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STRENGTH AND DEFORMATION BEHAVIOUR OF A LOCAL SAND

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Abstract: Behavior of buried structure is significantly governed by the strength and deformation behavior of the surrounding soil. In conventional analyses for assessing soil-structure interaction, soil parameters are selected based on typical values available in published literature. Extensive information on soil parameters is available in literature, obtained from various laboratory tests. However, most of these parameters were obtained from testing of standard soils, which are likely to be different from those of local soil where a buried structure is located. The objective of the current study is to investigate the strength and deformation parameters of a local sandy soil used as backfill material in a buried pipeline test. The soil is used in a buried pipe test facility recently developed at the Memorial University of Newfoundland (MUN). The test facility is designed to simulate a ground condition to study the response of a buried pipeline. The soil is classified as well-graded clean sand (SW) according to the Unified Soil Classification System (USCS). The direct shear tests are conducted with the local sand and a silica sand to examine the stress-strain responses and shearing mechanisms with different confining pressures and densities. The study reveals that the shearing resistance of the sand is influenced by the stress level, relative density and moisture content of soil.

1 INTRODUCTION

Soil around buried structures such as pipelines play an important role in the stability of the structure, which depends on the strength and deformation behavior of the soil. Particularly, flexible pipes are susceptible to change in cross-sectional area under load, which depends on the soil-pipeline interaction. For soil-structure interaction analysis, the soil properties are generally estimated based on data available in published literature. Several studies were performed in the past to assess the soil-pipeline interaction based on an experimental approach as well as analytical and numerical modeling (Cheuk et al. 2008; Jung et al. 2013; Weerasekara 2011; Roy et al. 2018). In these studies, different constitutive models for the surrounding soil were employed based on data available in the literature. They obtained significantly different behavior of the pipelines for different soil models. The findings demonstrate the needs for correctly modeling the soil behavior in the soil-pipeline interaction analysis. Researchers employed different types of laboratory tests such as direct shear test, triaxial test, plane strain test, direct simple shear tests for determining soil parameters which are available in the literature (Jung et al. 2013; Roy et al. 2016; Robert 2017). Most of these tests were carried out on standard and known sands such as silica sand, Fraser River Sand, Chiba sand, Cornell sand, RMS graded sand and others (Cheuk et al. 2008; Jung et al. 2013; Robert 2017; Weerasekara 2011). In the current study, a laboratory test program is carried out for characterizing the strength and deformation properties of a local sand. Tests on a silica sand are also conducted for comparison of test results. The local sand is used as a backfill material in a testing facility for investigating pullout behavior of medium density polyethylene pipes. Pipe pullout behavior has been tested for different levels of compaction and moisture contents of the backfill material. The confining stress in the pipe backfill is typically less than the stress in the soil under foundation. Strength and deformation behavior of the soils under these conditions are not well-known.

The shearing strength of sand is achieved with the combination of rolling and sliding friction between grains and interlocking. Interlocking offers resistance against volume expansions (dilation), contributing to the shearing strength of dense sand (Terzaghi et al. 1996). Dense sand loses its interlocking after the peak shear stress is achieved. As a result, the post-peak shear strength is less than the peak shear strength. The peak friction angle increases with the increase in density but decreases with the increase of stress level (Taylor 1948). However, Bareither et al. (2008) revealed that the dilatant behavior showing different peak and post-peak shear stresses depends also on the type of soil deposit. For a weathered sandstone deposit with rounded particles, the dilatant behavior was not observed even at a 95% of the maximum dry unit weight of the soil.

Studies were also conducted on the effects of particle size on the shear strength of soil. Some studies showed that the shear strength increases with the increase of particle size (Kolbuszewski and Frederick 1963; Zolkov and Wiseman 1965). However, Kirkpatric (1965) and Marschi et al. (1972) observed a decrease in shear strength with the increase in particle size. No significant influence of particle size on the shear strength of granular soils was found in Holtz and Gibbs (1956) and Selig and Roner (1987). Most of these studies on the behavior of sand were conducted at high-stress level (> 50 kPa). Besides, only limited study is currently available in the literature on the effect of moisture content on the shear strength/deformation behavior of sand. However, natural soil always contains some amount of moistures which could influence the behavior of the soil. Wei et al. (2018) showed for a soil-rock mixture that the shear strength decreases with the increase of water content. The objective of the current study is to address the effect of particle size, sand dry unit weight and the moisture content on the strength and deformation behavior of a local sand. The behavior of a standard silica sand is also examined for comparison of behaviors of the local sand and the standard silica sand.

2 TEST MATERIAL

A local and a standard silica sand are used for the testing program. Figure 1 shows the grain size distribution of the sands. It reveals the local sand as well-graded clean sand with median particle size (D_{50}) of 0.742 mm. The coefficient of uniformity (C_u) of 5.81 and coefficient of curvature (C_c) of 2.04 are obtained for the sand. It has fines content of around 1.3% and gravel content of around 0.87%. However, the silica sand is poorly graded sand with median particle size (D_{50}) of 0.22 mm. The coefficient of uniformity (C_u) and coefficient of curvature (C_c) are 2.04 and 1.09, respectively. Two types of samples are extracted from the local sand which is denoted as sample A and sample B. Sample A consists of particle passing #4 sieve (opening 4.76 mm) and sample B consists of particle passing #8 sieve (opening 2.38 mm). The local sand contains 99% of particles passing #4 sieve and 82% of particles passing #8 sieve. Materials passing #4 sieve are selected to meet with the requirement of direct shear box size relative to maximum particle size according to ASTM D3080 standard. Materials passing #8 sieve are selected to investigate the effect of removing coarser particles for using in triaxial test. Particles passing #8 sieve can only be used in the triaxial test facility at MUN's Geotechnical laboratory. The silica sand is denoted as sample C. Standard proctor compaction tests are conducted for each of the samples (Figure 2). The compaction curves shown in Figure 2 are typical for cohesionless materials (Das 2010), which are different from the compaction curves for soils with cohesive particles. In cohesionless soil, the unit weight can be maximum at dry condition when the particles can move over each other during compaction/vibration. With addition of water, capillary actions may prohibit particles movements. The effect of capillary tension at lower moisture content dominates over lubrication effect caused by the water, resulting in a lower dry unit weight. At higher moisture contents, the capillary action diminishes that causes an increase in the dry unit weight. The maximum dry unit weight of sample A, Sample B, and Sample C are 19.3 kN/m^3, 19.2 kN/m^3 and 16.40 kN/m^3, respectively that occur at zero moisture content.

3 TEST DEVICE

The direct shear test apparatus at MUN's geotechnical laboratory used for the testing program here utilizes a pneumatic loading concept for applying the vertical load to the sample. Sample loads are obtained using two pneumatic pistons. A small diameter rolling diaphragm piston is used in low load mode which can apply loads from 4 lbs. (17.8 N) up to 100 lbs. (444.82 N). Another is a larger diameter piston used in high load mode that can be used to apply loads up to 1500 lbs (6672.33 N). This direct shear frame is supplied with dial indicators to measure vertical and shear displacements. The device includes 2.5" (63.5 mm) diameter shear rings, porous stones, drainage plates, and water chamber. Maximum shear displacement of 0.8" (20.32 mm) can be applied. Shear displacement rate can be applied at a range of 0.25 mm/min to 1 mm/min.

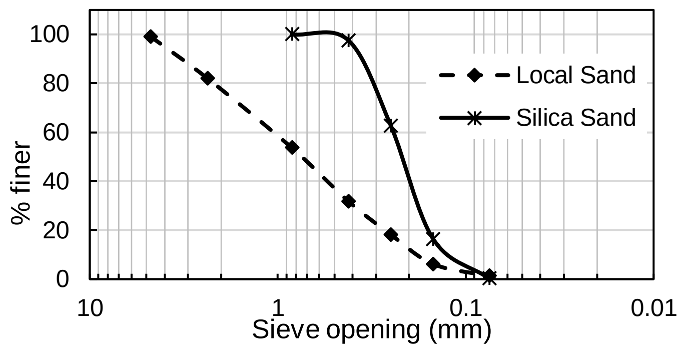

Figure 1: Grain Size Distribution of Local Sand & Silica Sand

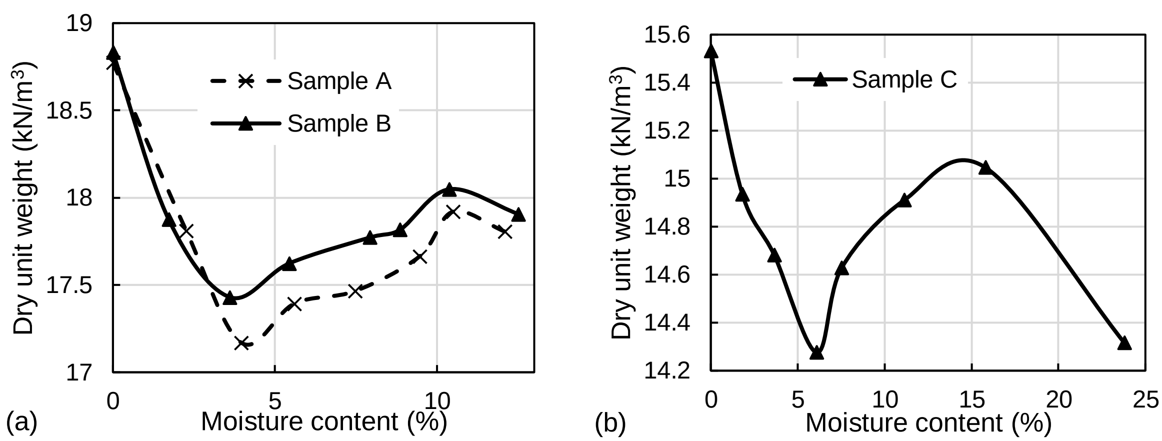

Figure 2: Results of Standard Proctor Compaction tests: a) Sample A & Sample B, and b) Sample C

4 SAMPLE PREPARATION

Samples are prepared with different moisture contents and compaction levels as it is very difficult to prepare sand samples to the same dry unit weight at variable moisture contents and compaction levels. In this test program, samples are compacted in three compaction levels namely high compaction, medium compaction, and no compaction, which are denoted as H, M, and N respectively. To achieve high compaction, the sample is poured into the shear box in three layers. Each layer is compacted with 25 blows of free falling temping rod with efforts to apply the same level of compacted energy to each particle in the shear box. For medium compacted soil, 4 blows were applied in each layer. After compaction, the volume and mass of the soil sample used in the shear box are measured to determine unit weight. The other sample is prepared by filling the shear boxes with the soil spread uniformly over the area of porous stone without compaction. The moist samples are prepared by placing the soil in the shear box immediately after uniform mixing of the soil with predetermined amounts of moisture. The soil in the shear box is placed in the same way as the dry sample. After completion of each test, the actual moisture content of each soil sample is determined through oven-drying. Shear displacement rate of 1 mm/min is applied for all the direct shear tests.

5 TEST PROGRAM

A total of 69 direct shear tests are conducted as summarised in Table 1. Three types of samples at three different compaction levels (high, medium & no compaction) are tested under three normal stresses and different moisture contents to check the effect of dry unit weight as a function of moisture content and relative compaction, as described in the Table 1.

Table 1: Direct shear test program of local sand & silica sand

Test No	Sand Sample Type	Actual Moisture (%)	Compaction Status	Sample ID	Normal stress (kPa)
1-3	Sample A (Local sand particle passing #4 sieve)	0	High Compaction	AH0	12.5, 25, 50
4-6			Medium Compaction	AM0	
7-9			No Compaction	AN0	
10-12		0.80	High Compaction	AH1	
13-15		1.25	No Compaction	AN1	
16-18		1.20	High Compaction	AH2	
19-21		2.0	No Compaction	AN2	
22-24		2.6	High Compaction	AH3	
25-27		2.7	No Compaction	AN3	
28-30	Sample B (Local sand particle passing #8 sieve)	0	High Compaction	BH0	
31-33			Medium Compaction	BM0	
34-36			No Compaction	BN0	
37-39		0.8	High Compaction	BH1	
40-42		1.2	No Compaction	BN1	
43-45		1.9	High Compaction	BH2	
46-48		1.50	No Compaction	BN2	
49-51		2.5	High Compaction	BH3	
52-54		3.00	No Compaction	BN3	
55-57	Sample C (Silica sand)	0	High Compaction	CH0	
58-60			Medium Compaction	CM0	
61-63			No Compaction	CN0	
64-66		1.5	High Compaction	CH1	
67-69		3	High Compaction	CH2	
A = Local Sand particle passing #4 sieve, B= Local Sand particle passing #8 sieve, C= Silica Sand, H=High compaction, M= Medium compaction, N=No compaction, 0= Dry Sample, 1, 2 & 3= Predetermined moisture levels					

6 TEST RESULTS

6.1 Stress-strain responses

Each sample of sand is tested at three normal stresses. For each normal stress, the shear stress against the shearing strains is examined. Shearing stress is represented as a 'stress ratio' defined as the ratio of the shear stress to the normal stress. The volumetric strain examined in term of a dilation rate defined as the ratio of difference of successive two values of vertical displacements and successive values of shear displacements, after Simoni and Houlsby (2006).

Figure 3 shows the variation of stress ratios with shear strain for four conditions of sample A subjected to high compaction. As seen in the figure, the peak stress ratio is not significantly affected by the level of normal stress for the dry sample (Figure 3a). However, for the moist samples, the peak stress ratio decreases with the increase of normal stress. The peak stress ratio is around 1.2 for the dry sand, which corresponds to a peak friction angle of 50°. For the moist sand, the peak stress ratio varies from 0.8 to 1.25. These correspond to friction angle variations from 38° to 51°, with the lowest value for the normal stress of 50 kPa and the highest value for the normal stress of 12.5 kPa. It is also to be noted that post-peak degradation of the shear stress is higher for the dry sand where the post-peak degradation is not significant for the moist sands. For the dry sand, the stress ratio is reduced from peak value of 1.2 to the critical state value of around 07. Thus, the critical state friction angle for the soil is 35°. The peak and post-peak behavior observed for the dry sand is commonly reported in the literature (Tarhouni et al. 2017). However, behavior of moist sand has not been extensively investigated to examine the behaviors.

For the moist samples, the peak stress ratio is higher for soil AH1 having around 0.8% of moisture content than for AH2 having around 1.2% of moisture content, at each of the stress levels considered. The shear strength is higher again for soil AH3 having moisture content of around 2.6%. Although similar approach of soil compaction is used in each of the tests, the degree of compaction of the soil samples in the test box might be different due to the presence of different moisture contents. The effects of moisture content on the shear strength are further discussed later in the paper.

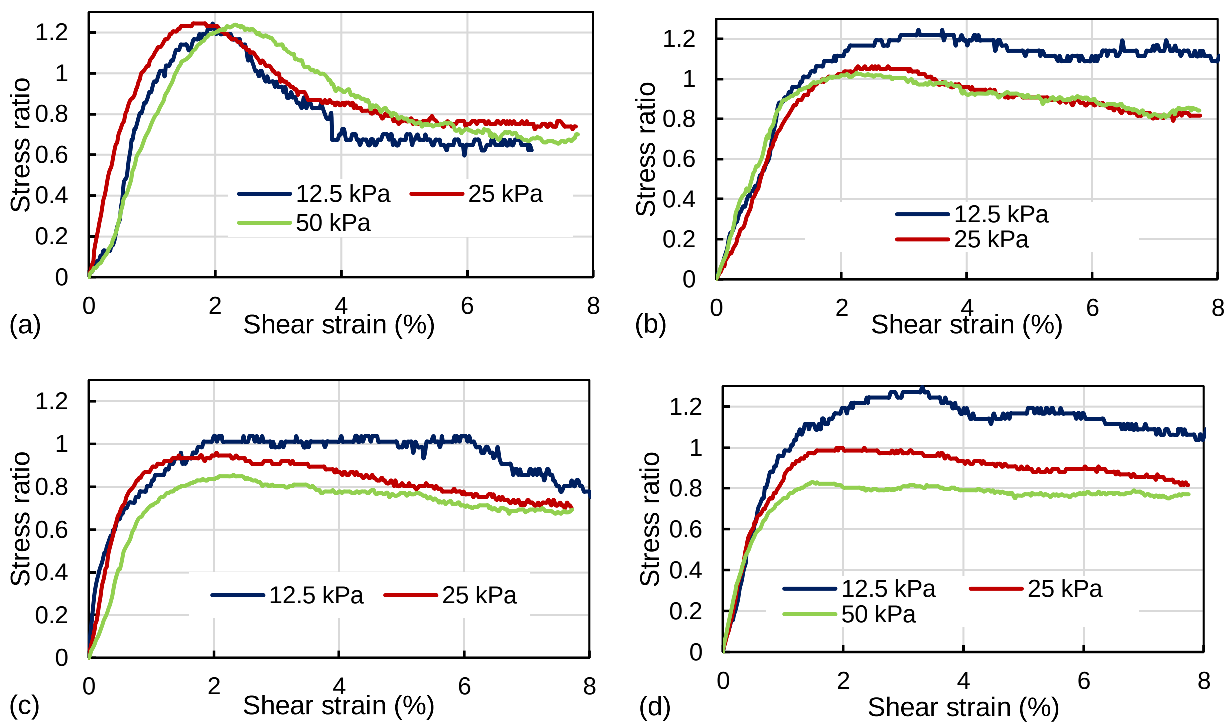

Figure 3: Variation of stress ratio for dense condition of sample A: a) AH0, b) AH1, c) AH2, and d) AH3

Figure 4 plots the calculated dilation rate against the shear strain. Moving average values of the dilation rate are plotted in the figure to minimize the noise in the calculated values. Figure 4 shows that each of the highly compacted samples experiences dilation, although post-peak degradation is not observed for the moist sample (shown in Figure 3). The mechanism of post-peak shear stress and soil dilation for the moist samples require further investigation. In general, the peak dilation rate is almost the same in all three normal stresses for dry sample which is 0.48. There is a rapid drop in dilation rate after reaching the peak. For the moist samples, there are differences in the peak dilation rate at different normal stresses for sample AH2. The difference is not significant for the other samples (AH1 and AH3). After the peak values, the dilation rate decreases gradually with increase in shear strain for each of the tests. The dilation angle for each of the samples eventually reaches to almost zero, which is essentially the critical state. However, the critical state stress ratio is apparently not constant for the moist samples, as shown in Figure 3.

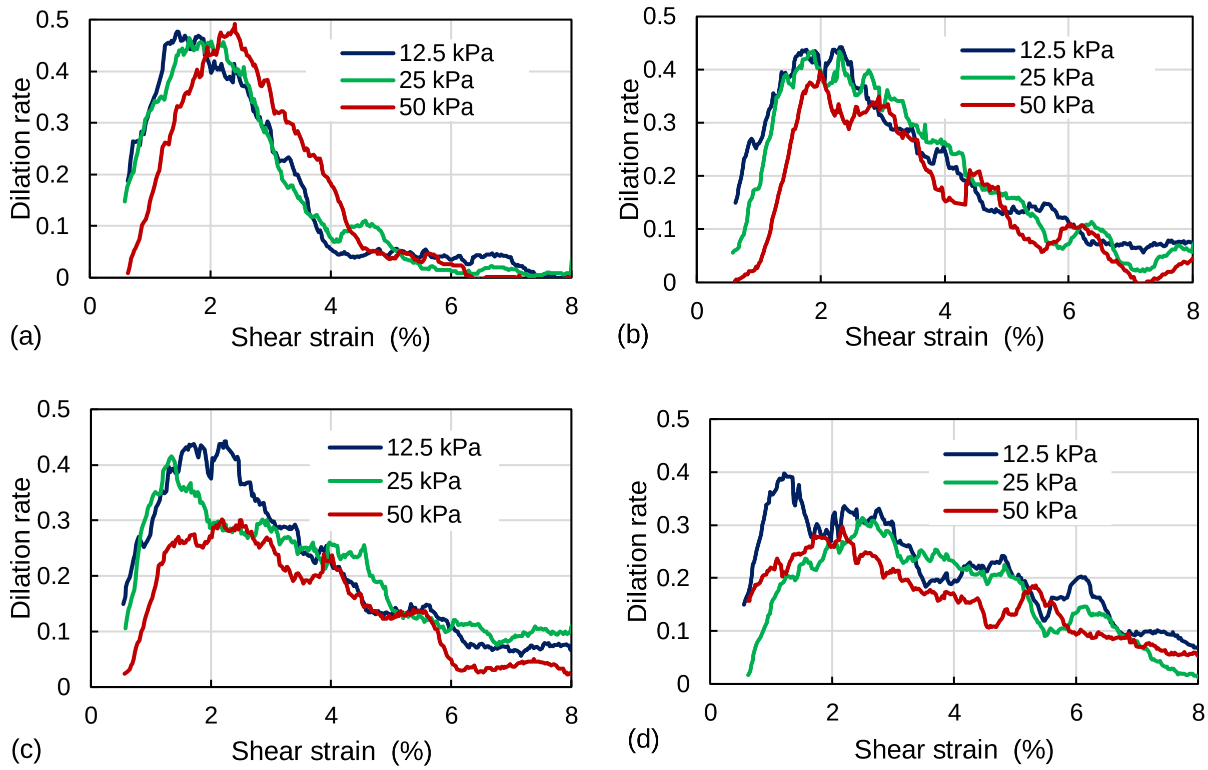

Figure 4: Dilation rates for dense condition of sample A: a) AH0, b) AH1, c) AH2, and d) AH3

For loose condition of the soil, no post-peak degradation in the stress ratio is observed in any of the tests, as expected. Figure 5 shows the variation of stress ratio with shear strain for sample A prepared without no compaction. As seen in the figure, the peak stress ratio is almost same for all normal stresses for the moist samples. The peak ratios are 0.6, 0.55 and 0.5 for samples AN1, AN2, and AN3, respectively, which correspond to the friction angles of 31°, 29° and 26.5°, respectively. The moisture contents in these samples are 1.25%, 2.0%, and 2.7%, respectively. The friction angles for the loose soils are 30% to 40% less than the peak friction angles for the dense soils discussed above.

For the dry loose sand, the stress ratio appears to decrease with the increase of normal stress. The peak stress ratio for the dry sample (AN0) varies from 0.69 to 0.88 (Figure 5a), corresponding to the friction angles variations from 34.5° to 41°. The higher value is for the normal stress of 12.5 kPa and the lower value is for the normal stress of 50 kPa. It is to be noted that as discussed above, the critical state friction angle for the soil in the dense condition is 35°. Thus, the critical state friction angle for the dense soil appears to be same as the peak friction angle of the loose soil at 50 kPa, which is consistent with the concept of the critical state friction angle. However, for the low normal stress of 12.5 kPa, the friction angle in loose condition is significantly higher than the critical state friction angle. Thus, the concept of critical state friction angle may not be applicable at the low confining pressure of the soil. Tarhouni et al. (2017) also questioned the critical state friction angle of sand at low confining pressure from direct simple shear and triaxial tests.

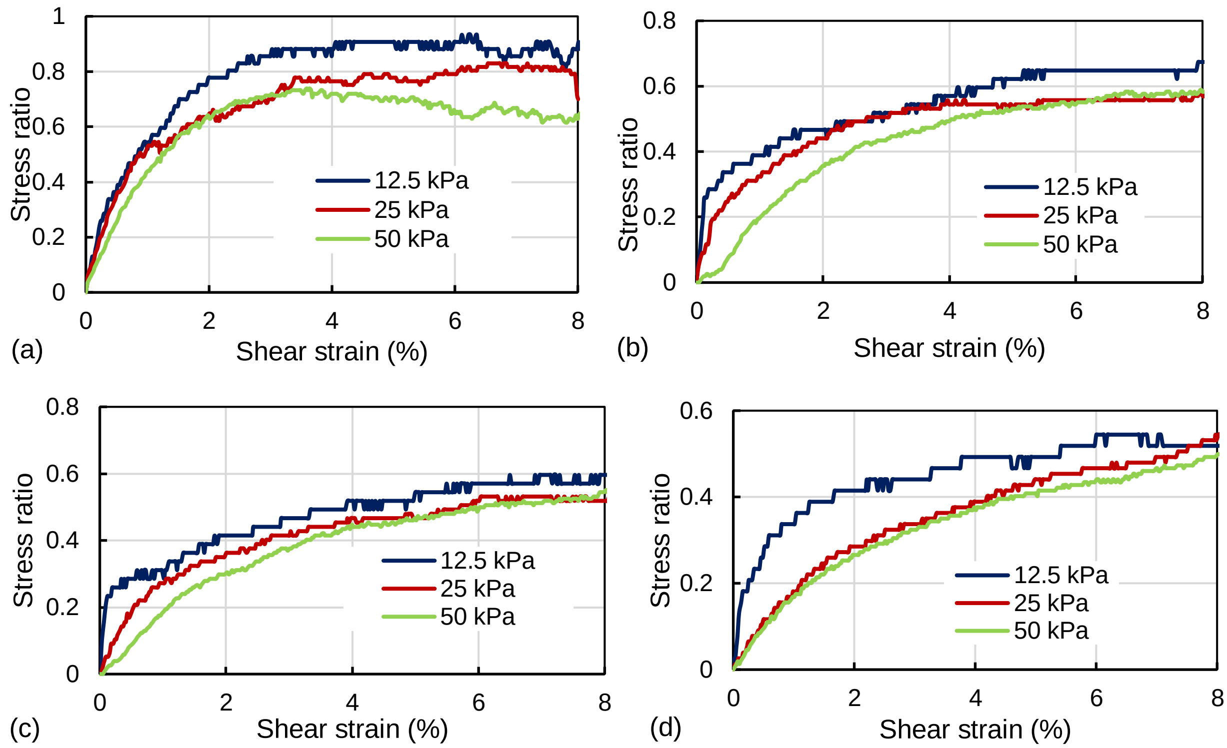

Figure 5: Variation of stress ratio for loose condition of sample A: a) AN0, b) AN1, c) AN2, and d) AN3

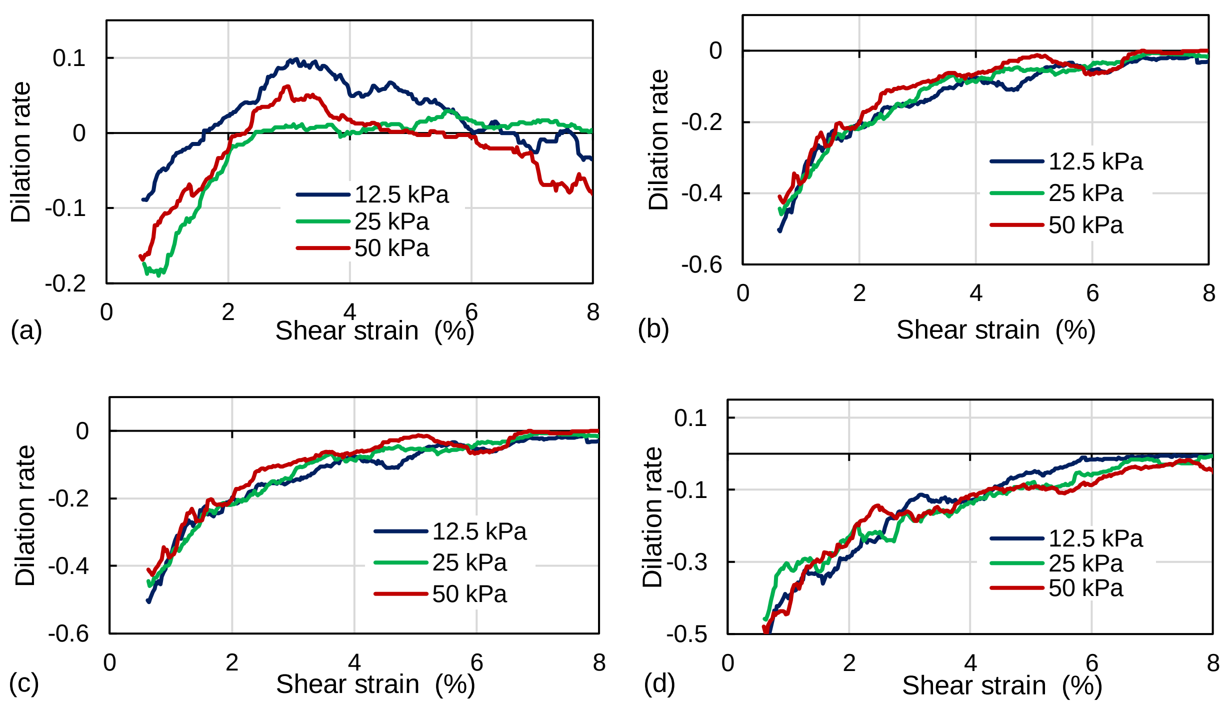

Figure 6: Dilation rates for loose condition of sample A: a) AN0, b) AN1, c) AN2, and d) AN3

The dilation rate for the loose soil is generally negative, indicating decrease of volume during the direct shear tests, as shown in Figure 6. As the shearing of soil occurs at constant volume, the dilation rates become zero at the point of shear failure. However, the increase in volume of the soil (positive dilation rate) is observed in the dry sample during shearing. Figure 6(a) shows that although the dilation rate is negative at the beginning, it increases with the increase of shear strain and reaches the maximum value at the shearing strain of around 3%. The stress ratio is also peak at the shearing strain (i.e., 3%).

Stress-strain responses and volume change responses of Sample B and Sample C are also examined and found to show similar behavior. These results are not included in this paper due to size limitation. However, key observations from those tests are briefly discussed below.

6.2 Peak stress ratios

The peak stress ratios for various soils are plotted against normal stress in Figure 7. The figure reveals that the peak stress ratios are constant beyond the normal stress of 25 kPa. The ratio is higher at the normal stress 12.5 kPa in some tests. The high-stress ratio at 12.5 kPa apparently depends on the moisture contents of the soils. In general, the stress ratio is the highest for dry soils and decreases with the increase in water content.

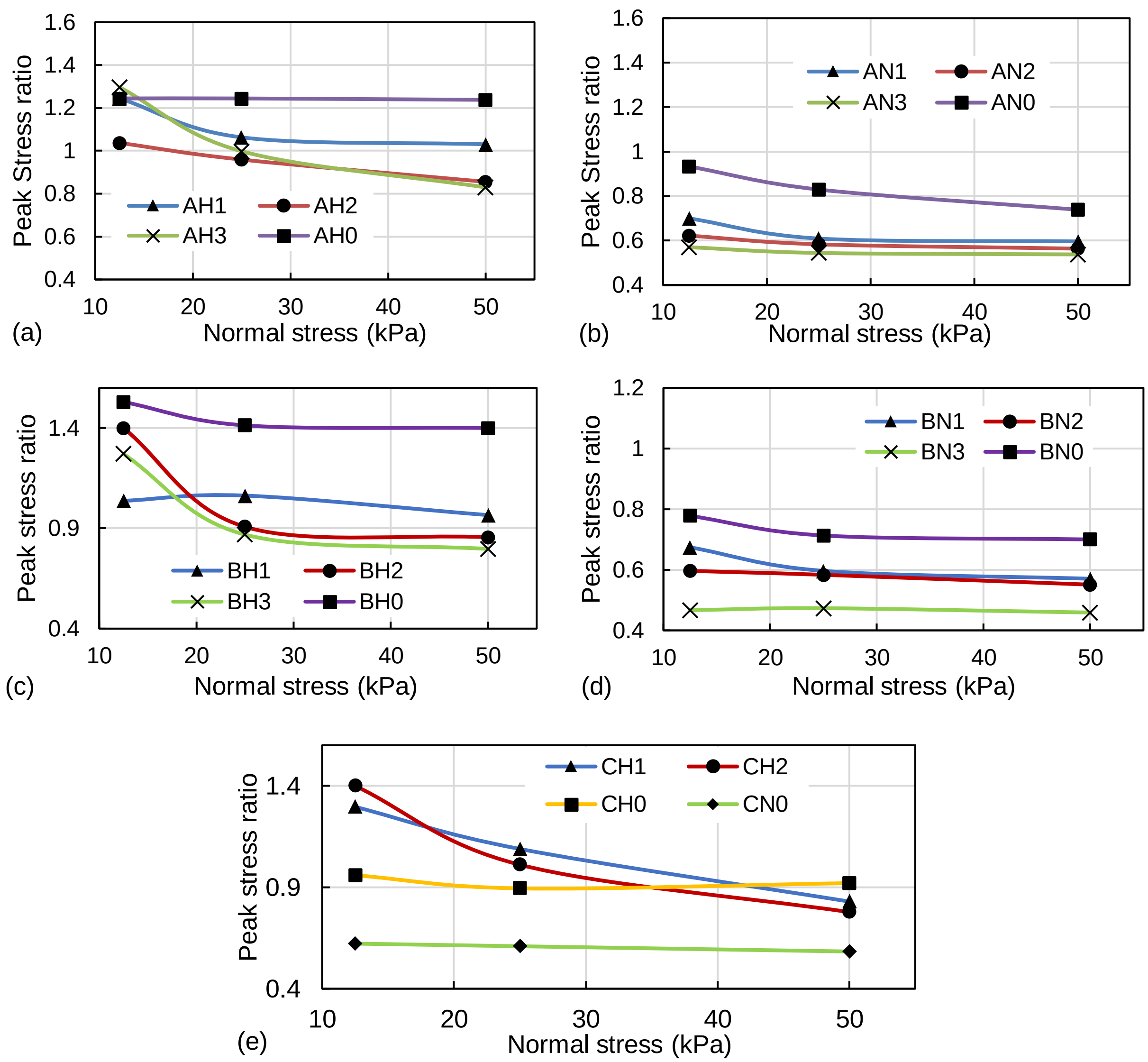

Figure 7: Peak stress ratio against normal stress: a) Dense sample A, b) Loose sample A, c) Dense sample B, d) Loose sample B, and e) Sample C

However, for the silica sand, the peak stress ratio is higher for higher moisture contents, particularly at low-stress levels (12.5 kPa and 25 kPa). For the normal stress of 50 kPa, the peak stress ratio is less for the soil with higher moisture contents.

As mentioned earlier, the density of the soil in the shear box may be different for using different moisture contents, even with the application of the same compaction effort. Thus, the observed variation of the parameters for different moisture contents might be due to the change in the relative density of the soil. To examine the effect, the peak stress ratio is plotted against the dry unit weight of the soil in Figure 8. The dry unit weight of each soil sample was determined during the tests. In Figure 8, the peak stress ratio increases with the increase of dry unit weight. Thus, the reduction of the peak stress ratio with moisture content, discussed above, is potentially due to the reduction of relative density of the soil. Although Tiwari & Al-Adhadh (2014) demonstrated for well-graded sand that the friction angle can decrease for changing from dry state to saturated state at the same relative density. The high-stress ratio observed at lower density for 12.5 kPa of normal stress in Figure 8 requires further evaluation to understand the mechanism.

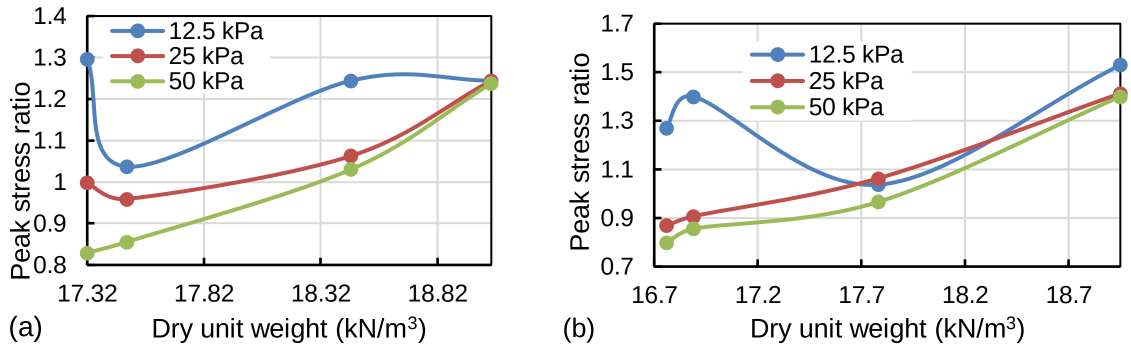

Figure 8: The peak stress ratio with dry unit weight: a) Sample A, and b) Sample B

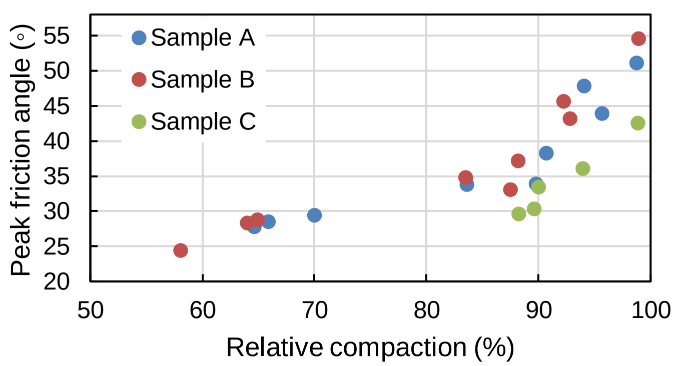

Figure 9: Variation of peak friction angle with relative compaction

The peak friction angle is calculated for each of the soils and plotted against the relative compaction using the maximum dry density obtained from the Standard Proctor Compaction tests (Figure 9). The friction angles for Sample A and B in Figure 9 are similar, indicating that the removal of coarse particles may not significantly affect the friction angle of the soil. The poorly graded silica sand is found to have lower friction angle at the same level of compaction.

7 CONCLUSION

This paper represents a study of the shear strength and deformation behavior of a local sand and a silica sand using direct shear tests. The study reveals that the peak friction angle and dilation rate of soil depends on the moisture content, the compaction level, and the stress level. The behavior of dry soil was found to be different from the behavior of moist soils. In the dense condition, dry soil exhibits a peak stress ratio and post-peak degradation of shear strength. The post-peak degradation is not observed for the moist sand. The peak stress ratios decrease with the increase of moisture contents. This reduction might be attributed to the reduction of relative density of the moist soil. Same level of compaction cannot be obtained for the moist soil using the same level of compaction energy. The stress ratio observed at low-stress level requires further evaluation.

Comparison of the behavior of different soil sample reveals that removal of coarse particles retained on #8 sieve from the soil for use in the triaxial cell may not affect the peak friction angle significantly for the local sand. Strength parameter for poorly graded silica sand is found to be less than the parameter for the well-graded local sand.

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