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TECHNICAL NOTE

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Water Content of Moist-Tamped Nonplastic Specimens for Constant-Volume Direct Simple Shear Testing

Reference

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ABSTRACT

Constant-volume direct simple shear (DSS) tests on dry soils are an efficient means to evaluate the behavior of cohesionless soils. However, cohesionless soil specimens prepared by dry techniques (e.g., air pluviation) may not be sufficiently contractive to directly evaluate the critical-state shear strength mobilized in monotonic loading. As an alternative, moist tamping allows preparation of loose specimens that exhibit contractive behavior throughout monotonic loading which, where feasible, can maximize testing efficiency in studying static liquefaction. This paper presents a theoretical background for this specimen preparation protocol and validates the approach using DSS test results for three nonplastic soils with various fines content (0 %, 15 %, and 60 %). The results illustrate that if the specimen preparation water content is selected to minimize initial soil suction within the moist-tamped specimen, then the effect of initial soil suction on shear resistance measured in constant-volume DSS testing is negligible when the specimen reaches the critical state. Thus, constant-volume DSS testing performed on moist-tamped nonplastic specimens without saturation appears to provide a feasible alternative to maximize testing efficiency for critical-state behavior studies.

Keywords

specimen preparation, nonplastic, moist tamping, simple shear

Introduction

Direct simple shear (DSS) testing has become a common laboratory testing technique for evaluating soil behavior. Among all testing protocols, constant-volume DSS tests on dry or saturated specimens often are used to replicate truly undrained tests on saturated specimens due to difficulties in achieving that drainage condition in many DSS devices (Boulanger 1990). The primary advantage of dry specimens is the ease of specimen preparation and therefore its efficiency for testing programs. In a constant-volume DSS test, it is assumed that the change in applied vertical stress equals the excess porewater pressure that would be generated in a truly undrained test with a constant vertical load (Bjerrum and Landva 1966; Iversen 1977; Dyvik et al. 1987).

Numerous laboratory specimen preparation methods have been proposed for testing reconstituted nonplastic soils, including air pluviation using either a funnel or a soil rainer (e.g., Vaid and Negussey 1988; Vaid, Sivathayalan, and Stedman 1999; Ghionna and Porcino 2006; Wood, Yamamuro, and Lade 2008; Sadrekarimi and Olson 2012; Bhaumik et al. 2020), water sedimentation/pluviation (e.g., Vaid and Negussey 1988; Ishihara 1993; Vaid, Sivathayalan, and Stedman 1999; Ghionna and Porcino 2006; Wood, Yamamuro, and Lade 2008), moist tamping (e.g., Ladd 1978; Ishihara 1993; Vaid, Sivathayalan, and Stedman 1999; Bradshaw and Baxter 2007; Sadrekarimi and Olson 2012; Jefferies and Been 2015; Polito 2017), and slurry deposition (e.g., Bradshaw and Baxter 2007; Carraro and Prezzi 2008; Wood, Yamamuro, and Lade 2008; Krage et al. 2020). Selecting a preparation method depends on various factors, including the fines content of the soil and the target specimen state.

When evaluating static liquefaction and critical-state behavior, moist tamping often is the only method that produces specimens that are sufficiently loose to remain contractive through monotonic shearing at moderate stress levels such that the critical state can be achieved within the displacement limit of the testing device (Ishihara 1993; Vaid, Sivathayalan, and Stedman 1999; Sadrekarimi and Olson 2012; Jefferies and Been 2015). This is especially true when the soil is relatively incompressible. In many studies, dry soil is thoroughly mixed with water to produce a gravimetric water content, w, of 5 % (mass of water per mass of dry soil). The moist soil then is tamped in layers into the specimen mold. The widely used 5 % water content is generally considered adequate to develop capillary forces among particles and allow creation of specimens at very high void ratios that can exceed the maximum void ratio determined per ASTM D4254-16, *Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*. In this study, the authors propose a consistent testing preparation water content protocol for using the moist-tamping method to reconstitute nonplastic specimens that will be subjected to constant-volume DSS testing. Then we examine the effect of suction pressure in those specimens on shearing resistance at the critical state.

Theoretical Background

Suction pressure (or soil suction), ψ, causes the shear strength of unsaturated specimens to differ from otherwise identical saturated specimens. Suction pressure in unsaturated soils consists of two parts (Fredlund, Rahardjo, and Fredlund 2012) and is expressed as follows:

$$\psi = (u_a - u_w) + \pi \tag{1}$$

where:

u_a = pore-air pressure,

u_w = porewater pressure,

$(u_a - u_w)$ = matric suction, and

π = osmotic suction.

If the salt concentration in the specimen is small (i.e., the particles are inert, and distilled water is used), then π is insignificant, and $\psi \approx u_a - u_w$.

The shear strength of an unsaturated, nonplastic (i.e., cohesionless) soil can be represented as follows (Fredlund, Rahardjo, and Fredlund 2012):

$$s = (\sigma_n - u_a)_f \tan\phi' + (u_a - u_w)_f \tan\phi^b \tag{2}$$

where:

s = shear strength,

σ_n = total normal stress,

$(\sigma_n - u_a)_f$ = net normal stress at failure,

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

ϕ' = internal friction angle, and

ϕ^b = angle indicating the rate of increase in shear strength with respect to the change in matric suction.

Accordingly, figure 1 illustrates the failure envelopes of soils with different matric suctions. As illustrated in the figure, at the same net normal stress, as matric suction in the soil increases, shear strength increases. Once matric suction equals zero, i.e., $(u_a - u_w) = 0$, the net normal stress $(\sigma_n - u_a)$ becomes $(\sigma_n - u_w)$, which is the effective normal stress used for saturated soils.

Alternatively, we can apply an effective stress framework for unsaturated soils (Bishop 1959; Lu and Likos 2004; Fredlund, Rahardjo, and Fredlund 2012). Here, the soil shear strength is defined as follows:

$$s = (\sigma_n - u_a)_f \tan\phi' + \chi_f (u_a - u_w)_f \tan\phi' = (\sigma_n - u_a)_f \tan\phi' + c''_f \tag{3}$$

where:

χ_f = effective stress parameter, which varies between 0 and 1, that reflects the contribution of matric suction to effective stress and usually is inversely related to matric suction, and

c''_f = capillary cohesion at failure that describes shearing resistance arising from capillarity.

Accordingly, suction stress becomes a material variable that results from partial saturation of the soil and is independent of initial external loading (Lu and Likos 2004). In constant-volume DSS testing, excess porewater pressure will be generated, and the pore-air pressure will be drained, which equates to a constant water content test as described by Fredlund, Rahardjo, and Fredlund (2012). To account for generated excess porewater pressure, equation (3) can be modified as follows:

$$s = [(\sigma_n - u_a)_o - \Delta u_{w,f}] \tan\phi' + \chi_f (u_a - u_w)_f \tan\phi' \tag{4}$$

where:

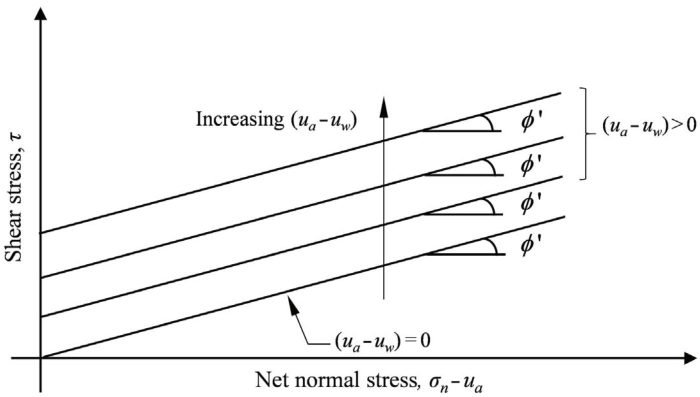
$(\sigma_n - u_a)_o$ = initial net normal stress, and

$\Delta u_{w,f}$ = excess porewater pressure at failure.

The generated excess porewater pressure might contribute to a decrease of matric suction as well. However, as shearing progresses to the critical state (i.e., generally when shear strain exceeds 20 %), significant particle

FIG. 1

Failure envelope on $\tau - (\sigma_n - u_a)$ plane for saturated and unsaturated nonplastic soils (adapted from Fredlund, Rahardjo, and Fredlund 2012).



movement including interparticle sliding and particle pushing, rearranging, and rolling occurs. These particle movements may "break" the initial particle contacts where water menisci exist, thereby decreasing capillary cohesion. Hence, it might be reasonable to infer that, at large strain and if the initial capillary cohesion is small, the generated excess porewater pressure contributes mainly to decreasing effective stress instead of decreasing matric suction. In this scenario, equation (4) holds true.

If the unsaturated, moist specimen is connected to atmosphere, then $u_a = 0$ throughout testing, and equation (4) can be further simplified as follows:

$$s = (\sigma_{n,o} - \Delta u_{w,f}) \tan \phi' + c''_f \tag{5}$$

It is further inferred that if the critical-state failure envelope of moist specimens plotted in $s - (\sigma_{n,o} - \Delta u_w)$ space, where Δu_w is inferred from the decrease in vertical stress, exhibits a zero intercept ($c''_f = 0$), then the effect of initial suction on critical-state shear strength is eliminated. In this case, the critical state of the unsaturated specimen should match the critical state of a dry or saturated specimen in constant-volume DSS testing. Khalili, Geiser, and Blight (2004) interpreted several sets of experimental data on unsaturated soils and concluded that the critical-state concept holds true for both saturated and unsaturated soils if effective stress is properly determined. Constant-volume DSS test results are presented here to justify this interpretation.

Initial soil suction in moist-tamped specimens is related to the preparation water content. The relationship between water content (either gravimetric or volumetric) and soil suction is termed the soil–water characteristic curve (SWCC) (Lu and Likos 2004; Fredlund, Rahardjo, and Fredlund 2012). The SWCC can be estimated from grain-size distribution or measured directly (Arya and Paris 1981; Fredlund, Wilson, and Fredlund 2002).

Materials and Methods

TESTING SOILS AND SOIL PROPERTIES

Experiments in this study involved three soil gradations: Ottawa F-65 sand and two nonplastic sand–silt mixtures with fines contents (*FC*) of 15 % and 60 %, respectively, created by mixing Ottawa F-65 sand with Min-U-Sil 40 fine-ground silica. Both Ottawa F-65 and Min-U-Sil 40 are available commercially from US Silica. Ottawa F-65 is a white, inert, uniformly graded, clean silica sand with subrounded to rounded particles. Min-U-Sil 40 is a bright white, inert, uniformly graded, nonplastic fine-ground silica material with subangular to angular particles. **Figure 2*A*** presents representative grain-size distributions for all three soil gradations.

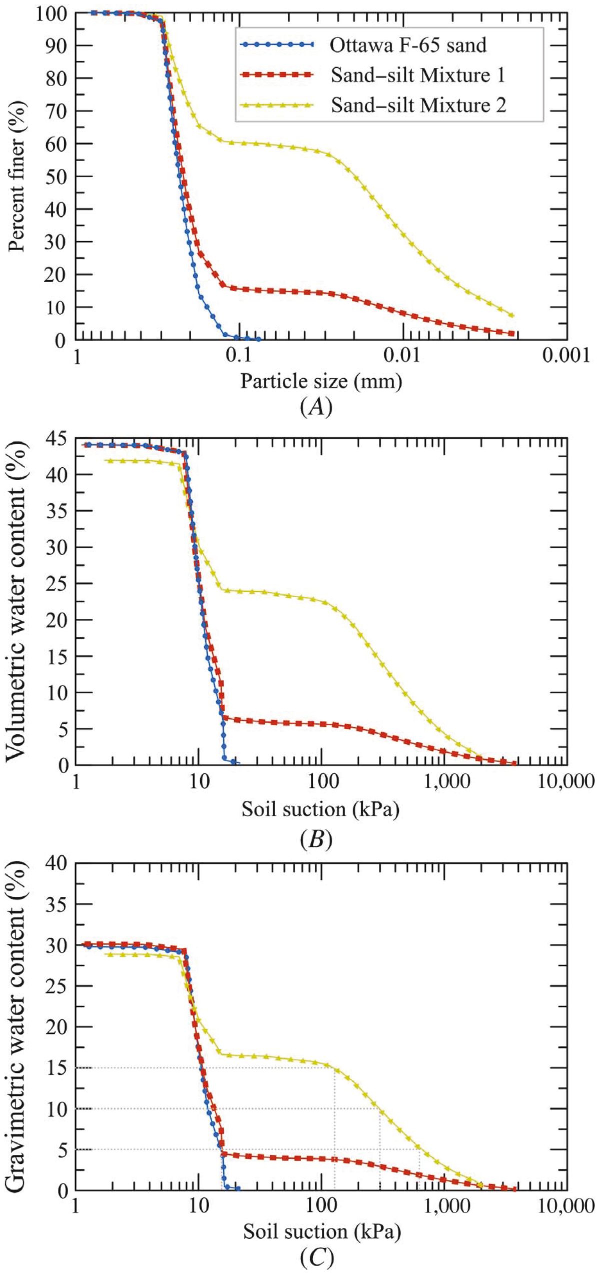
In this study, SWCCs for all tested soils were estimated using the grain-size distribution approximation proposed by Arya and Paris (1981). **Table 1** summarizes the parameters used to predict the SWCC for all three gradations. The resulting SWCCs are presented in **figure 2*B*** using volumetric water content. In **figure 2*C***, we converted volumetric water content to gravimetric water content using specific gravities and void ratios of the testing soils after consolidation.

TESTING METHOD

Constant-volume monotonic tests were carried out using the University of Illinois multidirectional cyclic DSS device (Bhaumik et al. 2017). During testing, specimens were confined by a stack of thin low-friction metal rings, with an inner diameter of 102.76 mm, to maintain a constant horizontal cross section. A thin latex membrane, with a thickness of 0.508 mm, separates the specimen from the confining rings. The as-prepared specimen height was about 19 mm, which yields a specimen diameter-to-height ratio of about 5.4. Specimens were prepared by moist tamping. For Ottawa F-65 sand and Sand-silt Mixture 1 ($FC = 15$ %), an initial gravimetric water content of 5 % was used during specimen preparation. For Sand-silt Mixture 2 ($FC = 60$ %), a preparation gravimetric water content was 15 %. The reason for selecting these water contents will be explained in the following section. The specimens were not saturated prior to consolidation. In this study, specimens were tested in a standard atmospheric environment, and air pressure was not controlled, suggesting that specimens were connected to the atmosphere and internal specimen air pressure should be zero. The top loading cap was fixed during testing, and generated excess porewater

FIG. 2

(*A*) Representative grain size distributions; and soil-water characteristic curves for soils in terms of (*B*) volumetric water content, and (*C*) gravimetric water content.



pressure was inferred from changes in measured vertical stress. Tests in this study resembled constant water content tests (Fredlund, Rahardjo, and Fredlund 2012).

Results and Discussion

As illustrated by the SWCC presented in **figure 2C**, the initial soil suction within the Ottawa F-65 sand and Sand-silt Mixture 1 ($FC = 15$ %) specimens is quite small (~15 kPa) with $w = 5\%$. For these gradations, additional water may not greatly reduce soil suction until the specimen approaches full saturation. Therefore, $w = 5\%$ appears to be a

TABLE 1

Input parameters for determining SWCC using Arya and Paris (1981) model

Soil	e	G_s	α
Ottawa F-65 sand	0.791	2.65	1.285
Sand–silt Mixture 1	0.814	2.65	1.285
Sand–silt Mixture 2	0.826	2.65	1.15

Note: e = void ratio after consolidation, average value from completed tests; G_s = specific gravity; α = model variable proposed by Arya, Richter, and Davidson (1982).

reasonable value for preparing specimens of the nonplastic Ottawa F-65 sand and Sand-silt Mixture 1 and to ensure an initial state that is relatively close to the $(u_a - u_w) = 0$ condition.

In contrast, for Sand-silt Mixture 2 ($FC = 60$ %), the soil suction is significant (~600 kPa) at $w = 5$ % due to the greater percentage of silt particles. As expected, the smaller the average particle diameter, the smaller the pore diameter, and therefore, the greater the soil suction (Arya and Paris 1981). At $w = 10$ %, the soil suction decreases to approximately 300 kPa (~1/2 of the value at $w = 5$ %) but is still significant. When w is increased to 15 %, soil suction decreases to ~110 kPa. Practically, $w = 15$ % was the largest gravimetric water content that could be used for specimen preparation, as further increases in w (≥17 %) caused the soil to become a slurry. Moreover, **figure 2C** indicates that $w = 15$ % approaches the part of the SWCC where soil suction decreases significantly. The authors note that the Arya and Paris (1981) model predicts the drying SWCC. However, specimen preparation (mixing dry soil and water) more closely resembles a wetting process. The SWCC typically exhibits hysteresis between the drying and wetting curve, and the wetting SWCC usually exhibits lower suction pressures than the drying curve at a given water content (Fredlund, Rahardjo, and Fredlund 2012). Yang et al. (2004) illustrated that the hysteresis between the drying and wetting SWCC may be in the range of 0.2 log cycles of suction for fine sand to 1.1 log cycles of suction for clayey sand. The grain size distribution of the Sand-silt Mixture 2 is more like the clayey sand reported by Yang et al. (2004). Thus, the soil suction in the moist-tamped specimen of Sand-silt Mixture 2 ($FC = 60$ %) with $w = 15$ % may be considerably smaller than inferred from **figure 2C**, i.e., well below 100 kPa. For sand–silt mixtures, it is the significant drop in soil suction that occurs in Sand-silt Mixture 1 around $w = 5$ % and in Sand-silt Mixture 2 between $w = 15$ and 20 % that appears to be key for specimen preparation.

As reported in **Table 2**, a total of 16 constant-volume DSS tests have been conducted on three moist-tamped, nonplastic specimens with $w = 5$ % (Ottawa F-65 sand and Sand-silt Mixture 1 [$FC = 15$ %]) and $w = 15$ % (Sand-silt Mixture 2 [$FC = 60$ %]). **Table 2** includes details of initial conditions of the specimens (i.e., void ratio after consolidation and initial effective vertical stress) and stresses at the critical state (both shear stress and effective vertical stress). In addition, **Table 2** summarizes seven drained DSS tests in which no excess porewater pressure was generated during shearing. Representative constant-volume DSS test results are presented in **figure 3**. As can be seen from the normalized stress–strain curve in **figure 3A**, all testing specimens exhibit strain-softening response after reaching a peak (yield) shearing resistance, indicating typical "static liquefaction" response. Similar to the examples presented in **figure 3**, all constant-volume DSS specimens reached a critical state of constant shearing resistance and constant excess porewater pressure ratio at large strains (beyond ~10–15 %).

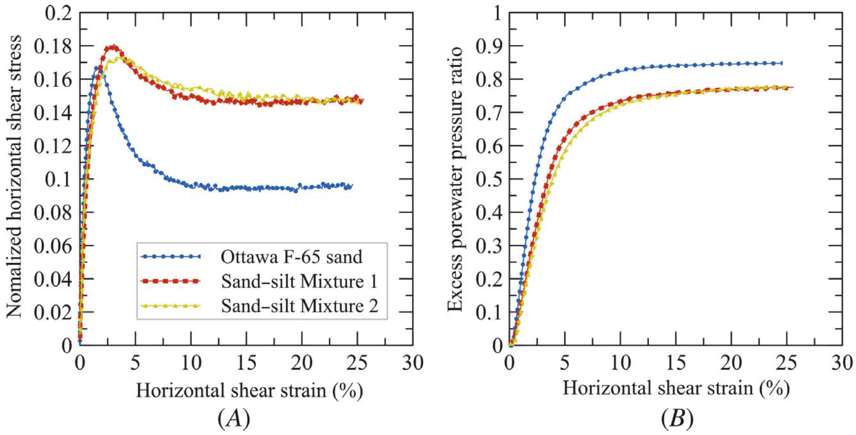
Figure 4 presents critical-state conditions (in $\tau - \sigma'_n$ space) for the DSS tests reported in **Table 2**. As discussed previously, if the failure envelope for the unsaturated specimens tested under constant-volume conditions exhibits zero cohesion, then the effect of suction pressure is minimal, and the failure envelope is likely to match the behavior of dry and saturated specimens. In addition, it also appears reasonable to extend this concept to drained tests in which no excess porewater pressure was generated during testing. As illustrated in **figure 4**, the failure envelopes for unsaturated specimens of all three gradations (under both constant-volume and drained conditions) exhibited practically zero cohesion. The resulting critical-state friction angle was $\phi'_{cs} = 28°$ with $c''_f = 0$ for the Ottawa F-65 sand gradation. For comparison, ElGhoraiby (2019) reported $\phi'_{cs} = 30.7°$ for isotropically consolidated undrained triaxial compression tests on saturated Ottawa F-65 sand specimens prepared by moist tamping. Considering the different modes of shear, these values of ϕ'_{cs} are quite

TABLE 2
Summary of DSS test results

No.	Soil	FC, %	w, %	Drainage	e_c	σ'_{no}, kPa	$\sigma'_{n,cs}$, kPa	s_{cs}, kPa
1	Ottawa F-65 sand	0	5	CV	0.806	191.0	25.8	17.2
2				CV	0.802	191.3	29.1	18.5
3				CV	0.789	95.8	33.8	22.1
4				CV	0.802	456.9	37.2	25.9
5				CV	0.785	346.8	53.2	34.4
6				D	0.811	101.3	101.3	56.2
7				D	0.759	301.2	301.2	157.9
8				D	0.771	204.8	204.8	108.7
9	Sand-silt Mixture 1 and Sand-silt Mixture 2	15	5	CV	0.797	205.5	51.0	31.0
10				CV	0.733	391.5	94.9	52.4
11				CV	0.848	120.4	30.4	20.7
12				CV	0.844	205.8	45.9	29.4
13				CV	0.877	127.2	28.4	18.8
14				CV	0.812	202.1	49.7	28.8
15				D	0.690	205.0	205.0	113.6
16				D	0.913	24.1	24.1	17.7
17	Sand-silt Mixture 1 and Sand-silt Mixture 2	60	15	CV	0.954	95.2	24.1	14.3
18				CV	0.945	94.4	25.7	15.5
19				CV	0.803	194.1	43.2	28.4
20				CV	0.827	196.1	37.5	28.1
21				CV	0.721	388.6	66.9	47.6
22				D	0.809	50.5	50.5	37.7
23				D	0.725	74.6	74.6	53.5

Note: FC = fines content; w = water content used during sample preparation; CV = constant-volume; D = drained; e_c = void ratio after consolidation; σ'_{no} = initial effective vertical (normal) stress; $\sigma'_{n,cs}$ = effective vertical (normal) stress at critical state; s_{cs} = critical-state shear strength.

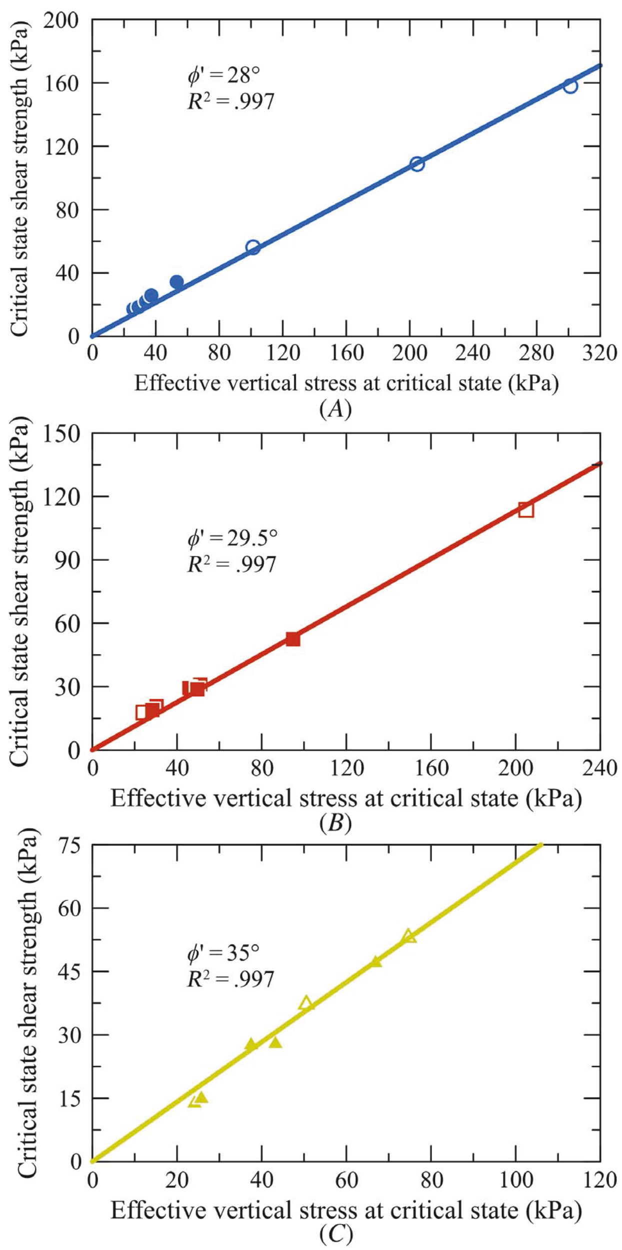
FIG. 3 Typical constant-volume DSS test results for Ottawa F-65 sand (Test #1), Sand-silt Mixture 1 (Test #12) and Sand-silt Mixture 2 (Test #20): (*A*) normalized horizontal shear stress versus horizontal shear strain; (*B*) excess porewater pressure ratio versus horizontal shear strain.



consistent. For those sand–silt mixtures that were manually created by mixing the subrounded to rounded clean sand with subangular to angular fine-ground silica, the mobilized ϕ'_{cs} increased as the percentage of angular silt-sized particles increased.

FIG. 4

Shear strength versus effective vertical stress at critical state for tested soils: (*A*) Ottawa F-65 sand; (*B*) Sand-silt Mixture 1 ($FC = 15$ %); (*C*) Sand-silt Mixture 2 ($FC = 60$ %). Solid symbol = constant-volume condition; open symbol = drained condition.



Conclusion

In this study, the authors propose a specimen preparation protocol that can be used to reconstitute contractive, moist (unsaturated) specimens for constant-volume DSS testing. The theoretical background and the tests results in this study indicate the following conclusions:

(1) Water content at preparation used for reconstituting moist-tamping specimens of nonplastic soils should be selected such that the initial suction pressure in the specimen is small according to its SWCC. By satisfying this requirement, the effect of initial suction pressure on the critical-state shearing resistance

should be largely eliminated, and the effective vertical stress at the critical state, inferred using the change in the vertical stress, should be reasonable.

(2) Nonplastic soil specimens prepared in this manner exhibited a critical-state failure envelope with no cohesion intercept, and accordingly, this interpreted critical state is likely to be identical to the critical state obtained from constant-volume DSS tests on dry or fully saturated specimens.

(3) Using this protocol, the efficiency and effectiveness of constant-volume DSS tests for studying critical-state behavior of nonplastic soils even with large *FC* may be greatly improved.

The authors note that further experimental validation, i.e., comparing the critical-state conditions (in $\tau - \sigma'_n$ space) for saturated specimens and specimens prepared using the proposed protocol tested under identical initial conditions, would strengthen the arguments made in this Technical Note.

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