Assessment of shear strength parameters of moist sands using conventional triaxial tests

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ABSTRACT
Shear strength parameters of moist soil are often required for the design of underground structures since the soil around the pipelines is generally moist. The strength parameters for the sand are usually determined from laboratory tests conducted on dry or saturated sand samples. However, the difference between the behavior of moist sand and dry or saturated sands is well-recognized. Researcher employed different methods through modification of conventional direct shear or triaxial test apparatus for testing of moist sands. This approach is usually complicated and time-consuming yet not flawless. In the present study, conventional triaxial test apparatus is used to assess the shear strength parameters of moist sand. The tests are conducted using a locally manufactured sand with varying moisture contents. Total stress analysis is adopted to interpret the test results for determining the strength parameters. Consolidated undrained test is also conducted on a saturated sand for comparison of the test results with the moist sands.

RÉSUMÉ
Les paramètres de résistance au cisaillement d'un sol humide sont souvent nécessaires pour la conception de structures souterraines, car le sol autour des pipelines est généralement humide. Les paramètres de résistance du sable sont généralement déterminés à partir d'essais en laboratoire effectués sur des échantillons de sable sec ou saturé. Cependant, la différence entre le comportement du sable humide et des sables secs ou saturés est bien connue. Le chercheur a utilisé différentes méthodes en modifiant l'appareil de cisaillement direct conventionnel ou l'appareil d'essai triaxial pour tester les sables humides. Cette approche est généralement compliquée et prend du temps mais n'est pas sans faille. Dans la présente étude, un appareil d'essai triaxial conventionnel est utilisé pour évaluer les paramètres de résistance au cisaillement du sable humide. Les tests sont effectués en utilisant un sable fabriqué localement avec des teneurs en humidité variables. L'analyse des contraintes totales est adoptée pour interpréter les résultats des tests afin de déterminer les paramètres de résistance. Un essai consolidé non drainé est également effectué sur un sable saturé pour comparer les résultats de l'essai avec les sables humides.

1 INTRODUCTION

Assessment of the shear strength of the soil is required for the prediction of the behavior of buried structures such as foundations, pipelines, culverts, etc. Conventionally, the shear strength of soil is assessed using Mohr-Coulomb (M-C) theory, assuming the soil as saturated or dry (for coarse grained soil). However, the soil around structures buried at shallow depth are typically moist and unsaturated. Therefore, the application of the conventional method for assessing the structures in moist and unsaturated soil may lead to erroneous results. For example, Al-Khazaali and Vanapalli (2019) revealed experimentally that the axial pullout force of pipe in unsaturated sand is significantly higher than in saturated sand. Large scale experiments on soil-pipeline interaction conducted with Cornell sand and Tokyo gas sand also suggest that the variation of moisture content of soil around the pipelines should be taken into consideration as the presence of moisture affects the strength parameters of soil and soil-pipeline interaction (Robert 2010). Robert (2017) demonstrated that soil load on buried pipe due to lateral ground movement is higher in unsaturated soil due to suction induced effect on the normal stress. He showed that the conventional M-C model with modification to include the suction effect can be used to predict the pipeline loads realistically.

Fredlund et al. (1978) proposed a modified M-C model within Terzaghi's conventional effective stress framework for shear strength of unsaturated soil as in Equation (1).

$$\tau_f = c' + (\sigma - u_a)_f \tan \emptyset' + (u_a - u_w)_f \tan \emptyset^b \qquad [1]$$

Where, τ_f is the shear stress at failure, c' is effective apparent cohesion, $(\sigma - u_a)_f$ is net normal stress at failure, $\emptyset'$ is effective angle of internal friction, $(u_a - u_w)_f$ is matric suction at failure, $\emptyset^b$ is the angle of internal friction with respect to matric suction, u_a is pore air pressure and u_w is pore water pressure. Vanapalli et al.

(1996) later revealed that the angle of internal friction is not significantly influenced by the soil suction and proposed a modification to the equation where $\emptyset^b$ is replaced by $\emptyset'$ and a term for the degree of saturation (Sw) is incorporated with a fitting parameter 'k' (Equation 2).

$$\tau_f = c' + (\sigma - u_a)_f \tan\emptyset' + S_w^k (u_a - u_w)_f \tan\emptyset' \qquad [2]$$

Robert (2017) proposed a similar equation as in Vanapalli et al. (1996) except removal of the fitting parameter 'k' in Equation (2), which is expressed as in Equation (3).

$$\tau_f = c' + (\sigma - u_a)_f \tan\emptyset' + S_w (u_a - u_w)_f \tan\emptyset' \qquad [3]$$

In the above model, the apparent cohesion (c') is intended to account for classical cohesion of soil and suction induced hardening effect on cohesion for unsaturated soil (Robert 2017). For purely cohesionless material, c' can be assumed to be zero. The third term in the equation is suction induced macroscopic stress mobilized in terms of shearing resistance, which is independent of the normal stress. This term is expressed as a contribution to the apparent cohesion for unsaturated soil (Robert 2017). Thus, the M-C model for unsaturated cohesionless sand can be expressed in the conventional form as in Equation (4):

$$\tau_f = c' + (\sigma - u_a)_f \tan\emptyset' \qquad [4]$$

Where, $c' = S_w (u_a - u_w)_f \tan\emptyset'$.

The models proposed for the shear strength of unsaturated soil reveals that the apparent cohesion is widely used to account for the intergranular bonding stress due to suction (suction stress). Lu and Likos (2006) defined the suction stress as the isotropic interparticle stress (termed as "isotropic tensile stress") arising from capillary mechanisms in unsaturated sand, which has an equivalent meaning of the "apparent cohesion". The isotropic tensile strength (σ_{ti}) can be obtained through a linear extension of the M-C failure envelope with total stress Mohr circles (Figure 1).

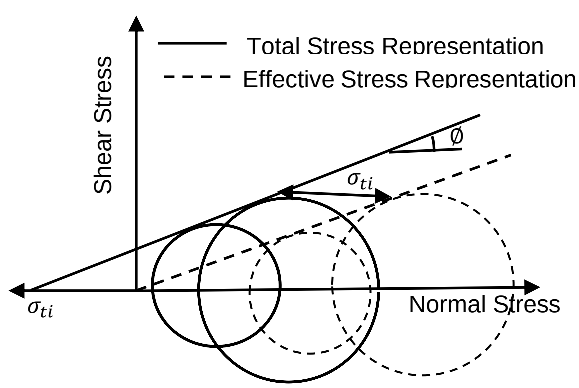


Figure 1: Isotropic tensile strength (σ_{ti}) relating the total stress and effective stress frameworks (after Lu et al. 2009)

The isotropic tensile stress, σ_{ti} (or the suction stress) can be considered additive to the total stresses to define effective stresses for assessing the shear strength within the effective stress framework (see Figure 1). Thus, the suction stress can be used to define shear strength behavior of unsaturated sand under various suction conditions. In this approach, angle of internal friction is assumed to be independent of matric suction, which is consistent with the results of direct shear tests of unsaturated silty sands (Edodaski and Chiba sands) reported in Gallage and Uchimura (2016).

Different experimental techniques are used to determine the parameters for the M-C model for the assessment of shear strength of unsaturated soil. The most rigorous approach involves modification of conventional triaxial apparatus to accommodate independent measurement and control of pore-air and pore-water pressures and the resolution of air and water components of volume change (Wulfsohn et al. 1998, Fredlund and Vanapalli 2002). Researchers also employed modification of direct shear and triaxial apparatus to perform testing on soils under constant suction (Gan and Fredlund 1988, Nam et al. 2011, Maleki and Bayat 2012, Gallage and Uchimura, 2016 and others). However, the modification of conventional triaxial and direct shear apparatus is complex and prohibitive for application in engineering practice. Testing using the modified equipment also requires skilled personnel and consumes a longer time to conduct test under desired matric suction (Bai and Liu 2012; Al-Khazaali and Vanapalli 2019). Another method is using an indirect approach where the suction related information is separately obtained using soil-water characteristic curves (SWCC). Then shear strength of unsaturated soil with respect to the suction is predicted from the extension of total stress approach accumulating the SWCC data, saturated soil property, and conventional shear strength test data (Oh et al. 2008, Vanapalli et al. 1996; Khalili and Khabbaz 1998; Fredlund et al. 1996). However, this method is only applicable for the soils for which the SWCC is developed. An alternative technique is sought by practicing engineer for testing of unsaturated soil using laboratory equipment used for conventional geotechnical engineering practice.

In the current paper, conventional triaxial testing is employed for examining the shear strength behavior of a locally manufactured sand. With depletion of natural granular materials due to the growing construction activities, manufactured sands are frequently used as the backfill materials for buried structures, which are often in moist conditions. It is important to understand the behavior of these materials for assessing the performance of the structure. Triaxial tests are conducted at various initial densities and moisture contents to examine behavior of the soil. The parameters obtained from the tests would be used for the assessment of the soil strengh using the total stress approach in the continuum mechanics framework.

Depending on the moisture content, the granular media can be at four different states of unsaturated conditions namely, pendular, funicular, capillary and slurry states (Mitarai and Nori, 2006). Typically, the pendular state occurs at the degree of saturation of 20%, the funicular state occurs at degree of saturation between 20 and 90%, and the capillary state occurs at the degree of saturation of 90 to 100% (Lu et al. 2009). The objective of the current study is to examine the behavior of the sand at its pendular and funicular states. The capillary state can occur at almost saturated condition of the soil. A set of tests with a saturated condition of the sand was also conducted.

2 TEST MATERIAL

The sand used in the test program is locally manufactured in the province of Newfoundland and Labrador. It is a well-graded clean sand with mean particle size (D_{50}) of 0.742 mm, coefficient of uniformity (C_u) of 5.81, coefficient of curvature (C_c) of 2.04, fines content of 1.3% and gravel content of 0.87%. Standard Proctor compaction test was conducted with the sand, which revealed that the dry unit of the sand is the highest at 0% moisture (Saha et al. 2019). The dry unit weight reduces initially with the increase of moisture content. Beyond 4%, the dry unit weight increases with the further increase of moisture content and after 10 %, the curve moves to the wet side of the compaction curve.

3 TESTING EQUIPMENT

A GDS Standard Triaxial Automated System available at Memorial University of Newfoundland was used in this study. The testing system has a cell of 3.5 MPa capacity with a base pedestal for 38 mm diameter sample. The back pressure and cell pressure transducer have a capacity of 3 MPa pressure with a volume controller. The pore water pressure transducer has a capability of 3.44 MPa. Figure 2 shows a schematic view of the apparatus. Using the loading frame, axial load could be applied at a velocity of 0.00001 to 10 mm/min. An LVDT with a capacity 50 mm is used to measure axial displacements. A 16-bit standard GDS 8-channel data acquisition device is used to collect the test data into a computer.

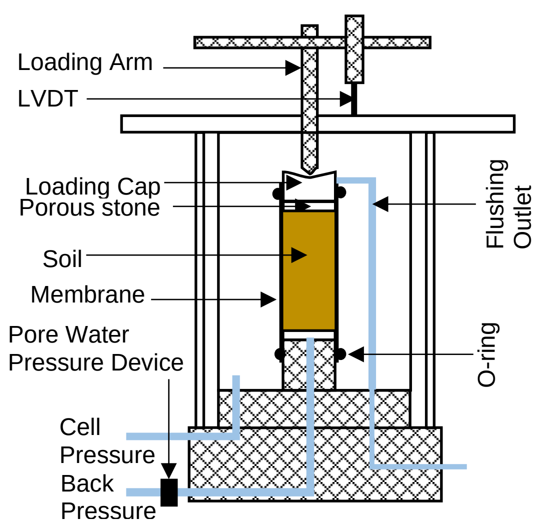


Figure 2: Schematic Diagram of Triaxial Setup

4 TEST METHOD

Triaxial tests were conducted on saturated and unsaturated sand samples. For the unsaturated samples, the moisture content is varied from 0 to ~12% that provided a degree of saturation of 16.2% to 60.5% for the samples. Details of the test program are listed in Table 1.

Table 1: Test Program

Test No.	Sample Condition	Average Moisture Content (%)	Dry Unit Weight (kN/m³)	Initial Void Ratio	Confining Pressure (kPa)
1-3	Unsaturated	2.93	17.58	0.46	50,100, & 200
4-6	Unsaturated	6.98	15.98	0.61	
7-9	Unsaturated	11.88	17.10	0.50	
10-12	Saturated	17.60	18.25	0.41	

Oven-dried sand was used to prepare the samples with addition of water. A porous stone sandwiched between two filter papers was seated on the pedestal of base plate of the triaxial apparatus. The pedestal with porous stone and filter papers was inserted inside a membrane with O-ring. The membrane was stretched and fitted inside a split cylindrical mold. The sample was then poured inside the membrane in five layers of equal thickness. Each layer was compacted using 25 blows of a compaction hammer. After compaction, filter paper, porous stone, and the loading cap was placed on the compacted sand and fitted inside the membrane with O-ring.

For the saturated soil, conventional consolidated undrained tests are conducted. For saturating the sample, the specimen was subjected to CO_2 and deaired water flushing from the bottom to the top. The CO_2 flushing was performed for 3-4 hours whereas the water flushing was performed until water volume in is equal to volume out. The split mold was then dispatched from the specimen after applying suction with the back pressure valve to hold the sample (Figure 3). The height and diameter of the sample were then measured. The specimen was then subjected to saturation with deaired water at high pore-water pressures (back pressure) in several stages for dissolving of any air bubbles into the water. A back pressure in the range of 580 kPa to 670 kPa was applied while a cell pressure of 20 kPa higher than the back pressure was maintained. This procedure provided a B value of around 0.93. While the B value should ideally be 1 for saturated soil, a maximum value of around 0.93 could be obtained during tests. After completion of saturation, consolidation was conducted on the sample at predefined confining pressures. Then, shearing was applied with a loading velocity of 0.065 mm/min under undrained condition.

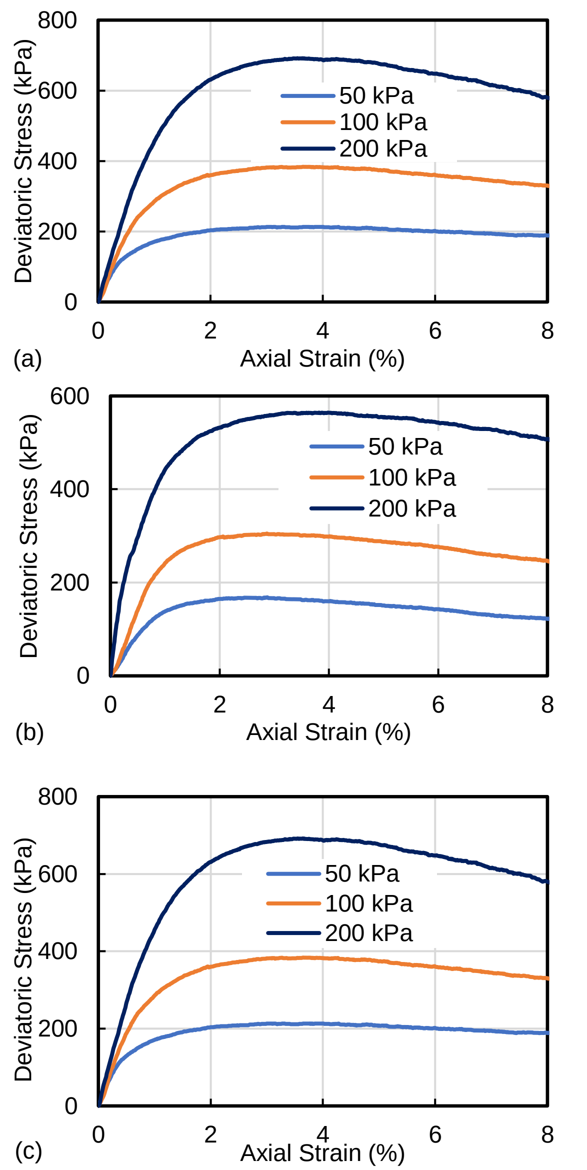
For unsaturated specimen, the mixture of oven-dried sand with a predetermined amount of deaired water was used. No CO2, deaired water flushing, saturation and consolidation were applied on the sample. A small back pressure of -5 kPa was applied into sample to hold it straight with minimum impact on the specimen. Back pressure valve was then closed to make it in an undrained condition before application of confining pressure. Immediately after application of confining pressure, shearing was applied in undrained condition to ensure constant moisture content of the specimen during the tests. The moisture content of the sample was measured after completion of each test for confirmation of the moisture contents. The loading was applied at the same velocity as that used for the saturated sample. Figure 4 shows a typical shearing mechanism observed during the test. The height and diameter of the samples were 73-73.5 mm and 38.8-38.9 mm, respectively.

Figure 3: Prepared Sample

Figure 4: Sample after shearing

5 RESULTS

The stress-strain responses during shearing in the triaxial tests for various conditions of the sand are shown in Figure 5. Shearing was applied in undrained conditions (with closed pore water valve) in all tests. Confining pressures were also applied with the closed pore pressure valve (undrained consolidation) for unsaturated soil to restrict any flow of water into the samples. Although undrained consolidation does not increase the shear strength of saturated soil, an increase of shear strength is expected with the increase of confining pressure in undrained condition for the unsaturated soil (Vanapalli et al. 1999). For the saturated soil, the confining pressure during consolidation was applied under drained condition (CU tests) that contribute to the increase of shear strength of the saturated soil (similar to the unsaturated soil).



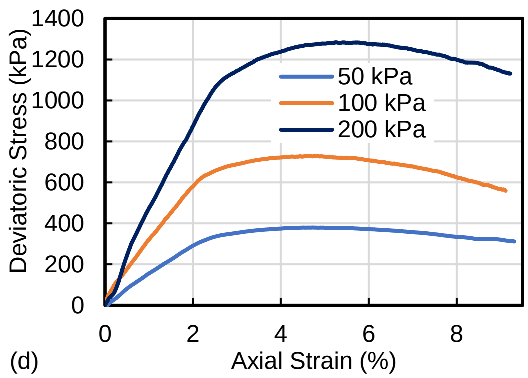


Figure 5: Stress-strain behaviour for sand samples for varying moisture contents (a) 2.93% (b) 6.98%, (c) 11.88% and (d) Saturated

Figure 5 shows significant increase of deviatoric stresses with the increase of confining pressure in all samples. Thus, shear strength of unsaturated moist soil is increased under the undrained confining pressures. The deviatoric stress reaches its peak at 2-4% of axial strain for unsaturated sand whereas the deviatoric stress reaches its peak at 4-6% axial strain for the saturated sand. Degradation of deviatoric stresses after peak values is observed, indicating a dense sand behaviour. Shear strength degradation is more significant at higher confining pressures. Within the strain level considered during the tests (~9%) the residual stress condition was not reached.

To examine the effect of moisture content on the shear strength, the deviatoric stresses under each confining pressure are plotted in Figure 6. For each confining pressure, the deviatoric stress of saturated soil is higher than the stresses for the unsaturated soil with different moisture contents (Figure 6). The unsaturated soil with 6.98% of moisture has the lowest deviatoric stress among all samples which is almost half of the maximum deviatoric stress of the saturated sample. These discrepancies are associated with differences in the densities of the soil specimens. Note that all samples are compacted using the same compaction effort. Same compaction effort in soil samples with different moisture contents are expected to provide different compaction levels to the samples. For the saturated soil, oven-dry sample was placed and compacted in the mold before water flushing and saturation was applied. Therefore, the dry unit weights of saturated specimens are higher than the dry unit weights of the unsaturated specimens. As a result, the shear strength of the saturated sand is higher. Robert (2010) also found higher shear strengths of fully saturated Cornell and Tokyo gas sands than their unsaturated conditions due to higher dry unit weights obtained applying same level of compaction effort.

However, suction was externally applied and controlled in most of the past research on unsaturated test. This suction is key to provide unsaturated soil higher strength than saturated soil (Houston et al. 2008; Maleki and Bayat 2012).

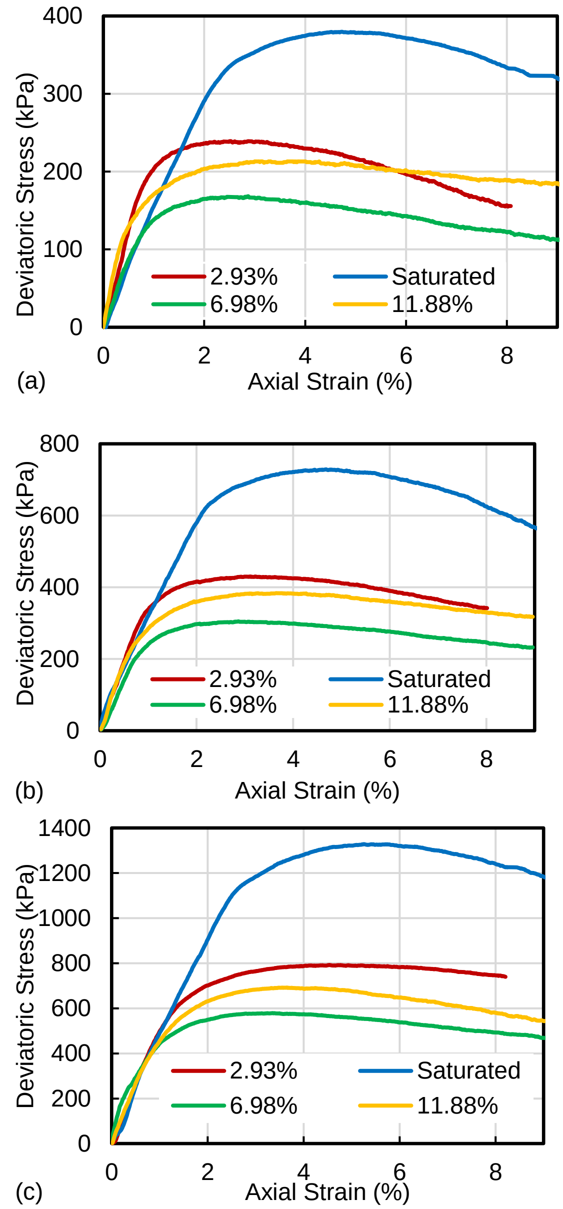


Figure 6: Stress-strain behaviour for sand samples for varying confining pressure (a) 50 kPa (b) 100 kPa (c) 200 kPa

To determine the shear strength parameters, such as the angle of internal friction and apparent cohesion, Mohr-Coulomb failure envelope is plotted as a tangent to the total stress Mohr circles corresponding to the failure points (Figure 7). The conventional straight line approach was found to reasonably represent the Mohr-Coulomb failure envelop for each test, where the slope of the straight line is the angle of internal friction, the intercept of the y-axis is the apparent cohesion and the intercept on the x-axis is the suction stress (Lu et al. 2009). The shear strength parameters obtained at different moisture contents are summarized in Table *2*.

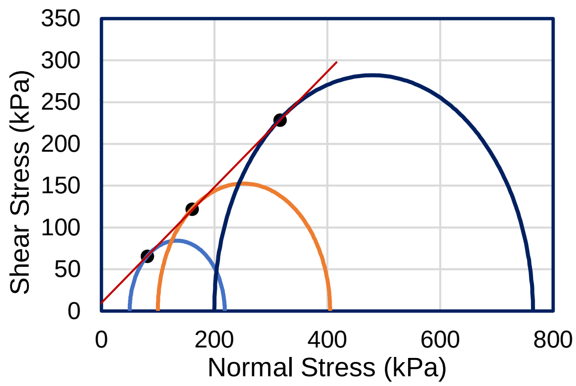


Figure 7: Mohr-Coulomb failure envelope

Table 2 shows that the apparent cohesion of the saturated soil is non-zero. This may be because the soil sample could not be fully saturated using the method employed. The B value of 0.93 was obtained during the test, which do not represent the full saturation condition. A suction of around 10.8 kPa are estimated for the saturated sample used in the tests. The apparent cohesion in the unsaturated soil ranged from 9.8 kPa to 14.3 kPa, which correspond to a suction stress of 14.1 kPa to 18.4 kPa. Soil-water characteristic curve for the soil was not available for comparison with the suctions at the moisture contents of the samples. However, the magnitudes of apparent cohesion (and suction stress) for the moist sands are not significantly high.

Table 2: Shear strength parameters

Moisture Content (%)	Apparent Cohesion (kPa)	Angle of Internal Friction (∘)	Suction stress (kPa)
2.93	15.5	40.3	18.2
6.98	9.8	34.7	14.1
11.88	14.3	37.9	18.4
Saturated	12.6	49.5	10.8

As expected, the angle of internal friction for the saturated sample is the highest in Table 2, which is due to a higher relative density. Among the moist soils, the angle of internal friction is the highest at the moisture content of 2.93% and lowest at the moisture content of 6.98%. To examine if the variation of the angle of internal friction is due to the variation in the density of soil, dry densities of the soil specimens are plotted along with the angles of internal friction in Figure 8. The variation of the angle of internal friction with moisture content for unsaturated sand is found to follow the variation of dry unit weight with the moisture content of the samples (Figure 8). Thus, the changes in the angle of internal friction is likely due to the changes in the dry density (or relative density) of the sand, not due to suction resulting from partial saturation. Schnellmann et al. (2013) also revealed from direct shear test of an unsaturated silty sand with same moisture content and density but different suctions that effective angle of internal friction does not increase significantly with the matric suction. However, the apparent cohesion was increased with the increase of suction.

The suction within unsaturated soil depends on the moisture content and the degree of saturation. The apparent cohesion (a measure of the effect of soil suction) is plotted against the moisture content in Figure 9. It appears the apparent cohesion of the unsaturated sand also decreases with the increase in moisture content up to 6.98% and then increases with further increase of moisture content (Figure 9). Test was conducted up to a moisture content of 11.88%, which is close to optimum moisture content of the material.

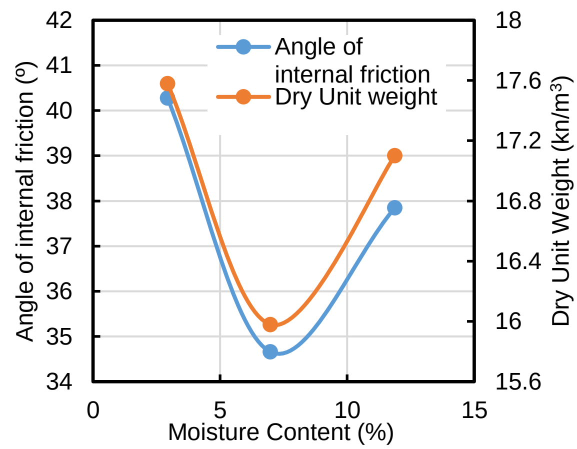


Figure 8: Variation of angle of internal friction for unsaturated soil

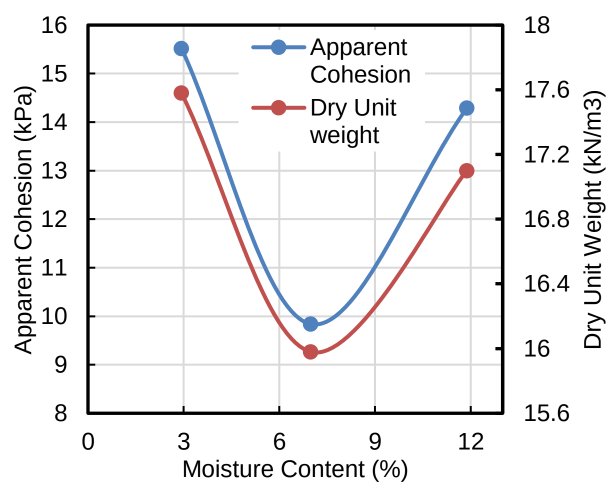


Figure 9: Variation of apparent cohesion for unsaturated Soil

Figure 10 plots the variation of shear strength parameters (apparent cohesion, angle of internal friction) with degree of saturation for unsaturated soil. It shows that the apparent cohesion and angle of internal friction are the lowest at the degree of saturation of 29.5%, which is attained at 6.98% moisture content. The dry unit weight was also the lowest at this moisture content.

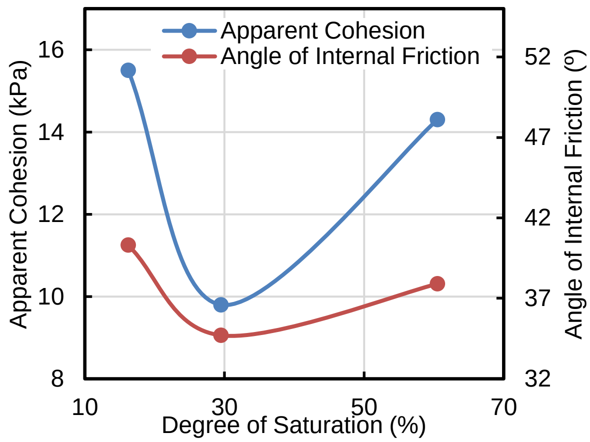


Figure 10: Effect of degree of saturation on shear strength on shear strength parameter

6 CONCLUSION

Triaxial tests are conducted on a locally manufactured sand under unsaturated and saturated conditions using conventional triaxial machine. The findings from the study are summarized below:

- The shear strength parameters of the sand significantly depend on the dry density of the sand regardless of the moisture contents.
- Both apparent cohesion and angle of internal friction of the sand vary with the moisture contents, which initially decreased and then increased with the increase of moisture content (and degree of saturation). The change in the apparent cohesion and the angle of internal friction was found to have strong correlation with the change in the dry density of the soil. The saturated soil sample showed the highest magnitudes of shear strength parameters, which had the highest dry unit weight.
- Apparent cohesion, resulting from suction stress, was not significantly high for the sand (ranged from 9.8 kPa to 15.5 kPa). The corresponding isotropic tensile strength (suction stress) are 10.8 kPa to 18.4 kPa.
- Obtaining full saturation (with B values of 1) of the soil during the test is very challenging. As a result, effect of suction (apparent cohesion) was observed for the saturated soil.
- Although it is difficult to obtain same density level in multiple triaxial tests, a special effort would be required to achieve similar densities in the tests to identify the effect of degree of saturation on the shear strength parameters.

7 ACKNOWLEDGMENT

The financial and/or in-kind support for this research is provided by the Collaborative Research and Development Grant program of Natural Science and Engineering Research Council of Canada, Innovate NL program of the Government of Newfoundland and Labrador, FortisBC Energy Inc. and WSP Canada Inc., which are gratefully acknowledged.

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