). Nuntasarn and Wannakul (2017) stated that SWCCs with double (*bimodal*) air inlet values may be compressed during dry preparation.

The time and effort required to find the shear strength of unsaturated soils are generally greater than that of the conventional shear strength of saturated soils. Nam *et al.* (2011) in their experiments carried out at a suction value of greater than 200 kPa, they washed out the accumulated air in the system every 8 to 12 hours. During the consolidation, the balancing phase lasted between 1 and 3 days at air pressure values less than 100 kPa and between 5 and 12 days at air pressure values higher than 100 kPa. Ho and Fredlund (1982) conducted multi-stage unsaturated soil tests on soils with high permeability. They also stated that the test periods are prolonged if unsaturated soil procedure is applied for soils with low permeability.

Vanapalli *et al.* (2008) said that the axis translation technique and negative water column techniques are generally used in experimental studies in order to obtain the data of interpretation and engineering behavior of unsaturated soils. In their studies, they mentioned the limitations of air diffusion, water volume change and evaporation related to axis translation.

Table 1 presents the summary of the studies on unsaturated soils other than those described above.

Table 1 Examples of multistage studies in unsaturated soil conditions

Literature	Test equipment	Soil type	Stress (kPa)		Descriptions
			Net	Matric	
Vanapalli *et al.* (2008)	TX	Silty Sand (SM)	40	100	They were defined that the negative water column technique is more useful in the lower suction range (0-30 kPa) and the axis translation technique is more useful in larger suction ranges (0-500 kPa).
Nam *et al.* (2011)	DST	SM, MH and CL	43 68	25 50 100 200 290	There is a nonlinear relationship between the matric suction value and shear strength. The friction angle according to matric suction ϕ^b is higher than the effective friction angle (ϕ') at matric suction values below the air entry value (AEV). ϕ^b is decreased as matric suction higher than AEV value.
Rahardjo *et al.* (2004)	TX (CD) (CW)	Sandy clay (CL)	50 100 150	0 50 100 150 290	In triaxial tests, samples exhibited normal consolidated behavior at suction values below the air entry value. They exhibited over consolidated behavior at suction values higher than the air entry value. The clay samples continued to take load after reaching the highest value according to the shear test results. This result was explained as the clayey soils act as a whole with flocculation.
Farouk *et al.* (2004)	TX	Sand	50 150 300	0 30 50 150	Increase in matric suction did not affect the relationship between stress and strain. Therefore, the stress-strain graphs obtained from saturated samples are similar. When net normal stress (σ–u_a) was applied, the shear strength of the unsaturated sample was higher than that of the saturated sample.
Hamid and Miller (2009)	DST	Lean clay (CL)	105 140 210	20 50 100	Changes in matric suction ratio do not affect post peak values after the highest shear strength, however, the changes in net normal stress affect the values after the highest strength value.
Sun *et al.* (2007)	DST	Sandy Silt	12.3 24.6 49.2 98.4	0 11 22 28 35	ϕ^b value decreases with increasing matric suction. ϕ^b angle is said to be lower than the ϕ' value of saturated soils. It is said that the angle ϕ^b can be calculated by formula according to the matric suction and net normal stress value of the saturated and unsaturated soil sample. Also the friction angle (ϕ^b) was calculated from the SWCC.
Rassam and Williams (1999)	TX	Silt	30 125 250	0 20 60 100	There is a linear relationship between matric suction and shear strength until the air entry value. Volume changes occur in the sample after the air entry value. As the matric suction value increases the shear strength value shows a nonlinear increase.
Nuntasarn and Wannakul (2017)	TX (CD)	Silty sand (SM)	100	30 180 280	They found cohesion value of 54 kPa and effective friction angle of 310 from saturated samples. And they found a linear relationship between matric suction and shear strength in unsaturated samples and found the angle ϕ^b of 28^0.

TX=Triaxial Test, DST=Direct Shear Test

In this study, it was aimed to investigate the shear resistance behavior of unsaturated fine-grained soils with varying sand content. For this purpose, 4 different natural fine-grained samples were obtained. Although all of the samples are classified as clay, they have different sand contents. Therefore, it was possible to evaluate the effect of the clay/sand content in the investigation of the shear resistance behavior of unsaturated fine-grained soils.

2. Materials

In lifecycle modelling of corrosion damaged RC structures serving in aggressive environments, the effect of corrosion on the resistance of the RC structures can be illustrated in Fig. 1 (Chen and Alani 2013). In this paper, the lifecycle of a RC structure subjected to reinforcement corrosion is defined as the period for the completion of construction to the collapse of the structure. As observed from Fig. 1, in the crack initiation phase, structural resistance remains almost the same as the original capacity. In literature research, it is seen that the experiments on unsaturated soils are mostly made with coarse-grained materials. In this study, fine-grained soils were studied and four different fine-grained soil samples were used. The samples used were obtained from different parts of Adapazarı city center (Turkey) where many problems caused by the soil during 1999 earthquake. Taking the samples from the same region is thought to be an advantage as the geological origins are the same. These soils are fluvial deposits in the sedimentation basin of the Sakarya River. According to USCS (Unified Soil Classification System) and TS1500 (Turkish Standart, 2000) classification system; two of these samples are classified as non-plastic silt (ML). The other two samples are classified as medium plasticity clay ($35<w_L<50$)

In this study, triaxial compression tests were conducted on both saturated and unsaturated samples prepared by slurry consolidation method. In addition, it was possible to observe and evaluate the 3-D stress failure surface.

(CI) according to TS1500 and low plastic clay (CL) according to USCS. The physical properties of the samples are shown in Table 2. The clay and sand contents of all samples are different as shown in this table. Sample numbers (S) are ordered according to increasing sand content. S-1 has the lowest sand content (1%) and S-4 has the highest sand content (23%). This situation is the opposite in terms of clay content.

S-2, S-3 and S-4 samples are natural samples obtained from the field. S-1 sample was reconstituted from the natural samples (S-2, S-3, S-4) by sedimentation method. For this purpose, these natural samples were mixed in a large steel vessel and left to settle. With time segregation occurred, i.e., coarse grains (sand) deposited first, then medium grains (silt) and the last smallest size (clay)

Table 2 Soil properties used in experimental studies

Sample number	S-1	S-2	S-3	S-4
Soil class (TS 1500)*	CI	CI	ML	ML
Soil class (USCS)**	CL	CL	ML	ML
Liquid limit (w_L)	40	36	33	34
Plastic limit (P_L)	24	21	NP	NP
Plasticity index (I_p)	31	15	NP	NP
Specific gravity (G_s)	2.66	2.76	2.67	2.78
Clay content (%)	22	18	14	10
Silt content (%)	77	78	68	67
Sand content (%)	1	4	18	23

*TS 1500: Turkish Standart of Soil Classificaitons in Cvil Engineering

**USCS: Unified Soil Classification System

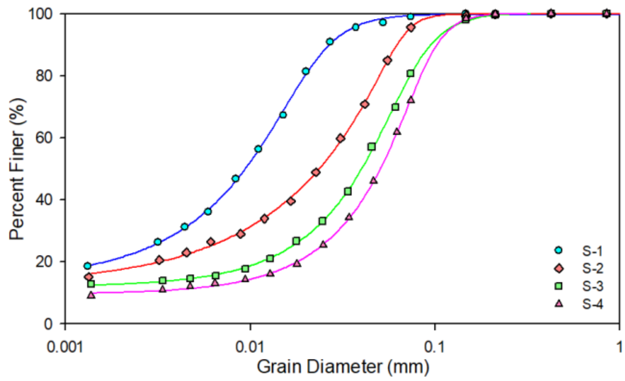

Fig. 2 Grain size distribution of samples

particles. The top layer of sediment with the highest clay/silt content was collected in a separate container and designated as S-1.

Fig. 2 shows the grain size distribution of the samples used in the experimental study. The defining criterion for the samples whose physical properties have been determined is the percentages of clay and sand. As the clay content decreases while the sand content increases.

3. Sample preparation method

In this study reconstituted samples were prepared under one dimensional slurry consolidation. This method has long been used to obtain reconstituted samples (Liu *et al* 2017, Kuerbis and Vaid 1988, Carraro and Prezzi 2007, Hyde and Ward 1985, Yasuhara *et al.* 2003, Sheeran and Krizek 1971, Krizek *et al.* 1975). For this, dry sample weights were calculated to generate specimens having a diameter of 50 mm and a height of 100 mm. These samples were slurried at a water content of 1.5 times the liquid limit value. The slurry was subjected to vacuum to remove any air in the sample after mixing to ensure homogeneity in the slurry. All samples were consolidated under a vertical stress of 100 kPa to induce the same consolidation stress histories. This consolidation process was carried out with a special system designed in this study (Fig. 3). In this system, porous stone

Fig. 3 One-dimensional consolidation stage of samples

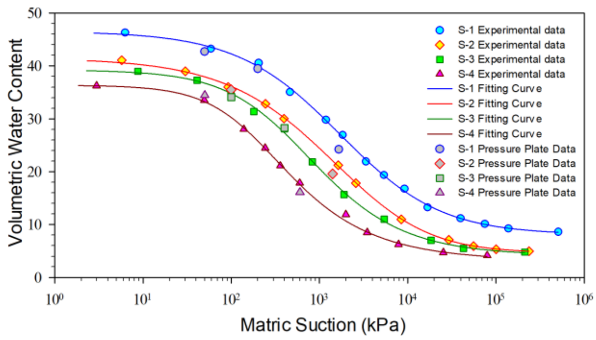

Fig. 4 SWCC obtained with pressure plate and filter paper method

Table 3 SWCC test results

	$(u_a\text{-}u_w)_{ae}$ (kPa)	θ_{aev} (%)	$(u_a\text{-}u_w)_r$ (kPa)	θ_r (%)
S-1	110	46	18000	10.2
S-2	90	41	13000	6.8
S-3	80	39	6800	6.5
S-4	50	36.2	2700	6.3

and filter paper were placed on the base of a plexiglass cell with an internal diameter of 50 mm. Then slurry was placed on the filter paper in the cell. After placing the filter paper, porous stone and loading cap on top of the sample, the upper cell cover was closed. After the loading piston was placed, the slurry was consolidated for some time under the weight of the loading cap and the piston. During this period, water outflow and settlement were monitored. When the settlement became neglible and the water outflow stopped 100 kPa load was gradually applied to the sample via the loading beam. Again the water outflow and settlement were monitored. The specimen was taken out after both the water outflow and settlement were completed. At the end of the process, the specimen was removed from the plexiglass cell and placed in a triaxial cell.

The air entry water content (θ_{aev}) and the residual water content (θ_r) of the unsaturated samples can be determined by generating the SWCC. Fig. 4 shows the SWCC of four different samples. Pressure plate and especially filter paper method were used to determine SWCC. Filter paper technique is a method that allows measurement in the range of 10 to 10^6 kPa for fine-grained soils and was performed in accordance with ASTM D5298 standard. Pressure plate tests were carried out according to ASTM D6836. The results given in Table 3 were obtained according to the SWCC plotted. According to these results, it was founded that air entry value decreased with increasing amount of sand in the sample. The results show that samples with more coarse grain content achieve the same matric suction value at lower water contents than other samples.

Table 3 determined from the graphics shows that the sample with the highest residual water content and bubbling pressure/air entry value is the S-1, which has the highest liquid limit value and the lowest sand content. The air entry values obtained were used as the limit for the matric suctions in the triaxial cell for these samples. The matric suctions applied in the triaxial compression tests were selected above these values. The air entry value indicates the lower limit of matric suctions, which causes the air become continuous in the soil pore structure. Therefore, the matric suction values used to determine the shear strength of unsaturated soils were kept above 50 kPa.

4. Shear strength parameters of saturated samples

In order to determine the shear strength of unsaturated soils, it is necessary first to carry out the tests on saturated soil and to determine the drained shear strength parameters. Conventional multi-stage consolidated-drained triaxial compression tests (CD) were performed on the saturated specimens to determine the shear strength parameters of the samples used in this study. Saturation procedure was performed for the specimen placed in the cell until a B check value exceeding 0.95 was obtained. Consolidated-drained tests (CD) were carried out in multiple stages on a single specimen. The method recommended in ASTM (D7181) was used in deciding the shear rate. Accordingly the shear rate was set at 0.01 mm/min for all specimens. Additionally, the adequacy of this shear rate to assure drained, i.e., no excess pore pressure during shearing was checked by observing the pore water pressure values during the shearing.

For the saturated specimens used in this study, net confining stresses and deviator stresses used in each experiment are presented in Table 4. The method proposed by Ho and Fredlund (1982) was followed in deciding when to end each shearing step of multistage tests. In this method, the deviator stress-deformation graph is plotted and the loading is terminated as soon as the graph reaches its peak. The stress-strain graphs of each saturated specimen are given in Fig. 5 collectively. The curves display the soft soil shear behavior.

The test results obtained from these graphs are summarized in Table 4. When these tables and graphs are evaluated together, the following observations are made.

Comparison of the peak deviator stresses at the end of third shearing stage indicates that as the sand content increases shearing resistance increases, i.e., the effective friction angle increases with increasing sand content. The cohesion intercept was zero for all samples.

In order to determine the drained strength parameters of the sand fraction, a slurry of the samples (S-2, S-3 and S-4) was made by adding water. This slurry was allowed to settle. This process was repeated 5 times and the bottom sand fraction free of finer particles was collected. Drained triaxial compression tests (CD) were carried out on this accumulated sand fraction. Fig. 6 shows the results of the CD tests corresponding to the maximum (e_{max} =1,03) and the minimum (e_{min} =0,64) void ratios. The friction angle for

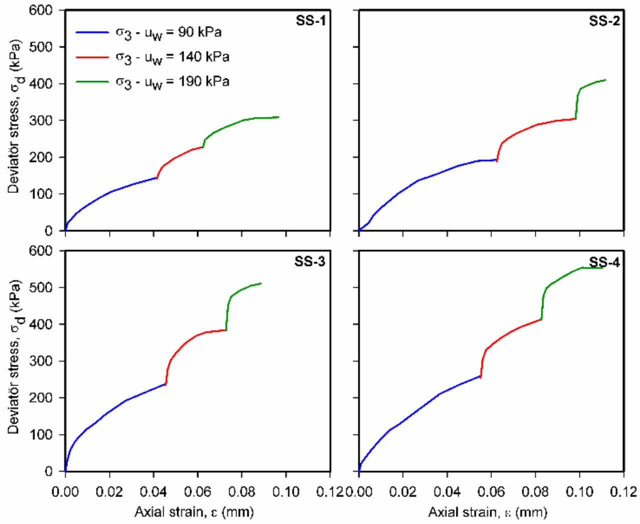

Fig. 5 Shear strength results for saturated soil samples

Table 4 Saturated soil samples test results (u_{back}=50 kPa)

Sample No	Shear Stage	σ_3 (kPa)	σ_3-u_w (kPa)	σ_d (kPa)	c' (kPa)	ϕ' (deg.)
	1	140	90	145		
SS-1	2	190	140	228	0	27
	3	240	190	308		
	1	140	90	193		
SS-2	2	190	140	304	0	31
	3	240	190	410		
	1	140	90	238		
SS-3	2	190	140	384	0	35
	3	240	190	511		
	1	140	90	259		
SS-4	2	190	140	413	0	37
	3	240	190	554		

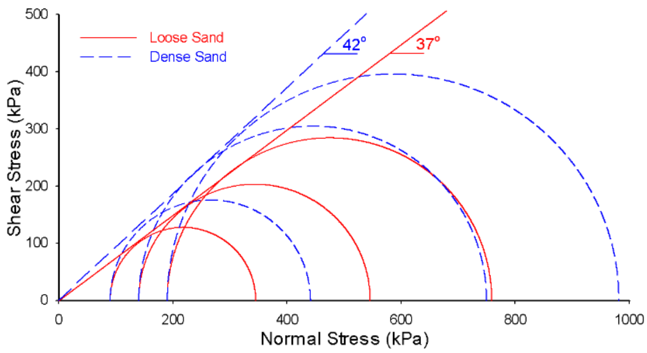

Fig. 6 Friction angles for the loose and the dense sand fraction

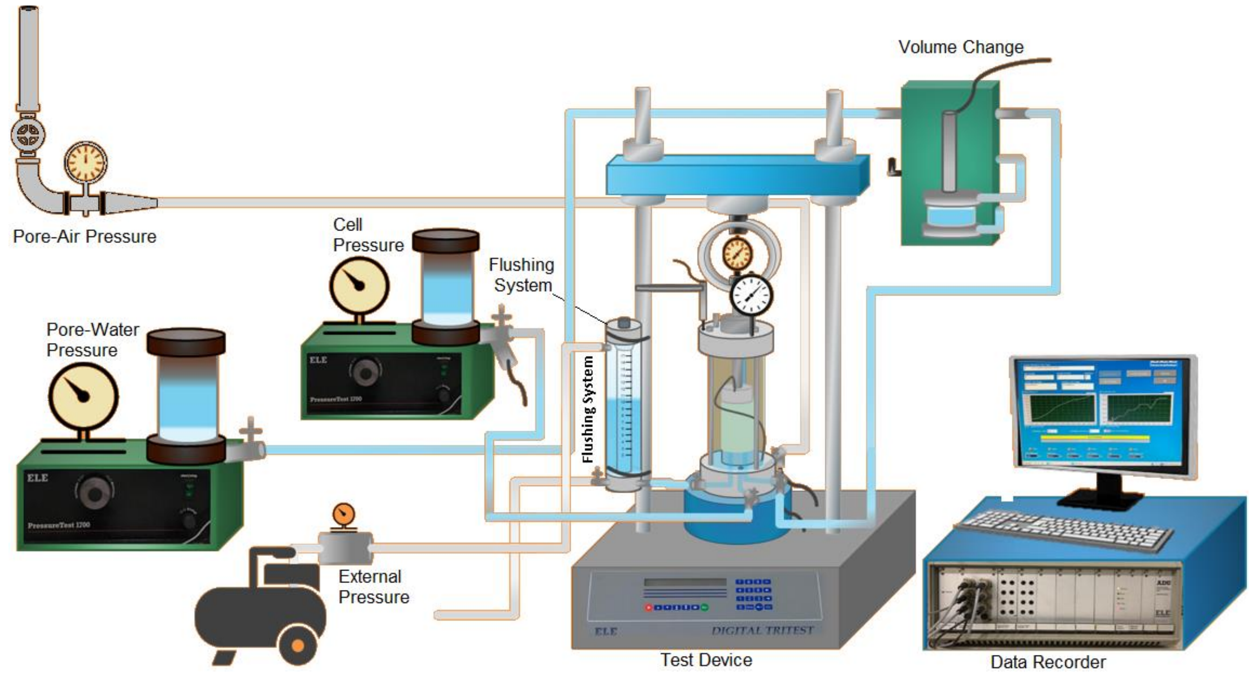

Fig. 7 Triaxial compression test device used for unsaturated samples

the loose and the dense sand condition are 37° and 42°, respectively. In the SS-4 sample, which has the highest sand ratio (23%), the angle of shear resistance (37°) approaches these values of the loose sand.

5. Experimental procedure for unsaturated samples

Unsaturated soil behavior can be successfully followed by matric suction and net normal stress variables (Fredlund and Morgenstern 1977)). Consolidated-Drained (CD) test method was used to obtain the shear strength parameters of unsaturated fine-grained soil samples similar to the procedure for the saturated samples. Unlike the saturated test system; the air and water pressure system is incorporated to the test device to use the axis translation technique to ensure that the sample has the desired matric suction. Fig. 7 shows the schematic representation of the experimental system used in this study.

For the axis translation technique, the air pressure was connected to the triaxial test cell through a valve connected to the top of the specimen. Pore water pressure was provided from the sample base. The tests of unsaturated samples were carried out in multiple stages, similar to saturated samples, to eliminate changes in the specimens and to obtain a lot of data with a small number of specimens.

Pore-air and pore-water pressures need to be measured or controlled to determine the mechanical behavior of unsaturated soils. Hence, a special ceramic disc, which do not allow the air drainage but allow the water drainage is required in the pedestal. The air entry value of the ceramic disc used in this test was 500 kPa (5 bars) and its thickness was 7.25 mm. Pore water pressures were measured by a 1700 kPa capacity transducer placed under the ceramic disc. The air pressure to be used in these systems must be

Table 5 Pressure values of unsaturated soil triaxial multistage shear test

Stage	σ_3 (kPa)	u_a (kPa)	u_w (kPa)	(σ_3-u_a) (kPa)	(u_a-u_w) (kPa)
I	140	100	50	40	50
II	190	150	50	40	100
III	290	200	50	40	150

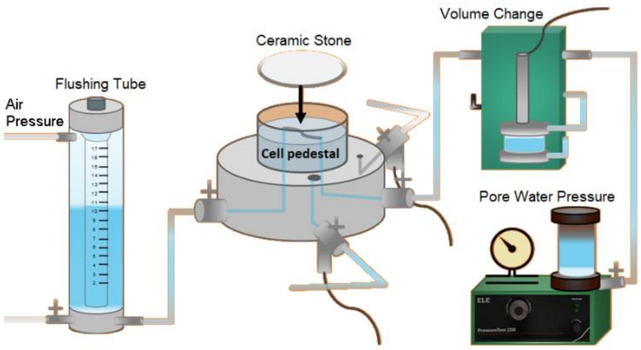

Fig. 8 Schematic illustration of the flushing system

controlled and this control was provided by the air pressure line connected to the loading cap. A 6.11-mm thick porous stone was used between the loading cap and the sample.

The dissolved air contained in the pressurized water applied through the sample base can turn to free air state during the test and accumulate under the ceramic disc. So it can prevent water passing between the sample and water reservoir in the pedestal and may cause incorrect water pressure readings. Air accumulated under the ceramic disc must be removed to correct the drained water readings through a flushing system. Thereby it is possible to make a correction for the water volume change. The flushing system used this study is shown in Fig. 8. Air supply that is required both for the flushing system and for controlling

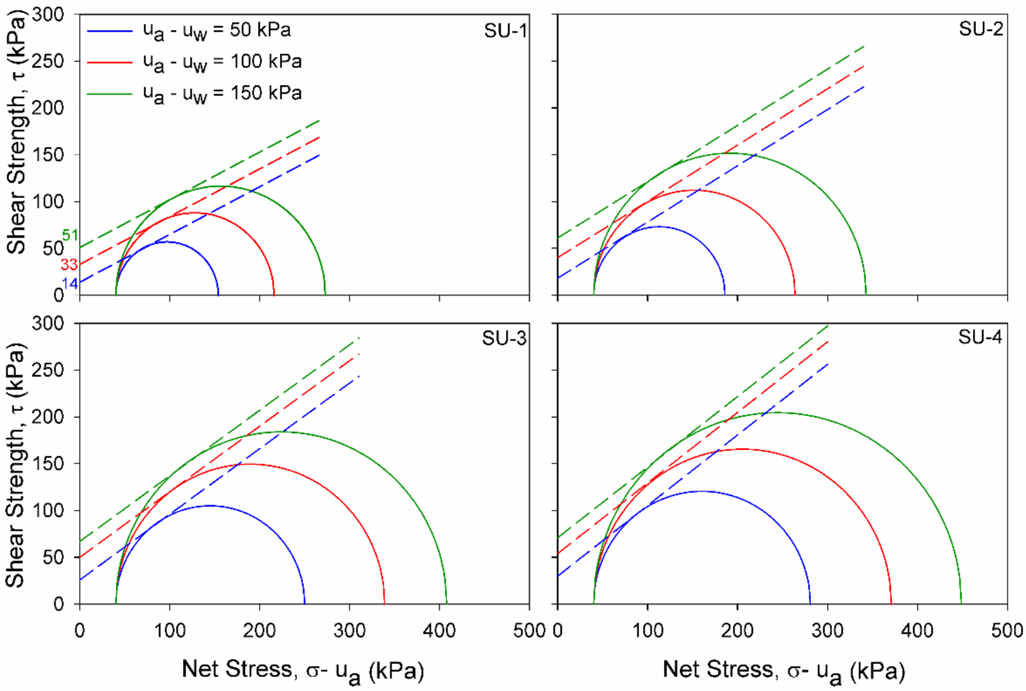

Fig. 9 Effect of matric suction on apparent cohesion on unsaturated samples

Table 6 Test results for unsaturated soil samples (u_w=50 kPa, σ_3-u_a=40 kPa)

Sample No	Shear Stage	σ_3 (kPa)	u_a (kPa)	u_a-u_w (kPa)	σ_d (kPa)	c' (kPa)	ϕ^b (deg.)
SU-1	1	140	100	50	114	14	18
	2	190	150	100	176	33	
	3	240	200	150	231	51	
SU-2	1	140	100	50	146	17	22
	2	190	150	100	224	40	
	3	240	200	150	299	60	
SU-3	1	140	100	50	207	25	25
	2	190	150	100	294	48	
	3	240	200	150	360	67	
SU-4	1	140	100	50	241	30	37/18(bilinear)
	2	190	150	100	326	52	
	3	240	200	150	400	70	

the matric suction was provided by an external compressor. It is stated that the axis translation technique is suitable for matric suctions over 100 kPa (Bocking and Fredlund 1980, Ho and Fredlund 1982).

After the cell was filled with water, the specimen was allowed to consolidate under the pressures shown in Table 5. The volume of water draining from specimen during the consolidation step was measured by the volume change apparatus. The consolidation phase was terminated when the water draining from the specimen stopped. Since the air accumulated under the ceramic disc during the consolidation phase cannot pass through the ceramic disc, it is discharged via the flushing system. The desired air pressure (u_a) was applied from the top of the specimen and the matric suction (u_a-u_w) application step was initiated. At this stage, when the water draining from the specimen stops, the specimen is considered in equilibrium at the desired matric suction value. The multi-stage shearing phase of the specimen that reached equilibrium then began by applying pressures as shown in Table 5.

5.1 Shear strength of unsaturated samples

In unsaturated soil tests, four different soil samples with the same characteristics as saturated samples were used. To these samples 40 kPa net normal stress and 50, 100 and 150 kPa matric suction were applied. Table 6 shows the mechanical test results of the unsaturated soils. When the results of all samples were examined, SU-4, which had the highest sand content, has the highest strength similar to the saturated state. The maximum deviator stress value reached by the SU-4 is 400 kPa. The deviator stresses were examined for saturated and unsaturated experiments. The matric suction term (u_a-u_w) in Eq. (1), which gives the shear strength of unsaturated soils, is defined as the contribution to total or apparent cohesion (C) [Eq. (3)] (Ho and Fredlund 1982). In other words; cohesion increases as matric suction increases on unsaturated soils.

$$C = c' + (u_a - u_w)_f \tan\phi^b \tag{3}$$

The tangent with the friction angle in saturated state drawn to each Mohr circle obtained in the unsaturated state gives the apparent cohesion. Fig. 9 shows the Mohr circles obtained for SU-1, SU-2, SU-3 and SU-4.

According to the graphs in Fig. 9, SU-1 with the highest amount of clay gives the lowest cohesion value, while SU-4 with the least clay content gives the highest cohesion. Apparent cohesion value increases with increasing matric suction. The effect of this increase in shear strength with

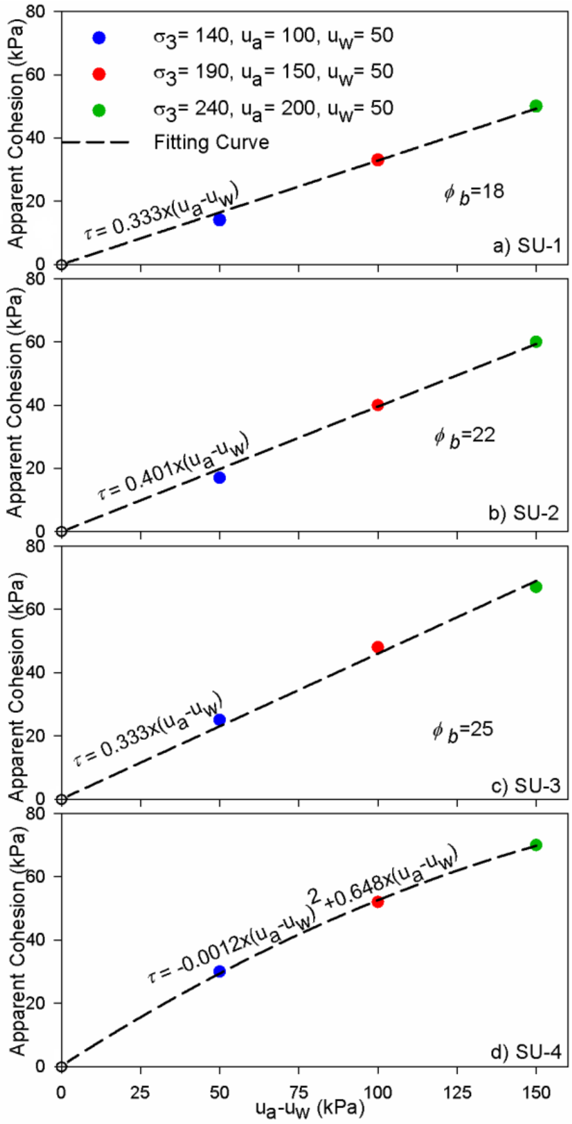

Fig. 10 Apparent cohesion versus matric suction (the cohesion of saturated samples are shown as open symbols at 0 suction)

increasing matric suction values is higher on specimens with high sand content.

Fig. 10 shows apparent cohesion versus matric suction (u_a-u_w) for unsaturated soils. All samples show essentially a linear relationship within the suction range used in this program, i.e., 50-150 kPa. However, SU-4 shows slight non-linearity. It is also possible the relationship may become non-linear beyond a suction of 150 kPa. The air entry values varied between 50-110 kPa for these samples. In the literature, there are studies suggesting that there may be bilinear behavior depending on the applied suction relative to the air entry value (Zhao *et al.* 2013).

Vanapalli *et al.* (1996), Fredlund *et al.* (1987), Rassam and Cook (2002) and Fredlund *et al.* (2012) indicated that shear strength increases linearly up to the air entry value obtained from the SWCC and it increases non-linearly between the air entry value and the points corresponding to the residual water content. They indicated that after the residual suction, shear strength could increase, decrease or remain constant. It is reported that shear strength may decrease after the residual suction value especially for sandy and silty soils. When the samples reach the residual water content value generally very little water remains in the pores. Especially in sands and silts the residual suction values may be very low. In contrast, clays can contribute to shear strength even at high suction values and contain some water even at the residual suction value. Because of high silt contents SU-1 and SU-2 gave low air entry values in this study, i.e., 110 and 90 kPa, respectively.

According to the results of triaxial tests on clay soils, the shear envelope is expected to be linear. This linearity is due to the high air entry values of clayey soils. Chen and Hai (2012) reported that the fracture envelope of coarse-grained soils may not be linear due to their lower suction values at the residual water content. However, the same investigators state that the non-linearity of the failure envelope may vary due to the net normal stress applied in testing.

The failure envelopes of all unsaturated samples used in the study are plotted in Fig. 11. In these figures, increases in shear strength of the samples due to matric suction can be seen. The failure envelopes drawn for SU-1, SU-2 and SU-3

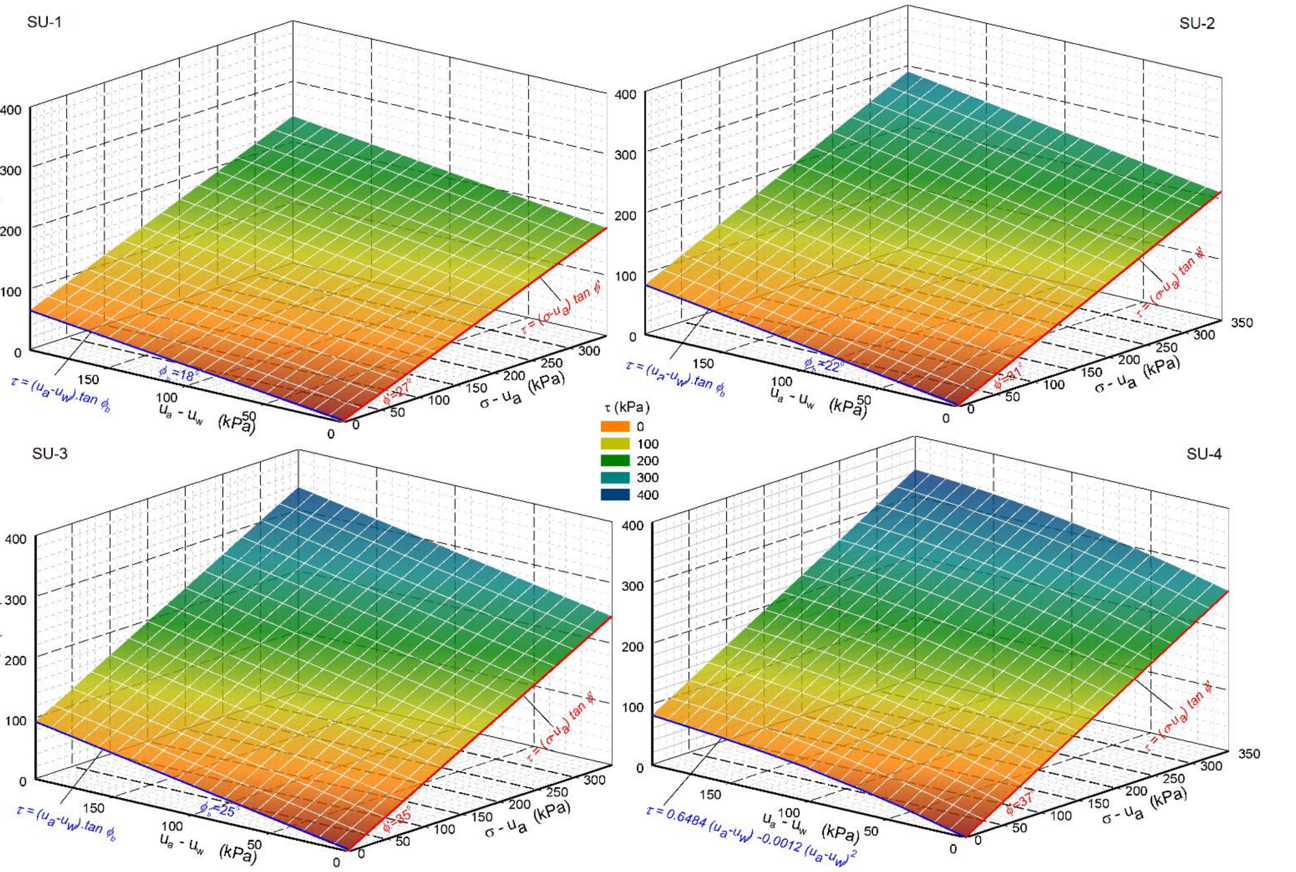

Fig. 11 Failure surfaces of four unsaturated samples

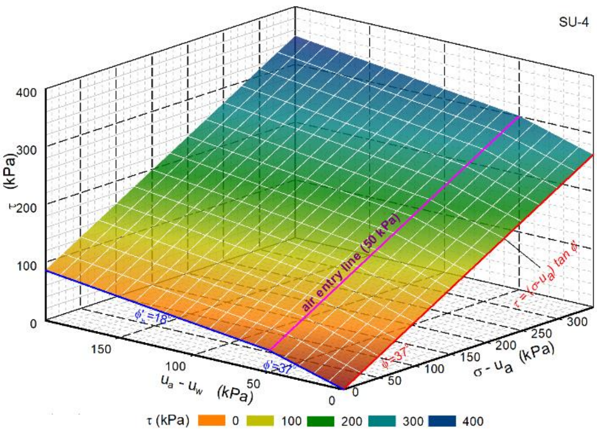

Fig. 12 Double planar surface

are highly linear, while the failure envelope of SU-4 with the highest sand content is slightly nonlinear. The bubbling pressure/residual water content is relatively low for SU-4. The failure envelopes plotted for all four specimens and the obtained ϕ^b values are shown in Fig. 11. ϕ^b increases with increasing amount of sand in the sample. Shear strength behavior of the samples is drawn in 3D. The shear strength (ϕ^b) value of the sample with a nonlinear failure envelope is represented by a simple bi-linear model. In this case, the general equation of the surface of the sample showing nonlinear behavior is given as

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan\phi' + a(u_a - u_w) - b(u_a - u_w)^2 \quad (4)$$

Here the coefficients a and b are real coefficients of the quadratic equation appearing on the τ - (u_a-u_w) axis. The apparent cohesion value of SU-4 soil used in this study was obtained as 0. The mathematical model of this sample exhibiting nonlinear behavior is presented in Eq. (5).

$$\tau_{ff} = (\sigma_f - u_a)_f \tan 37 + 0.648(u_a - u_w) - 0.0012(u_a - u_w)^2 \quad (5)$$

In Fig. 12, the failure envelope of unsaturated soils showing nonlinear behavior is demonstrated using bi-linear envelopes with two different angles. In this representation the soil exhibits saturated behavior at matric suction values up to the air entry value. The angle (ϕ^b) representing the increase in shear strength due to matric suction is then equal to the shear strength angle (ϕ') of the soil up to the air entry value (37° in this example). In a second plane that will appear after the air entry value, the failure surface continues with a lower and constant angle ϕ^b (18° in this example). There is an advantage in representing the nonlinear surface two planes (double planar surface) as shown here. This evaluation also shows a strong agreement with the study results in the literature (Fredlund *et al.* 1978, Gan *et al.*1988, Oloo *et al.* 1997). The equation for shear strength of an unsaturated soil at zero net normal stress, based on a bilinear envelope, is then given by

$$\tau_{ff} = c' + (u_a - u_w)\tan\phi' + [(u_a - u_w) - (u_a - u_w)_b]\tan\phi^b \quad (6)$$

where $\phi^b = \phi'$ if $(u_a\text{-}u_w) \leq (u_a\text{-}u_w)_b$ and $(u_a\text{-}u_w)_b$ is the air-entry value (Gan *et al.* 1988).

The entire shear strength, τ, at zero net normal stress given above is considered the total cohesion of the unsaturated soil. If the net normal stress, (σ-u_a), is greater than zero, the general equation for shear strength for an unsaturated soil based on a bilinear envelope becomes

$$\tau_{ff} = \left[c' + (u_a - u_w)\tan\phi' + [(u_a - u_w) - (u_a - u_w)_b]\tan\phi^b\right] + (\sigma - u_a)\tan\phi' \tag{7}$$

(Gan *et al.* 1988, Oloo *et al.* 1997). The apparent cohesion of the sand and clay show a unimodal distribution. The apparent cohesion increases linearly up to the AEV. In the transition zone, the apparent cohesion increases nonlinearly as water drains from the soil (Zhao *et al.* 2013).

6. Conclusions

Shear strength behavior of fine-grained soils with varying amounts of sand content was investigated under unsaturated conditions in this study.

The study was carried out on re-constituted cylindrical specimens prepared by one-dimensional consolidation of soil slurry. Consolidated-drained triaxial compression tests (CD) were performed on saturated specimens and the cohesion and angle of internal friction were determined. Drained triaxial compression tests under varying matric suctions were performed on unsaturated specimens. In order to obtain unsaturated strength parameters, cohesion and internal friction angle values of saturated specimens were used.

The initial values of the matric suction were obtained from the SWCC by the filter paper method. The samples in the unsaturated state were subjected to a net normal stress (σ-u_a) of 40 kPa and tested at different matric suctions (u_a-u_w) of 50, 100 and 150 kPa. The matric suction values applied in the triaxial compression tests were selected according to the bubbling pressures determined from the soil-water characteristic curves (SWCC).

The following conclusions are advanced:

(1) The effective friction angle increases as the percentage of sand increases for saturated soils. All saturated samples showed zero effective cohesion. However, cohesion increased with increasing suction for unsaturated samples. Shear strength angles (ϕ^b) that reflect the effect of matric suction increased as the percentage of sand in the samples content increased.

(2) Failure surfaces of unsaturated soil were mostly linear except for the soil with the highest sand content (23%). This behavior is attributed to relatively low bubbling pressure/permanent water content associated with high sand content. Some researchers observed that in general, the rate of increase in shear resistance decreases with increasing matric suction beyond the air entry value in unsaturated silts and sandy silts resulting in a non-linear failure envelope (Zhao *et al.* 2013). In this study, SU-4 with the highest sand content showed some similarity to the behaviour described in the literature. This naturally affects the apparent cohesion value. The increase in the sand ratio to 23% in SU-4 made this behavior evident in these samples.

(3) The nonlinear surface representing the shear strength can be represented consisting of two planes (double planar surface) and a simple mathematical formulation.

This study is limited to investigating the shear resistance variation of unsaturated fine-grained soils with different sand content. The test process may be repeated in order to make interpretation for soils with other mixture contents.

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