Near-Surface Soils: Self-Supported Unconfined Drained Sand Specimens

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Abstract

Soil behaviour in the near surface is fundamental to a broad range of military strategic research efforts. The prevailing mathematical representations of near-surface shear strength do not necessarily reflect the observed shear strengths. This paper presents an experimental procedure for testing self-supported, unconfined-drained specimens to investigate the shear strength of a poorly-graded sand with the inferred unsaturated behaviour from typical effective stress expressions. The impact of soil suction is quantified from the soil-water characteristic curve obtained from a modified, unsaturated, one-dimensional oedometer device: Fredlund's device. Seventy-seven unconfined tests illustrate that while individual or discrete states can be explained by effective stress relationships, those expressions do not accurately quantify the observed shear strength from one state to the next. Saturation failure is observed when the degree of saturation is between 70.5% and 76.5%, well below the assumed threshold. The results show that the dry shear resistance is a function of the initial density state, but as the degree of saturation increases, the shear resistance becomes independent of the initial density, and converges to the saturation failure. These findings expose the limitations of effective stress expressions and suggest a revised non-linear three-dimensional unconfined Mohr failure envelope based on an applied external stress.

Keywords

Fabric/structure of soils; shear strength; laboratory tests; low confinement; unsaturated.

1. Introduction

The soil behaviour of the near surface, the upper 1 m in depth at K_o -confinement of less than 20 kPa, is fundamental to a broad range of military mission spaces and strategic research efforts (Taylor et al. 2014; Taylor et al. 2016; Winters et al. 2016; Taylor et al. 2017 a,b; Taylor and Winters 2017). Laboratory research on non-plastic soil behaviour in low confining pressure environments relies on the use of effective stress principles to infer behaviour (Rowe 1962; Bolton 1986; Schanz and Vermeer 1996; Sture et al. 1998; Lee 2000; Fannin et al. 2005; Lancelot et al. 2006; Graham et al. 2014; Huang et al. 2015). However, such tests are not indicative of low-confining pressure in situ environments due to the application of confining pressures, e.g., membranes, testing cell fluid head (e.g., water within a triaxial cell chamber), and/or applied vacuum pressure used to maintain sample stability of loose non-plastic material. This is also true for laboratory testing of unsaturated soils, especially in the development of the soil-water characteristic curve, SWCC, wherein specimens are confined and subjected to pore-air and pore-water pressures, u_a and u_w respectively, to quantify the air-water phase or matric suction, ψ , that defines the unsaturated soil strength and behaviour (Fredlund and Rahardjo 1993; Lu and Likos 2004).

The quantification of ψ is obtained from the SWCC and is typically measured using an unsaturated one-dimensional oedometer, or by the transient outflow response of a sample when air and pore pressure increments are applied, using axis translation to generate a specific value of ψ . Once a discrete set of measurements are taken, a mathematical model is fitted to the data to form a continuous SWCC (van Genuchten 1980; Fredlund and Xing 1994). Research into the resulting shear strength, τ , in terms of Coulomb mechanics has suggested that the impact of suction to the overall shear strength can be treated as an additive component to conventional saturated relationships, e.g., Equation 1 (Vanapalli et al. 1996; Han and Vanapalli 2016).

$$\tau = c' + (\sigma_n - u_a) \tan(\phi') + \psi S_r^\xi \Gamma \quad (1)$$

In the above equation, σ_n is the normal stress along the failure plane, u_a is the pore air pressure, c' is the soil cohesion, ϕ' is the effective internal friction angle and the $\psi S_r^\xi \Gamma$ parameter is the contribution of non-linear suction strength from the SWCC in unsaturated soil mechanics (Vanapalli et al. 1996; Han and Vanapalli 2016). Han and Vanapalli (2016) suggest that for shear strength calculations, the Γ term is assumed equal to $\tan(\phi')$, the ξ parameter is 1.0 and S_r is the saturation for sand. Lu et al. (2010) presented a similar equation, based on an effective saturation, S_e , the Mohr-Coulomb failure criterion and the van Genuchten (1980) SWCC fitting parameters, n and α :

$$\tau = \left[(\sigma - u_a) + \frac{S_e}{\alpha} \left(S_e^{\frac{n}{1-n}} - 1 \right)^{\frac{1}{n}} \right] \tan \phi' + c' \quad (2)$$

For most geotechnical problems, these equations yield relatively accurate approximations of shear strength behaviour (Lu et al. 2010; Han and Vanapalli 2016). However, the shear strength suggested by Equations 1 and 2 do not reflect the observed shear strengths within near-surface and low confinement environments (Taylor et al. 2014; Taylor et al. 2016; Winters et al. 2016; Taylor et al. 2017b). Therefore, this paper presents an experimental procedure for construction and testing of self-supported unconfined-drained sand specimens and resulting observations into the behaviour of near-surface soils subjected to ultra low confining stresses, their failure surfaces and shear strength.

2. Experimental Protocols

2.1 Sample Preparation

Unconfined-drained (UD) uniaxial compression testing was performed on a medium-fine poorly-graded quartz-silica sand (SP via the USCS, ASTM D2487-11), Figure 1. The mean grain size, coefficient of uniformity and curvature are 0.35 mm, 1.52 and 1.12, respectively. The particles are rounded to sub-rounded and the specific gravity is 2.70 and internal friction angle, determined from traditional isotropically consolidated–drained strain-controlled triaxial testing, ICD_{tx} , ranges from

33.1° to 38.0° over a range of different densities at confining pressures between 10-100 kPa (Winters et al. 2016). Test specimens were prepared via an energy controlled moist tamping method (Taylor et al. 2017a) using a 71 mm diameter mould to a height of 145 mm at a remoulding saturation of 24%¹. Since specimen repeatability is critical in *UD* testing, specimens deviating more than 1.5% of the target reconstitution, in water content, sample volume and/or density, were excluded (this occurred in less than 10% of cases). The density index, Equation 3, is used to define the specimen's density state relative to the maximum and minimum dry densities from ASTM Standard Methods D4253 and D4254, respectively (Holtz et al. 2011). However, as the sample preparation follows the energy-based preparation method from Taylor et al. (2017) the minimum and maximum densities must also be derived from the same compactive manner to define a maximum and minimum stable reconstituted soil structure. Herein, the term *index density*, I_d , is used to define a soil's density state in relation to maximum and minimum values as described in Holtz et al. (2011), Equation 3, with the notable exception that per Taylor et al. (2017) the minimum and maximum values of density should be determined in the same compactive manner as the sample preparation. The index densities to which the soil structure remained stable ranged from 0.0 (the loosest condition by which no volumetric strain is observed during typical backpressure saturation in a standard triaxial test: $\rho_{d, min} = 1.618$ g/cc, which correlates to a compactive effort of 100 kJ/m³) to 1.0 (the very dense condition correlating to the Modified Proctor density: $\rho_{d, max} = 1.717$ g/cc, with a compactive effort of 2707 kJ/m³).

$$I_d = \frac{\rho_d - \rho_{d, min}}{\rho_{d, max} - \rho_{d, min}} \quad (3)$$

where ρ_d is the dry density of the reconstituted sample. Each test was recorded via high-speed high-resolution videography (two cameras at a 1280x800 dpi resolution and 389 fps) at the orientation shown in Figure 2. Still-frame images, at 1280x800 dpi resolution, were created and enhanced to 150% of the sample dimensions for determining specimen strains via pixel matching;

¹ See Appendix A sections A1 and A2

computer-aided and manually. All water used was filtered then distilled to remove the potential of chemical contaminants that could introduce an artificial bonding of soil particles and de-aired to allow for more efficient saturation. Moisture content and saturation calculations were performed in accordance with ASTM Standard Test Method D2216 to a gravimetric water content accuracy of $\pm 0.01\%$ based on the scale resolution of ± 0.01 g.

To ensure drained conditions, i.e., preventing the generation of excess pore pressures, throughout testing, all sample extents are open to atmospheric pressure and a 20-second time lag between axial loading increments was used. All specimens are unconfined and uncontained, i.e., no chamber, cell, membrane or apparatus is used to support the sample post reconstitution, at atmospheric conditions such that effective stress and total stress regimes are equivalent. The time lag is required to ensure no significant strain rate effects develop (Yamamuro and Abrantes 2005), internal excess pore pressure is eliminated, and that any dilatancy effects are observed before additional load is added.

The duration of the time lag was chosen based on the assumption that sands undergo immediate settlement but can exhibit secondary compression, e.g. particle re-orientation. A 20-second time lag, corresponding to a mean loading rate of less than 1 kPa/min, was chosen to optimize the videography data and image resolution while allowing for potential strains, immediate and secondary, to be observed. Further, this loading rate is comparable to typical drained triaxial testing to ensure that drained conditions are met and pore pressure generation does not occur (ASTM Standard Test Method D7181).

Unconfined specimens were loaded using increments of 1000 g (2.40 kPa), 500 g (1.20 kPa), 200 g (0.36 kPa), 100 g (0.24 kPa) and 50 g (0.12 kPa) calibrated weights, placed on top of the sand specimen until initial failure was observed. Table 1 provides the incremental loading stages such that the final loading stage was at a minimal increment before failure.

2.2 Soil-Water Characteristic Curve Testing

To quantify the matric suction of the sand, a SWCC was developed experimentally using a modified unsaturated one-dimensional oedometer (K_0) device, i.e. the Fredlund device, manufactured by GCTS, using axis-translation to control suction (Padilla et al 2005; Pham et al. 2004; Perez-Garcia et al. 2008; Fredlund and Houston 2013). The Fredlund device was employed using a three-bar high air entry ceramic disk. The SWCC was tested over a range of matric suctions from 2-180 kPa.

The data were derived from experimental data for $I_d = 0.54$ and numerically fit with the Fredlund and Xing (1994) model, Equation 4, (with empirical fit parameters $a = 5.7$; $m = 1.2$; $n = 15.0$) to develop the full spectrum SWCC shown in Figure 3.

$$S_r = \left\{ \frac{1}{\ln \left[2.718 + \left(\frac{\psi}{a} \right)^n \right] \right\}^m \quad (4)$$

where S_r is the degree of saturation. Walshire et al. (2017) conducted an extensive laboratory investigation on the SWCC variance of this soil in respects to both initial density state and various testing devices. This work showed for this material that there was no statistical difference in the SWCC between different initial density states. Therefore, the SWCC for this material is expected to be the same over the range of densities tested for the fit shown Figure 3.

2.3 Unconfined Drained Dry (UD_{κ}) Testing

The purpose of UD_{κ} -testing is to investigate a soil's fabric strength, κ , defined as the applied stress required to initiate failure of a dry soil specimen. Changes to the inter-particle stresses in the near surface, or uppermost meter of the earth, have a first-order effect on the quantification of a soil's structure or fabric used to calculate strength characteristics and elastic moduli that govern modeled behavior (Taylor et al. 2014; Taylor et al. 2016). Mitchell and Soga (2006) define soil fabric as, "Particle sizes, shapes and distributions along with the arrangement of grains and grain contacts..."

and the soil's structure as, "...the arrangement of particles, density, and anisotropy." Therefore, used herein, the term *soil fabric* is defined as the inclusion of both the soil fabric and structure as defined by Mitchell and Soga (2006), the influence of particle shape (Cho et al. 2006) as well as the sum of the granular contact forces that govern a specimen's strength and volumetric behaviour.

To eliminate the influence of matric suction, samples must be void of moisture such that only granular friction provides resistance to applied and gravitational stresses. To determine the length of time required to produce no measurable gravimetric water content, samples were constructed both oven-dried (105°C for 24 hours) and air-dried within a controlled environment. Samples air-dried for a minimum of 48 hours yielded equivalent gravimetric water contents compare with oven-dried specimens to a gravimetric water content accuracy of $\pm 0.01\%$ in accordance with ASTM Standard Test Method D2216. Therefore, to minimize potential sample disturbance by moving specimens from an oven to a vibration-controlled testing platform, UD_K specimens were allowed to air-dry in a controlled environment for 48 hours, and then carefully loaded with precision weights until failure². Table 1 provides the loading increments that were used to determine the maximum applied axial stress held before failure was initiated. The amount of weight held before failure and the amount of weight at failure was recorded. To ensure that the specimen were dry, moisture content samples were taken immediately post failure. Failed specimens with moisture contents greater than 0.02% were discarded from the presented data as they could be considered to contain residual moisture, Table 2. To ensure that cementation, as defined by Santamarina (2001) and observed by Cho and Santamarina (2001), was not the cause of the UD_K observations, five tests were repeated using isopropyl alcohol in place of the water to eliminate potential cementation and residual suction that may be present for the air-drying specimens²,. In these cases, the specimens behaved in the same characteristic manner³ as those prepared with water therefore, cementation

² See Appendix A section A1.3.1 and Videos A3.6v (isopropyl alcohol) compared to Videos A3.1v to A3.5v (water).

³ See Appendix A section A3.1, Figure A3.6 and Videos Videos A3.1v to A3.6v.

and any residual matrix suction are not believed to contribute to the quantification of κ . This suggests that the strength of the dry specimen is a function of the structural skeletal forces from the soil reconstitution, as described by Santamarina (2001).

2.4 Unconfined Drained Unsaturated Shear Testing (UD_P)

The purpose of UD_P testing is to investigate a soil's ability to resist shear under unsaturated conditions, e.g., Equations 1 and 2. As the SWCC is a mathematical fit to laboratory data, therefore carrying explicit uncertainty, the minimal confining pressure resulting from capillary rise was also calculated as (Terzaghi and Peck 1948; Terzaghi et al. 1996):

$$\sigma_c = \frac{C}{eD_{10}} \rho_w g \quad (5)$$

where e is the void ratio, ρ_w is the density of the pore fluid, g is gravity, D_{10} is the 10% particle diameter fraction of the grain size distribution (0.25 mm for this soil), and C is an empirical coefficient that is a function of granular surface impurities and shape ($0.01 < C < 0.05$). Based on the mean grain size and average void ratio, C is approximately 0.015 in this study (Peck et al. 1974; Atterberg 1908). The results of Equation 5 were compared to the contribution of suction towards the skeleton constitutive strength, $\psi S_r^{\xi} \tan(\phi')$ in Equation 1 and $\frac{S_e}{\alpha} \left(S_e^{\frac{n}{1-n}} - 1 \right)^{\frac{1}{n}} \tan(\phi')$ in Equation 2; see columns (b), (c), and (d) respectively in Table 3. The contributions of suction varied between Equations 1, 2 and 5, but were in relative agreement with the laboratory SWCC data, Figure 3, at 24% saturation. Therefore, an average value was used, column (e) in Table 3, to define the confining stress, σ_3 , attributed to suction forces.

The UD_P test was conducted immediately after specimen reconstitution and carefully loaded with precision weights until failure per the loading plan in Table 1. The amount of weight held before failure and the amount of weight at failure was recorded. After failure, three moisture content

samples were taken from the centre, middle and top of the failed sample to ensure moisture uniformity throughout the sample⁴, Table 2.

2.5 Unconfined Drained Suction Failure Testing (UD_S)

The purpose of UD_S testing is to investigate the behaviour and suction strength component, $\psi S_r^{\frac{\epsilon}{\zeta}} \Gamma$ in Equation 1 and $\frac{S_e}{\alpha} \left(S_e^{\frac{n}{1-n}} - 1 \right)^{\frac{1}{n}} \tan(\phi')$ in Equation 2, when the specimen was subjected to only a gravitational influx of water and no external loading. The UD_S test was conducted immediately after specimen reconstitution. Using a calibrated dropper or pipet, the top of the specimen was uniformly saturated with distilled de-aired water in 3 mL increments. Due to sands having relative high permeabilities, a lag time of 20 seconds between each fluid increment was adopted to allow for permeation of the water through the entire sample. At the onset of failure, defined as deformation observed on high-resolution videography, saturation efforts were suspended. If the observed deformation ceased within 20 seconds, the saturation process was continued in the same incremental manner until additional deformation was observed in the high-resolution videography. Post failure, three moisture content samples from the failed specimen were compared to the calculated saturation based on volume of added water to ensure that there is no significant water loss from the specimen⁵, Table 2.

3. Experimental Observations

3.1 UD_{κ} Observations

Thirty-three UD_{κ} tests were completed wherein failure was observed to be a bulk failure, Figure 4.

The initial hypothesis behind quantifying the fabric strength of the soil, κ , in which a shear plane would develop quickly due to the lack of any confining pressure supporting dry sands, suggested

⁴ See Appendix A section A1.3.2

⁵ See Appendix A section A1.3.3

that an internal friction angle, ϕ' , would be observed and the contribution of $(\sigma_n - u_a) \tan(\phi')$ would be the shear resistance intercept, for use in defining the failure criterion within the near-surface. This hypothesis was observed to be incorrect, as all UD_K samples failed in a similar manner, irrespective of density, in which the upper third (~50 mm) of the sample sheared outwards with the edges maintaining a block shape⁶. In all cases, it was observed that failure occurred within approximately 50 mm of the added weights as the failure progressed downwards, i.e., a progressive collapse failure. The sections of the specimen that were further than 50 mm from the weights did not deform (either in shear or bulk) and no strains were discernable from the high-speed, high-resolution videography and still-frame imagery.

The weights fell in near-perfect alignment to the centre axis suggesting that no single or dominant internal shearing plane developed in this failure mode and that a bulk failure, i.e., a radially expansive progressive failure, was achieved. While Figure 4 shows the failure of the specimen, there was no discernable straining that could be observed through computer aided pixel matching or manual observations of still-frame images⁷ (images are enlarged to 150% the actual specimen size) prior to the application of the failure load, at a strain resolution of 0.0067% of the specimen dimensions based on the a ± 0.01 mm measurement accuracy. As no soil specimen is perfectly rigid it is assumed that particle straining, as noted by Santamarina (2001), occurs at a lower magnitude than the resolution achieved through pixel matching of enlarged still-frame imagery. It is assumed that these small strains must be the catalyst for the observed macroscopic failure.

At a granular level, the top of any specimen is not perfectly planar, i.e., the top of all the sand grains are not in perfect alignment with the same plane from which the sample height is determined, Figure 5a. Therefore, some grains will extend above a mean plane in relation to the neighbouring grain. As the load is applied, i.e., a comparatively rigid solid plane along the bottom of the weights,

⁶ See Appendix A section A3.1 and Videos A3.1v to A3.5v.

⁷ See Appendix A section A3.1 and Videos A3.1v to A3.5v.

the grains above the theoretical boundary plane are stressed axially, in respect to the neighbouring grains, until a stiffer critical mass of grains supports the applied load, Figure 5b. Additionally, a horizontal resultant force vector is generated due to the anisotropic loading. While it is not possible to see the potential for particle movement of the grains in the interior of the sample directly underneath the applied weights, the degree of micro-straining can be hypothesised. Assuming that 10% of the d_{50} sand grain is above the theoretical boundary plane for which the sample height is determined, and a mean granular axial strain of 2.5-3.5% occurs before the applied load is supported by additional grains, then the resulting observable macro-strain per loading increment is in the order of 0.00005% of the sample height. These micro-strains would yield a horizontal stress resultant as a function of the applied load similar to rotational frustration described by Santamarina (2001). This process is then repeated for each grain layer until a critical summation of all the horizontal stress resultants from each loading step exceeds the skeletal resistance of the granular contacts, i.e., the fabric strength, κ . Once this critical stress depth is achieved a progressive failure would be initiated with a horizontal ejection of the boundary particles from the internal horizontal resultant, Figure 5c. This is what is observed within the high-speed videography at the initiation of failure. As the applied weights fall axially, the thickness of the critical depth is maintained (~50mm) in which sections of the specimen outside this region do not exhibit radial granular ejecta.

Given the radial manner of failure over a critical depth, and the lack of measurable water and pre-failure strain, the quantification of κ should not be calculated as an effective stress process, i.e., $(\sigma_n - u_a) \tan(\phi')$. This is due to the means by which ϕ' is classically determined and no shear failure plane was observed in the sample. Furthermore, if ϕ' is assumed to be an intrinsic soil property and if used in Equations 1 and 2 the resulting shear strength, or κ , is non-tangential to the Mohr's circle and in excess of the maximum shear stress. This implies that ϕ' is intrinsic to the behaviour relating to the applied stresses not the internal state of the soil. Therefore, the mineralogical friction angle, ϕ_m , an intrinsic property to the physical make-up of the specimen that encompasses the inertial

effects at the granular level, should be used to quantify κ . For this material ϕ_m , is approximately 26° (Terzaghi and Peck 1948; Terzaghi et al 1996).

Given the progressive failure mode, Figure 5, κ should be expressed as:

$$\kappa = \Omega \tan(\phi_m) \quad (6)$$

where Ω is the maximum resisted rotational frustration, i.e., the maximum resisted deviator stress to prevent particle rotation and ultimate sample failure, σ_d , as a function of I_d :

$$\Omega = f(\sigma_d, I_d) \quad (7)$$

The functional form of Ω can be found as a best-fit of the UD_K laboratory data, Figure 6.

The results of the UD_K tests are shown in Figure 6 with a slight exponential increase of strength with increasing density. This increase is not believed to be the product of dilation, rather a function of the increase in granular contacts, lower void ratios, and larger grain-to-grain contact pressures as a product of higher compactive energies applied during sample preparation required to change specimen density states. This observation is in agreement with the buckling of particle chains and rotational frustration amongst particles as described by Ladd (1967) and Santamarina (2001).

3.2 UD_P Observations

Twenty-eight UD_P tests were performed to investigate failure in an unsaturated state, Table 3 and Figure 7. In all cases⁸, straining and shear failure surfaces were observed throughout the entirety of the sample. The shear strength was calculated based on Equations 1, 2 and 5, Table 3. The

⁸ See Appendix A section A3.2 and Videos A3.7v to A3.11v.

shear strength calculated by Equation 2 consistently produced the lowest values while Equation 1 was consistently the largest. The contribution of suction to the shear strength, columns (b), (c), and (d) in Table 2, varied +/-10% of the mean range, column (e). All calculated values of shear stress were below the maximum shear stress developed during UD_P testing, τ_{max} .

The secant friction angle, ϕ'_s is often used to express failure strengths for curved strength envelopes in non-plastic material with typical values between 35° and 60° (Terzaghi et al. 1996). Therefore, ϕ'_s is calculated based on the average τ in Table 3 as:

$$\phi'_s = \tan^{-1} \left(\frac{\sigma_n - u_a}{\tau} \right) \quad (8)$$

For these samples, ϕ'_s varied less than ϕ' and slightly increased with sample density. However, the mean ϕ'_s for all tested specimens (64.59°) was within the data range and within approximately one standard deviation of the calculated mean ϕ'_s at each density suggesting that ϕ'_s is independent of specimen density and can be assumed 64.59° for this material at 24% saturation. The mean ϕ'_s is above the range typically found for non-plastic materials and is closer to rockfill and granular material at higher confinement than poorly-graded sands. Based on the void ratio for each index density, Table 3, the typical ϕ'_s , at low effective stresses, should be approximately 49°, substantially lower than the observed test data (Terzaghi et al 1996).

If we examine the components of ϕ'_s for a curved Mohr failure envelope, in terms of ϕ_m and the interlocking or interconnection of the physical grains, termed geometrical interference ϕ'_g , it is found that the contribution of ϕ'_g is approximately 38.6°, Equation 9 (Terzaghi et al. 1996).

$$\phi'_s = \phi_m + \phi'_g \quad (9)$$

The geometrical interference is in keeping with soils under low confining pressures (Terzaghi et al. 1996) and is relatively independent of initial density state.

3.3 UD_S Observations

Sixteen UD_S tests were performed to investigate the influence of matrix suction on the shear strength of near-surface soils, Figure 8. However, post-initial failure slumping was observed, Figure 9 (red digital overlay is taken from the onset of failure and the grey base image is the initial condition) wherein a 2 mm axial deformation can be observed with a 1.5 mm (maximum) radial deformation. As shown in Figure 9, there is no re-orientation or movement of the visible grains in the upper 75 mm of the sample, evidenced by the granular ridge shape at the top of the sample and other distinctive features at high-resolution. After failure was initiated the specimen experienced uncontrollable volumetric changes, i.e., sloughing, until the eccentricity of the continuum forced a shear failure at the sample base, Figure 8. However, this process is extremely slow in comparison to UD_K and UD_P testing; it can take in excess of 60 seconds for a UD_S sample to transition from initial failure to eccentrically sheared failure. This process, albeit slow, is continuous and no additional fluid is added. In no case was the pore water expelled from the sample matrix, and the upper 75 mm of the sample retained its shape throughout the failure collapse⁹.

The UD_S failure stress is calculated as the gravitational load from the soil and water applied to the base of the sample. The predicted matric suction strength from Equations 1 and 2 and using the critical state friction angle, ϕ_{cs} (33.9°) in the Γ parameter of Equation 1 is tabulated in Table 4. The results indicate that the predicted suction shear strength from Equation 1, column (b) in Table 4, is greater than the total stress at the sample base, for I_d of 0.42, 0.54, 0.63 and 0.70, suggesting that the specimen should be stable. In these cases, this strength differential is approximately 0.5 kPa, this difference is equivalent to an additional five of the 50 g loading increments; a significant increase in loading for these tests. Only in the loosest case, I_d of 0.21, is the suction resistance

⁹ See Appendix A section A3.3 and Videos A3.11v to A3.16v.

equal to the total stress at the base of the specimen. However, Equation 2, column (c) in Table 4, presents a significantly reduced suction shear resistance implying that the specimen would have undergone failure much sooner than observed experimentally.

When ϕ_{cs} is used in determination of the suction shear strength, column (a) in Table 4, the suction shear resistance is nearly identical to the base stress test data at the failure saturation, Table 4 and Figure 10. However, Figure 10 illustrates, even using ϕ_{cs} in the calculation of Γ , that a specimen should increase in suction shear strength until the saturation exceeds 94.3%, the maxima of the $\psi S_r^\xi \Gamma$ curve corresponding to the air-entry value (AEV) of the SWCC. However, all samples experienced failure at saturations between 70.5% and 76.5%. This suggests that a suction failure surface exists in advance of the AEV maximum predicted strength when samples are unconfined. It should be noted that the observed failure saturation occurs at the inflection point of the SWCC in Figure 3.

Similar to Equation 9, we can express ϕ_{cs} as the summation of ϕ_m and ϕ_p defined as the frictional component corresponding to granular material resistance at no volumetric change (Terzaghi et al 1996):

$$\phi_{cs} = \phi_m + \phi_p \quad (10)$$

The resulting value of ϕ_p is 7.9° and is slightly higher than typical values, 5° to 7°, per Terzaghi et al. (1996).

4. Implications to Near-Surface Shear Strength

The quantification of τ , especially under low confining pressures, cannot be assumed from effective stress expressions (Santamarina 2001). While individual or discrete states can be explained by effective stress relationships using intrinsic soil properties, e.g., Equations 1, 2 and 9 for UD_p

testing, the transition from one state to the next, e.g., UD_K to UD_P or UD_P to UD_s , does not conform to effective stress principles. For example, in dry specimens, UD_K -testing, τ cannot simply be estimated by $(\sigma_n - u_a) \tan(\phi^*)$, where ϕ^* is the friction angle defined by either the effective friction angle, ϕ' , the critical state friction angle, ϕ_{cs} , or the secant friction angle, ϕ'_s . Rather, the effective stress term, by $(\sigma_n - u_a)$, takes the non-linear functional form of Equation 7. This functional form is the product of the interactions of the micro-scale forces detailed by Santamarina (2001). This non-linear function is required as the Mohr's circle, for the dry soil, starts in a failed case, i.e., the applied shear of the first loading increment is in excess of the $(\sigma_n - u_a) \tan(\phi')$ failure envelope and ϕ'_s shear strength of Equation 8.

As pore fluid is added to the specimen, UD_P , τ is reduced due to the lubrication of the grain-to-grain contact friction but increased by the presence of matrix suction. The quantification of the reduction in granular friction is not explicitly accounted for in Equations 1 and 2, rather this reduction is assumed to be implicitly contained in a reduction of σ_n . The inapplicability of this assumption is demonstrated in Table 3 where if the inclusion of $(\sigma_n - u_a) \tan(\phi')$, based on the self-weight of the specimen at the base of the sample, is included in columns (a) through (c) the resulting τ , for columns (a) and (b) are in excess of failure, suggesting specimen stability, and column (c) is still suggests that failure should have occurred well in advance of the test data. However, Equations 1 and 2 are able to adequately reflect a discrete saturation, 24% saturation as presented herein, however it would appear that this is more coincidental than accurately representing the strength behaviour of these unconfined specimens especially when samples transition to the UD_K case, Figure 3 or when saturated, the UD_P case, Table 4.

As the pore fluid is removed, as in the UD_K case, the shear resistance from suction is eliminated and the remainder of Equations 1 and 2 fail to yield an accurate estimation of shear resistance. Similarly, if the saturation is increased, UD_s test case, Equation 2 significantly under-estimates the shear resistance implying the sample should have failed well before the test data shows. However, using ϕ' per Han and Vanapalli (2016), Equation 1 indicates that the shear resistance should be adequate to prevent failure. If the Γ term in Equation 1 is calculated based on ϕ_{cs} then the suction shear resistance is equal to the total stress at the base of the sample and failure is achievable, Table 4. However, if the sample self-weight is ignored, as is the case in Equations 1, 2 and 8, then the applied stress from the additional water is insufficient to exceed the suction resistance, Table 4, and the specimen should remain stable contrary to the observed data.

The ability of a soil specimen to resist external loading defines the soil's shear strength, τ (Peck et al. 1953). Therefore, the Mohr failure envelope takes on a non-linear three-dimensional shape at low confinement based on the macroscopic behaviour of an applied external stress, Figure 11. This shape is similar to the unsaturated failure planes described by Fredlund and Rahjardo (1993) except: (1) for specimen under no confinement, the shear resistance becomes less dependent on the initial density state as saturation increases, and (2) the shear resistance converges to a single point of failure under its self-weight at a much lower saturation than expressed by traditional unsaturated testing methods, e.g. Figures 3 and 9.

As pore fluid is increased, the shear resistance can be expressed in terms of ϕ'_s in Equation 8, or ϕ' in Equations 1 and 2 until a saturation failure, UD_s test, where the self-weight as a function of ϕ_{cs} best represents the zero external loading shear resistance. It should be noted that the self-weight is required to describe UD_s failure but is not considered in either Equations 1 and 2 nor UD_K and UD_P testing. The only common element in calculating the shear resistance across all cases is ϕ_m , Equations 6, 9 and 10. The significant changes to the effective stress expressions needed to

calculate the shear resistance at discrete intervals suggests that such expressions are not representative of the underlying physics or actual behaviour of unconfined or near-surface soils.

There is a wealth of research by A.W. Bishop, D.G. Fredlund, N. Lu and many others that suggest that unsaturated soil mechanics of confined specimens can be approximated by effective stress principles in most geotechnical applications. However, when those effective principles are applied to the unconfined state, they fail to adequately represent the test data presented herein or the micro-scale forces presented by Santamarina (2001).

5. Conclusions

Seventy-seven self-supported, unconfined, drained sand specimens were tested to evaluate the use of effective stress expressions, by Lu et al. (2010) and Han and Vanapalli (2016), to infer and quantify the shear resistance of near-surface soils. These seventy-seven tests were classified into three discrete cases and recorded on high-speed videography, each demonstrating a different failure mode: (1) dry specimens exhibit a progressive bulk failure, (2) unsaturated specimens exhibit a distinct shear failure plane when tested at a degree of saturation of 24%, and (3) saturated specimens exhibit a viscoelastic slumping when the degree of saturation is between 70.5% and 76.5%.

Based on these laboratory findings, significant changes to the effective stress expressions by Terzaghi and Peck (1948), Lu et al. (2010) and Han and Vanapalli (2016), are needed to calculate the shear resistance at discrete intervals and explain the transition from one test case to the next. Furthermore, the effective shear expressions are not representative of the underlying physics or actual behaviour of unconfined or near-surface soils. These findings expose the limitations of effective stress expressions and suggest a revised non-linear three-dimensional unconfined Mohr failure envelope based on an applied external stress for near-surface soil. The revised Mohr failure envelope indicates that as saturation increases, the shear resistance decreases, and becomes less dependent on the initial density state.

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Figure captions

Figure 1. Grain size distribution curve of the SP sand.

Figure 2. Camera orientation for observation of UD failures.

Figure 3. Full spectrum SWCC derived from experimental data for $I_d = 0.54$ tested using the Fredlund device and fit empirically with the Fredlund and Xing (1994) model.

Figure 4. Typical UD_K failure ($I_d = 0.21$ shown).

Figure 5. Progressive failure mode of UD_K testing. (a) sample heights, from which strains are observed assume a horizontal plane however, soil grains do extend above this plane; (b) as weight is applied the grains above the horizontal plane in (a) strain until a new critical mass of grains support the weight and develop a new horizontal plane (the resultant of this is a horizontal force over this new critical mass supporting the applied load); (c) once a critical horizontal resistance (at a critical mass) is exceeded sand is ejected horizontally from the sample and a progressive failure is initiated.

Figure 6. UD_K shear strength, κ , as a function of index density, I_d (blue line), Equation 6. The black line defines the functional form of Equation 7 in terms of the applied failure stress and index density, I_d .

Figure 7. Typical UD_P failure ($I_d = 0.21$ shown).

Figure 8. Typical UD_S failure ($I_d = 0.21$ shown).

Figure 9. Typical observation of the onset of UD_S failure. Grey image is the initial condition prior to the addition of water; red overlay is the observation during the 20-second lag defining the onset of failure and the final water increment.

Figure 10. Suction strength and observed failure stress versus saturation levels for sixteen UD_S test.

Figure 11. The revised Mohr failure envelope for unconfined samples at various index densities, I_d .

Table caption

Table 1: Loading increments for UD_K and UD_P testing.

Table 2: Sample uniformity and average post failure moisture contents within the sample for each test type and index density.

Table 3: Results of UD_P testing, and shear strengths calculated from different effective stress-based equations. Average values at each density state are shown in bold.

Table 4: Results of UD_S testing and contributions of effective suctional shear resistance equations.

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Figure 2. Camera orientation for observation of UD failures.

196x178mm (96 x 96 DPI)

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Figure 3. Full spectrum SWCC derived from experimental data for $I_d = 0.54$ tested using the Fredlund device and fit empirically with the Fredlund and Xing (1994) model.

Figure 4. Typical UD_{κ} failure ($I_d = 0.21$ shown).
228x131mm (300 x 300 DPI)

Figure 5.

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Figure 10. Suction strength and observed failure stress versus saturation levels for sixteen UDS test.

Figure 11. The revised Mohr failure envelope for unconfined samples at various index densities, I_d .

386x201mm (72 x 72 DPI)

Table 1. Loading increments for UD_{κ} and UD_p testing.

Test	Compactive Energy	Initial	Stage 2	Stage 3	Stage 4	Stage 5
UD_{κ}	100 kJ/m ³	3-200 g	2-100 g	50 g until failure		
	200 kJ/m ³	4-200 g	1-100 g	50 g until failure		
	400 kJ/m ³	5-200 g	3-100 g	50 g until failure		
	600 kJ/m ³	5-200 g	3-100 g	50 g until failure		
	800 kJ/m ³	1-500 g	5-200 g	5-100 g	50 g until failure	
	1000 kJ/m ³	1-1000 g	1-500 g	5-200 g	5-100 g	50 g until failure
UD_p	200 kJ/m ³	4-200 g	2-100 g	50 g until failure		
	400 kJ/m ³	5-200 g	3-100 g	50 g until failure		
	600 kJ/m ³	5-200 g	4-100 g	50 g until failure		
	800 kJ/m ³	1-500 g	4-200 g	1-100 g	50 g until failure	
	1000 kJ/m ³	1-500 g	4-200 g	2-100 g	50 g until failure	

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Table 2: Sample uniformity and average post failure moisture contents within the sample for each test type and index density.

Index Density I_d	Test Type	Average Specimen Height ¹ mm	Average Specimen Reconstitution Saturation ²	Average Post Failure Moisture Content ³			Average Post Failure Saturation			Total Water Added at Failure g
				Top	Middle	Bottom	Top	Middle	Bottom	
0.70	UD_K	145.93	24.18%	0.02%	0.01%	0.02%	0.1%	0.0%	0.1%	
0.63	UD_K	146.06	24.21%	0.02%	0.02%	0.01%	0.1%	0.1%	0.0%	
0.54	UD_K	146.21	24.03%	0.01%	0.02%	0.01%	0.0%	0.1%	0.0%	
0.42	UD_K	146.35	24.21%	0.02%	0.02%	0.01%	0.1%	0.1%	0.0%	
0.21	UD_K	145.76	24.32%	0.02%	0.02%	0.02%	0.1%	0.1%	0.1%	
0.00	UD_K	145.60	24.22%	0.01%	0.01%	0.01%	0.0%	0.1%	0.0%	
0.70	UD_P	146.12	24.27%	5.13%	5.17%	5.22%	24.0%	24.2%	24.4%	
0.63	UD_P	145.90	24.00%	5.17%	5.20%	5.27%	23.9%	23.9%	24.2%	
0.54	UD_P	145.42	24.17%	5.26%	5.33%	5.32%	24.0%	24.1%	24.1%	
0.42	UD_P	145.35	24.05%	5.36%	5.39%	5.42%	23.8%	23.9%	24.1%	
0.21	UD_P	145.61	24.08%	5.54%	5.58%	5.56%	24.0%	24.2%	24.0%	
0.70	UD_S	146.43	24.14%	14.40%	16.27%	16.48%	63.0%	71.2%	72.1%	115.35
0.63	UD_S	146.65	24.11%	13.72%	16.49%	16.38%	60.1%	72.2%	71.7%	115.24
0.54	UD_S	146.53	24.13%	14.60%	16.19%	16.73%	63.9%	70.9%	73.2%	118.13
0.42	UD_S	146.19	24.11%	14.24%	15.79%	16.65%	62.3%	69.1%	72.9%	115.86
0.21	UD_S	146.85	23.99%	13.04%	14.05%	17.64%	56.4%	60.8%	76.3%	111.13

1. Target sample height is 145 mm for all specimen.

2. Target reconstituted saturation is 24% for all specimen.

3. The moisture content lowest value, 0.01%, is expressed at minimum accuracy of the testing equipment.

Table 3. Results of UD_p testing, and shear strengths calculated from different effective stress-based equations. Average values at each density state are shown in bold.

Index Density I_d	Initial Void Ratio e	UDp Laboratory Results				Contribution of Suction to Shear Strength					Shear Strength, τ				ϕ'_s degrees
		σ_3 kPa	σ_{1f} kPa	τ_{max} kPa	ϕ' degrees	(a) $(\sigma_n - u_a) \tan(\phi')$ kPa	(b) Equation 5 kPa	(c) $\frac{s_e}{\alpha} \left(\frac{n}{s_e^{1-n}} - 1 \right) \frac{1}{n} \tan \phi'$ kPa	(d) $\psi S_r^{\frac{\epsilon}{\alpha}} \tan \phi'$ kPa	(e) Average kPa	(a)+(b)	(a)+(c)	(a)+(d)	(a)+(e)	
0.70		0.00	4.11	2.06		0.62	1.03	0.92	1.13	1.03	1.65	1.54	1.74	1.65	64.37
0.70		0.00	3.78	1.89		0.57	1.03	0.92	1.13	1.03	1.60	1.49	1.69	1.60	65.55
0.70		0.00	3.31	1.65		0.50	1.03	0.92	1.13	1.03	1.53	1.42	1.62	1.53	67.39
0.70		0.00	3.39	1.69		0.51	1.03	0.92	1.13	1.03	1.54	1.43	1.63	1.54	67.07
0.70		0.00	3.75	1.87		0.56	1.03	0.92	1.13	1.03	1.59	1.48	1.69	1.59	65.66
0.70	0.571	0.00	3.67	1.83	38.0	0.55	1.03	0.92	1.13	1.03	1.58	1.47	1.68	1.58	65.97
0.63		0.00	3.39	1.69		0.51	1.02	0.92	1.12	1.02	1.53	1.43	1.63	1.53	66.86
0.63		0.00	3.78	1.89		0.57	1.02	0.92	1.12	1.02	1.59	1.48	1.68	1.59	65.33
0.63		0.00	3.39	1.69		0.51	1.02	0.92	1.12	1.02	1.53	1.43	1.63	1.53	66.86
0.63		0.00	3.78	1.89		0.57	1.02	0.92	1.12	1.02	1.59	1.48	1.68	1.59	65.33
0.63		0.00	4.11	2.06		0.62	1.02	0.92	1.12	1.02	1.64	1.53	1.73	1.64	64.16
0.63	0.577	0.00	3.69	1.84	37.9	0.55	1.02	0.92	1.12	1.02	1.57	1.47	1.67	1.57	65.67
0.54		0.00	3.99	2.00		0.60	1.01	0.91	1.11	1.01	1.61	1.51	1.71	1.61	64.28
0.54		0.00	3.72	1.86		0.56	1.01	0.91	1.11	1.01	1.57	1.47	1.67	1.57	65.26
0.54		0.00	4.23	2.12		0.64	1.01	0.91	1.11	1.01	1.65	1.55	1.75	1.65	63.45
0.54		0.00	3.78	1.89		0.57	1.01	0.91	1.11	1.01	1.58	1.48	1.68	1.58	65.04
0.54		0.00	3.81	1.90		0.57	1.01	0.91	1.11	1.01	1.58	1.48	1.68	1.58	64.93
0.54	0.585	0.00	3.91	1.95	37.7	0.59	1.01	0.91	1.11	1.01	1.60	1.50	1.70	1.60	64.58
0.42		0.00	3.69	1.90		0.55	0.99	0.86	1.05	0.96	1.54	1.41	1.60	1.51	63.31
0.42		0.00	3.51	1.95		0.53	0.99	0.86	1.05	0.96	1.52	1.38	1.57	1.49	64.05
0.42		0.00	3.81	1.84		0.57	0.99	0.86	1.05	0.96	1.56	1.43	1.62	1.53	62.84
0.42		0.00	3.57	1.75		0.53	0.99	0.86	1.05	0.96	1.52	1.39	1.58	1.49	63.80
0.42		0.00	3.27	1.90		0.49	0.99	0.86	1.05	0.96	1.48	1.35	1.53	1.45	65.09
0.42	0.597	0.00	3.57	1.87	36.0	0.53	0.99	0.86	1.05	0.96	1.52	1.39	1.58	1.49	63.80
0.21		0.00	3.45	1.72		0.51	0.95	0.79	0.97	0.91	1.46	1.30	1.48	1.42	61.81
0.21		0.00	2.78	1.39		0.41	0.95	0.79	0.97	0.91	1.36	1.21	1.38	1.32	65.07
0.21		0.00	2.96	1.48		0.44	0.95	0.79	0.97	0.91	1.39	1.23	1.41	1.35	64.12
0.21		0.00	3.01	1.51		0.45	0.95	0.79	0.97	0.91	1.40	1.24	1.42	1.36	63.87
0.21		0.00	3.33	1.66		0.49	0.95	0.79	0.97	0.91	1.44	1.29	1.46	1.40	62.36
0.21		0.00	3.01	1.51		0.45	0.95	0.79	0.97	0.91	1.40	1.24	1.42	1.36	63.87
0.21		0.00	3.08	1.54		0.46	0.95	0.79	0.97	0.91	1.41	1.25	1.43	1.37	63.51
0.21		0.00	3.25	1.62		0.48	0.95	0.79	0.97	0.91	1.43	1.27	1.45	1.39	62.72
0.21	0.617	0.00	3.11	1.55	33.9	0.46	0.95	0.79	0.97	0.91	1.41	1.25	1.43	1.37	63.38

Table 4: Results of UD_s testing and contributions of effective suctional shear resistance equations.

Index Density	Saturation at Failure	Total Stress at the Base of the Specimen	(a)	(b)	(c)
			$\psi S_r^\xi \Gamma$	$\psi S_r^\xi \tan \phi$	$\frac{S_e}{\alpha} \left(S_e^{\frac{n}{1-n}} - 1 \right)^{\frac{1}{n}}$
	%	kPa	kPa	kPa	kPa
0.70	76.4%	2.83	2.88	3.35	1.26
0.70	75.3%	2.81	2.85	3.31	1.26
0.63	75.3%	2.82	2.85	3.30	1.26
0.63	75.6%	2.82	2.85	3.31	1.26
0.63	75.6%	2.81	2.85	3.31	1.26
0.54	75.3%	2.81	2.85	3.28	1.25
0.54	76.2%	2.80	2.87	3.30	1.25
0.54	75.9%	2.81	2.86	3.29	1.25
0.42	74.1%	2.78	2.82	3.04	1.18
0.42	73.1%	2.77	2.79	3.02	1.18
0.42	74.1%	2.78	2.82	3.04	1.18
0.21	70.6%	2.75	2.72	2.72	1.09
0.21	73.2%	2.76	2.79	2.79	1.09
0.21	73.2%	2.76	2.79	2.79	1.09
0.21	71.0%	2.74	2.73	2.73	1.09
0.21	71.2%	2.75	2.74	2.74	1.09

Note: The Γ term in (a) is calculated based on the critical state friction angle, of 33.9° , for this material [Winters et al. 2016]

Appendix A: Supplementary Supporting Material for “Near-Surface Soils: Self-Supported Unconfined Drained Sand Specimens”

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A1. Unconfined Drained Self-Supporting Test Instructions

A1.1. Apparatus

The unconfined self-supporting specimens are built on a tabletop using a split mold with a bottom platen. The bottom platen contains a porous stone, Figure A1.1, to allow free drainage through the bottom of the specimen. Instead of a latex membrane around the specimen, the mold is lined with wax or parchment paper for easy removal from the specimen, reducing the potential for disturbance of the reconstituted specimen. The load is applied to the specimen by a technician stacking loose, cylindrical precision weights, shown in Figure A1.1, that have the same diameter of the specimen to ensure even distribution of the weight across the specimen. A range of different weights, from 50 to 1,000 grams, are used as the load is applied to the specimen. The heavier weights are brass while the lighter weights are made out of aluminum.

Figure A1.1. Unconfined self-supporting test mold, platen, weights and wax paper.

A1.2. Test specimen preparation

The protocols outlined herein for unconfined self-supporting tests for near surface material applications were developed through research by the Engineer Research and Development Center's Geotechnical and Structures Laboratory.

A1.2.1 Specimen size

The protocol developed for unconfined self-supporting testing uses a reconstituted specimen with a height and diameter of 145 and 71 mm, respectively.

A1.2.2 Preparing mold

The first step in preparing the mold for unconfined self-supporting tests is to attach wax or parchment paper to the bottom platen, Figure A1.2B. The paper should fit around the outside diameter of the platen and overlap a half inch to one inch. If the paper does not overlap, the soil could stick to the mold as the mold is being removed; however, too much overlap in the paper could create a bulk in the paper during sample reconstitution and create cavities in the side of the specimen. The paper should be taped so that the paper fits snugly around the platen, creating a cylindrical shape around the platen, being careful that the diameter at the top of the paper is the same as the diameter near the platen. If the paper diameter is smaller at the top, the paper will not lay smoothly inside the mold as the specimen material is added during reconstitution and will cause cavities on the edges of the specimen.

Figure A1.2. Preparing mold for unconfined self-supporting specimen test.

With the wax or parchment paper attached to the platen, place the mold around the paper. At least two inches of paper should be visible above the top edge of the mold, Figure A1.2C. Using scissors, cut vertical slits in the paper $\frac{1}{2}$ to $\frac{3}{4}$ inches wide down to the top edge of the mold, Figure A1.2D. This allows the paper to be folded over the top edge of the mold and remain smooth on the inside of the mold. Secure the paper slits to the outside edge of the mold using rubber bands, Figure A1.2E. Measure from the top of the mold to the top of the platen.

A1.2.3 Sample reconstitution

Specimens are reconstituted using a modified moist tamping method similar to Bradshaw and Baxter (2007). The focus of this approach for sample reconstitution is maintaining a uniform compactive effort throughout the sample.

Oven dried material and water are thoroughly mixed to produce a target water content. The material is stored in a covered container for 10 to 15 minutes prior to compaction. When mixing the material, ensure additional material is accounted for in excess of what is required for the specimen. After the material is mixed and has been covered for the appropriate time, three samples from the mixture should be removed for determining the water content in accordance with ASTM Standard Test Method D2216. This moisture content is to verify the material is at the specified water content just prior to the specimen reconstitution. If the average moisture content error is greater than $\pm 1.5\%$, i.e., the water content is not between 5.319-5.481% for a target moisture content of 5.40%, the test specimen should be discarded.

The combination of number of layers, number of tamps per layer, force per tamp (rammer weight), rammer drop height and diameter of the tamper base directly affect the energy being applied to the specimen, as detailed in Taylor et al. 2017. The top of each compacted layer should be scarified before adding each additional layer. The top of the specimen should not be scarified.

A1.2.4 Verifying specimen dimensions

After the specimen has been formed, measurements from the top of the mold down to the top of the specimen should be taken from the center of the specimen and at least 4 other points over the top of the specimen. Using these measurements and the previously measured distance from the top of the mold to the top of the platen, determine the height of the specimen. Based off the average measured height of the specimen, if the volume is not within $\pm 1.5\%$ of the target volume, i.e., for a 145 mm tall specimen no more than ± 2.18 mm in height differential, the specimen should be discarded.

A1.2.5 Removing mold from specimen

Before the mold is removed from the specimen, the specimen can be carefully moved to a pan or clean surface so that the material can be contained during testing and failure of the specimen. For these study, the specimen was also placed on a raised disk for better image capturing while inside a pan. Once the mold is removed, the specimen should not be disturbed.

As the split mold is separated for removal, care should be taken that a suction between the wax or parchment paper and the mold does not cause disturbance of the specimen. Figure A1.3A shows the specimen after the mold has been removed. Next, the paper is carefully peeled away from the specimen, Figure A1.3B.

Figure A1.1. Removing wax paper from unconfined self-supporting specimen.

Once the paper is removed, the specimen should be inspected for imperfections. Should any cracks or cavities be found, the specimen should be discarded.

A1.3. Unconfined drained tests

Three unique tests, Unconfined Drained Dry (UD_K), Unconfined Drained Unsaturated (UD_P), and Unconfined Drained Saturated (UD_S) are detailed in the following sections. Each test method starts with an identical specimen, reconstituted at a given saturation.

A1.3.1 Unconfined drained dry (UD_K)

The UD_K tests are allowed to air-dry in a controlled temperature environment until no detectable moisture remains. For this research, the specimens were air-dried for 48 hours, and produced immeasurable water contents comparable to that of oven-dried material. The air-dried specimen is then carefully loaded by a technician with precision weights, waiting 20 seconds between each load application. Weight continues to be added until failure. The amount of weight held before failure and the amount of weight at failure is recorded. After failure, moisture content samples are taken in accordance with ASTM Standard Test Method D2216. If the water content of the failed specimen is greater than 0.02%, the test should be discarded.

UD_K tests were also conducted using isopropyl alcohol in place of water. An equivalent volume of isopropyl alcohol was substituted for the water during specimen construction. All other construction and testing methods were the same as UD_K specimens built with water.

A1.3.2 Unconfined drained unsaturated (UD_P)

The UD_P test is conducted immediately after specimen reconstitution. The specimen is carefully loaded by a technician with precision weights, waiting 20 seconds between each load application. Weight continues to be added until failure. The amount of weight held before failure and the amount of weight at failure is recorded. After failure, moisture content samples are taken in accordance with ASTM Standard Test Method D2216.

A1.3.3 Unconfined drained saturated (UD_S)

The UD_S test is conducted immediately after specimen reconstitution. A small container filled with de-aired distilled water and a dropper are needed for this test. Record the total weight of the container, de-aired distilled water and dropper. Uniformly saturate the top of the specimen with the de-aired water using the dropper filled with water in 3 mL increments. Wait 20 seconds between each application. At the onset of failure, suspend saturation. If complete failure does not occur within 20 seconds, continue the saturation process, suspending saturation once failure begins. After failure, record the

final total weight of the container, de-aired water and dropper. The amount of weight held before failure and the amount of weight at failure is recorded. Collect moisture content samples in accordance with ASTM Standard Test Method D2216, from the failed specimen. The saturation of the failed specimen is then calculated based on the final water content at failure.

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A2. Sample Laboratory Preparation Sheet

Free Standing Strength Test				
		Drop Ht + Ring	Volume	Mass Add
Layer No	Energy	mm	m ³	g
1	0.000457	48.71	0.000151	263.51
2	0.000482	50.89	0.000151	263.51
3	0.000508	53.07	0.000151	263.51
4	0.000533	55.25	0.000151	263.51
Ring Height =		9.51		
		Energy (kJ/m ³)=		200.0
		Equiv Energy (kJ/m ³)=		164.5
Saturation =		24%	W % =	5.59%
		<u>MATERIAL :</u> Clean Sand		
		Amount of material (g) =		1,300
		Amount of Water to add to material (g) =		72.61
		1% water added for loss =		73.34
<u>Target Specimen Height :</u>		145.00	mm	
<u>Mold / Specimen Diameter:</u>		72.71	mm	
<u>Hammer Weight:</u>		1188.48	g	
<u>Base Diameter:</u>		50.11	mm	
		Drops per layer =		50
<u>Height of Specimen:</u>		145.21	mm	

		Time/Date Built:		20160708	
Initial Moisture Content:		Scale Used :		A	
		Tare	Tare + Wet Soil	Tare + Dry Soil	
		14.18	72.72	69.61	5.61%
		13.68	64.55	61.84	5.63%
		14.23	72.49	69.38	5.64%
		Actual Initial MC =		5.63%	
		% Error =		0.7%	
Specimen Tested:		Dry		Wet	
Total Time Dried:		48 hours			
Amount of Weight HELD (grams) :		1400			
Amount of Weight @ FAILURE (grams) :		1500			
Final Moisture Content:		Scale Used :		A	
		Tare	Tare + Wet Soil	Tare + Dry Soil	
		13.37	65.99	66.00	-0.02%
		13.30	61.29	61.28	0.02%
		13.34	69.72	69.70	0.04%
		Average Final MC =		0.01%	

A3. Failure Progression Images and Videos

This supplementary material section contains still images and corresponding high-speed videography for each index density tested for each test type. Each set of six images corresponds to a single specimen.

T3.1 Reference table for high-speed videography to Supplemental Images.

Test Type	Index Density	Supplemental Figure	High-Speed Videography
UD _K	0.21	A3.1	A3.1v
	0.42	A3.2	A3.2v
	0.54	A3.3	A3.3v
	0.63	A3.4	A3.4v
	0.70	A3.5	A3.5v
UD _K (isopropyl alcohol)	0.21	A3.6	A3.6v
UD _p	0.21	A3.7	A3.7v
	0.42	A3.8	A3.8v
	0.54	A3.9	A3.9v
	0.63	A3.10	A3.10v
	0.70	A3.11	A3.11v
UD _s	0.21	A3.12	A3.12v
	0.42	A3.13	A3.13v
	0.54	A3.14	A3.14v
	0.63	A3.15	A3.15v
	0.70	A3.16	A3.16v

A3.1 Unconfined drained dry (UD_k)

Figure A3.1 Unconfined Drained Dry (UD_k), Index Density of 0.21

Figure A3.2 Unconfined Drained Dry (UD_K), Index Density of 0.42

Figure A3.3 Unconfined Drained Dry (UD_K), Index Density of 0.54

Figure A3.4 Unconfined Drained Dry (UD_K), Index Density of 0.63

Figure A3.5 Unconfined Drained Dry (UD_K), Index Density of 0.70

Figure A3.6 Unconfined Drained Dry (UD_K), Reconstituted with isopropyl alcohol, Index Density of 0.21

A3.2 Unconfined drained unsaturated (UD_P)

Figure A3.7 Unconfined Drained Unsaturated (UDP), Index Density of 0.21

Figure A3.8 Unconfined Drained Unsaturated (UDP), Index Density of 0.42

Figure A3.9 Unconfined Drained Unsaturated (UDP), Index Density of 0.54

Figure A3.10 Unconfined Drained Unsaturated (UDP), Index Density of 0.63

Figure A3.11 Unconfined Drained Unsaturated (UDP), Index Density of 0.70

A3.3 Unconfined drained saturated (UD_S)

Figure A3.12 Unconfined Drained Saturated (UDS), Index Density of 0.21

Figure A3.13 Unconfined Drained Saturated (UDS), Index Density of 0.42

Figure A3.14 Unconfined Drained Saturated (UDS), Index Density of 0.54

Figure A3.15 Unconfined Drained Saturated (UDS), Index Density of 0.63

Figure A3.16 Unconfined Drained Saturated (UDS), Index Density of 0.70