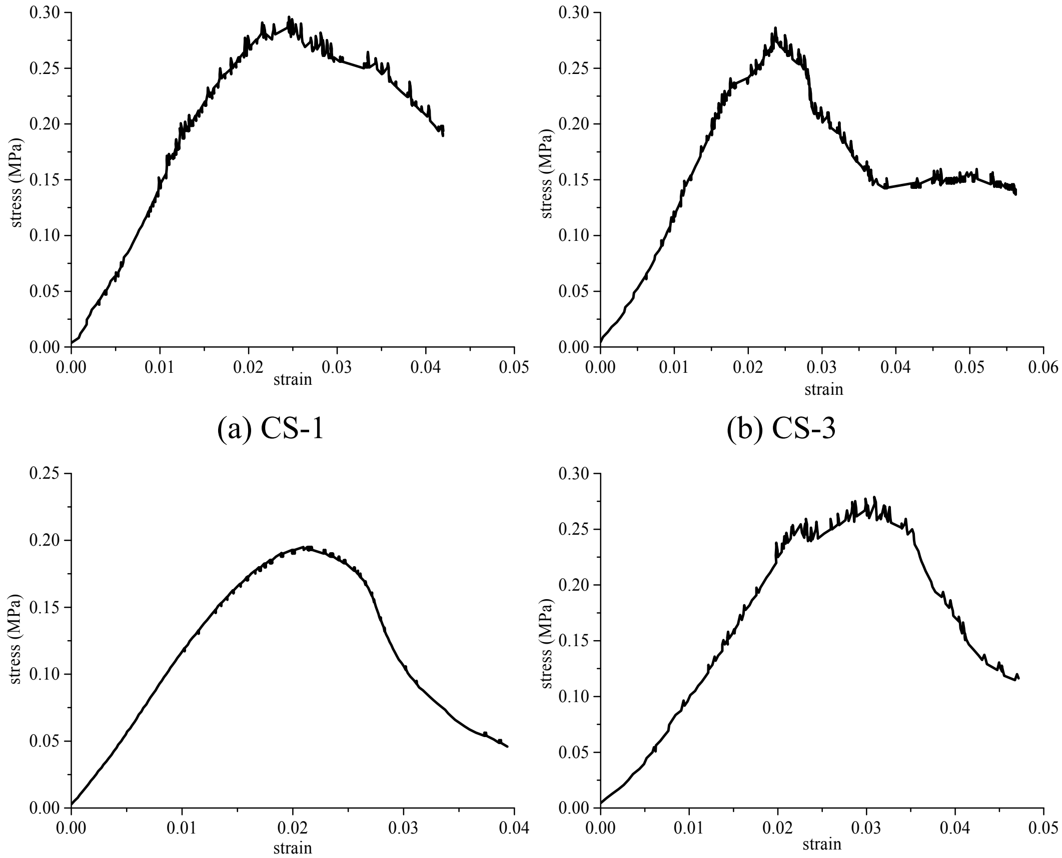
The stress-strain curves of samples

Stress-strain curves of typical samples with the labels of CS-1, CS-3, CS-5 and CS-6 are shown in FIG.4.



(a) CS-1 (b) CS-3

(c) CS-5 (d) CS-6

FIG.4. Stress-strain curves of typical samples

It can be concluded that the stress reaches the maximum value when strain is 0.015-0.04. Besides, the stress-strain curves show a wave-like law, this is because a lot of grout veins were formed inside the samples to influence the failure process.

Results of the range method

As shown in Table 3, based on the range method, the influence of ratio of C-S grout, structural plane angle and injection pressure on the uniaxial compression strength of samples were analyzed.

Table 3. Results of the range method.

I_j	0.72	0.62	0.70
II_j	0.64	0.48	0.44
III_j	0.40	0.66	0.62
k_j	3	3	3

I_j/k_j	0.240	0.207	0.233
II_j/k_j	0.213	0.160	0.147
III_j/k_j	0.133	0.220	0.207
Range R_j	0.107	0.060	0.086

With the range of R_1=0.107 > R_3=0.086 > R_2=0.060, it can be concluded that ratio of C-S grout is the most important factor followed by structural plane angle and injection pressure to influence the uniaxial compression strength of samples.

The failure process of samples

The grout veins played a role of skeleton during the process of compression. As shown in FIG.5, the CS-1 sample is taken as an example to show the failure process of samples during the uniaxial compression test as follows.

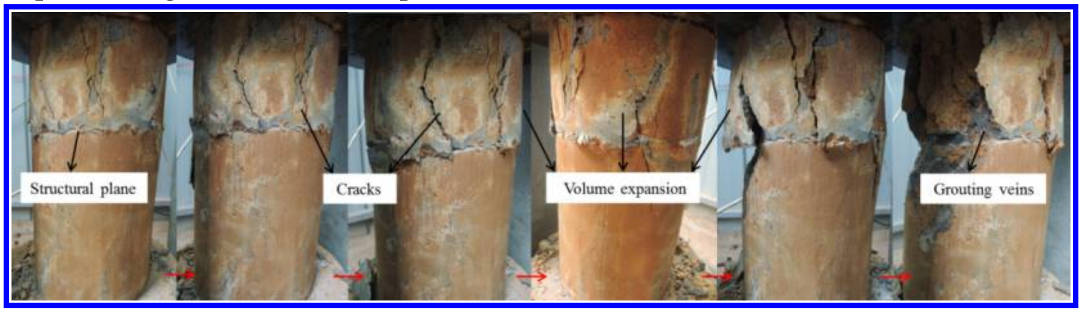


FIG.5. Compressive failure process of the CS-1 sample

(1) As the stress increased, the crack formed and developed above the structural plane.

(2) Both the number and size of cracks increased continuously.

(3) The volume of sample above the structural plane expanded significantly. Besides, the sample slipped slightly along the structural plane.

(4) A large number of spalling appeared finally, indicating the failure of sample.

In a word, the formation and development of cracks, the volume expansion and spalling of sample were all controlled by grout veins.

THE STRENGTHENING CHARACTERISTICS CONTRAST BETWEEN CEMENT GROUT AND C-S GROUT

Grouting strengthening mode

According to the distribution characteristics of exposed grout veins, it can be concluded that the fault gouge with preset structural plane was reinforced by filling, compaction and splitting effect of cement grout and C-S grout.

(1) Filling and compaction effect of grout.

Initially, grout played the filling and compaction roles inside the structural plane, grout veins with the thickness of 15-35 mm were therefore formed (see FIG.6). Due to the low viscosity and good liquidity, the sizes of grout veins formed by cement grout are larger.

(2) Splitting effect of grout

When the injection pressure was high enough to overcome the initial stress, the fault gouge was split by grout with the formation of more grout veins.

(a) C-1, α=0° (b) C-2, α=45° (c) C-5, α=90° (d) CS-1, α=0° (e) CS-7, α=90°

FIG.6. The exposed grout veins formed by cement grout and C-S grout

Grouting strengthening effect

The uniaxial compression strength of samples are the direct index to reflect the strengthening effect of grout for fault gouge. Under the same conditions, the uniaxial compression strength of samples reinforced by cement grout and C-S grout are shown in Table 4, and it can be concluded as follows.

Table 4. Strengthening effect contrast between cement grout and C-S grout

Angle of structural plane (°)	Strength before grouting (MPa)	Cement grout (MPa)	Strength increase	C-S grout (MPa)	Strength increase
0	0.09	0.26	189%	0.23	156%
45	0.03	0.09	200%	0.15	400%
90	0.14	0.23	64%	0.21	50%

(1) The strength of samples increased by 50%-400% after grouting. Both cement grout and C-S grout are effective for the strengthening of fault gouge.

(2) The strength of samples reinforced by cement grout are higher with α=0° and 90°, which can increase by 33% and 14% further, respectively.

(3) However, the strength of samples reinforced by C-S grout are higher with α=45°, which can increase by 200% further.

The consolidation strength of cement grout is higher than that of C-S grout, the strengthening effect of cement grout is therefore better for the samples with α=0° and 90°. However, for the samples with α=45°, the C-S grout is more effective for the improvement of structural plane performance.

The dominant factor of grouting test

According to the results of range method for using the cement grout, with the range of R_3=0.167 > R_1=0.120 > R_2=0.080, it can be concluded that structural plane angle is the most important factor followed by water-cement ratio and injection pressure to influence the uniaxial compression strength of samples. However, the dominant factor of grouting test is the ratio of grout for the use of C-S grout.

CONCLUSIONS

Some key conclusions are summarized as below:

(1) The uniaxial compressive strength of samples with α=0°, 45° and 90° increased by 156%, 400% and 50% after grouting, respectively. It was found that the formation and development of cracks, the volume expansion and spalling of sample were all controlled by grout veins during the uniaxial compression tests.

(2) By using the C-S grout, ratio of grout is the most important factor followed by structural plane angle and injection pressure to influence the strength of samples.

(3) In terms of the strengthening mode, strengthening effect and the dominant factor, strengthening characteristics of cement grout and C-S grout are contrasted.

ACKNOWLEDGMENTS

The study was sponsored by National Natural Science Foundation of China (Grant No. 51209128 and No. 41272385), and the Ph.D. Programs Foundation of Ministry of Education of China (Grant No. 20130131110032).

REFERENCES

[1] Q. S. Zhang, P. Li, G. Wang et al., "Parameters optimization of curtain grouting strengthening cycle in yonglian tunnel and its application", *Mathematical Problems in Engineering*, 2015.

[2] X. Zhang, *Study on mechanism of grout diffusion and sealing at the process of underground engineering moving water grouting and its application [Ph.D. thesis]*, Shandong University, Jinan, China, 2011.

[3] S. C. Nichols and D. J.Goodings, "Physical model testing of compaction grouting in cohesionless soil", *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 126, no. 9, pp. 848-852, 2000.

[4] A. Bezuijen, *Compensation grouting in sand- experiments, field experiences and mechanisms [Ph.D. thesis]*, Delf:t Doctoral thesis of Delft University of Technology, 2010.

[5] R. Gothall and H. Stille, "Fracture-fracture interaction during grouting", *Tunnelling and Underground Space Technology*, vol. 25, no. 3, pp. 199-204, 2010.

[6] K. Eisa, *Compensation grouting in sand [Ph.D. thesis]*, London: University of Cambridge, 2008.

[7] T. Bolisetti, "Experimental Investigations of Colloidal Silica Grouting in Porous Media", *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 135, no. 5, pp. 697-700, 2009.

[8] W. Hu, "Experimental investigation on mechanical properties and representative elementary volume for chemically grouted fractured rock masses", *CHINA SCIENCE PAPER*, vol. 8, no. 5, pp. 408-412, 2013.

[9] Y. J. Zong, "Mechanical characteristics of confined grouting strengthening for cracked rock mass", *Journal of Mining & Safety Engineering*, vol. 30, no. 4, pp. 483-488, 2013.

[10] Q. S. Zhang, P. Li, "Model test on grouting strengthening mechanism for fault gouge of tunnel", *Chinese Journal of Rock Mechanics and Engineering*, vol. 34, no. 5, pp. 7-7, 2015.

[11] P. Li, "Analysis of fracture grouting mechanism based on model test", *Rock and Soil Mechanics*, vol. 35, no. 11, pp. 3221-3230, 2014.

[12] R.T. Liu, "Experiment and application research on a new type of dynamic water grouting material", *Chinese Journal of Rock Mechanics and Engineering*, vol. 30, no. 7, pp. 1454-1459, 2011.

Effects of Wetting on the Shear Strength of Plastic Silty Sands

Ahmed M Elsharief[1]; Yahia E.-A. Mohamedzein[2]; and Sara H. Saad[3]

[1]Associate Professor, Building and Road Research Institute, Univ. of Khartoum, Sudan. E-mail: aelsharief@yahoo.com
[2]Associate Professor, Civil Engineering Dept., Sultan Qaboos Univ., Oman. E-mail: yahiaz@squ.edu.om
[3]Formerly Graduate Student, Building and Road Research Institute, Univ. of Khartoum, Sudan.

Abstract: This paper presents the results of intensive laboratory testing program investigating the effects of wetting on the drained shear strength and deformation characteristics of silty sands. The materials used were clean medium to fine grained Nile River bed sand and plastic Nile silt. The sand was mixed with 4%, 8%, 12%, 16%, 25% and 40% by dry weight of the Nile silt. The test specimens were prepared at two relative densities 30% and 79.4% and drained direct shear tests were performed on air dry and wetted "submerged" specimens. Results from the tests, i.e. shear stress versus displacement and volume change versus displacement, were analyzed and discussed. The dry specimens showed compression and then dilation during shearing. The volume change behavior of wetted silty sands depends on the silt content and the relative density. With silt content up to 25%, the wetted loose silty sands compressed during shear whereas the wetted dense mixtures showed compression and then dilation during shear. The wetted specimens with fines content of 40% showed compression only during shear. The wetted specimen with (sand + 40% fines) behaved like fine grained soil. Significant drop in shear strength caused by wetting has been measured for all tested specimens. The drop in shear strength was greater for the dense specimens compared to the corresponding loose ones. The drop showed general trend of increase with fines content.

INTRODUCTION

The properties of clean sands pertaining to shear strength have been studied extensively under laboratory and field tests. In nature natural sands are often mixed with plastic or non-plastic fines. The mechanical response of sands which contain significant amounts of silt and/or clay is different from that of clean sands. When granular soils contain a certain amount of fines, the strength characteristics vary with fines content. However they are evaluated as pure sands and correlations of in-situ tests are based on charts and relationships that have been developed for clean sands.

Further understanding of the factors contributing to the shear strengths observed for silty sands, both in laboratory and in the field and the role of fines on the reduction or increase in shear strength, would help in the formulation of a consistent method for strength characterization of sandy soils containing fines.

Khartoum city is covered with thick alluvial formations mainly deposited by the Blue Nile (Mohamed 2001). These formations constitute an upper blanket of very stiff to hard silty clay of high plasticity underlain with loose or medium dense clayey silty sand, and then dense poorly graded sand. The alluvial formations overlie old formations known as Nubian Sandstone formation (Mohamed 2001). Water table depth varies depending on the distance from the Blue Nile. The clayey silty sand or sandy silt are often subject to periodical wetting and drying caused by the fluctuating water table. The shear strength of the clayey silty sand given the variation in fine content and wetting conditions needs evaluation and research.

The objective of this laboratory study is to investigate the effects of fine content (plastic silt clay mixture) and wetting on the shear strength of plastic silty sands.

THE FACTORS AFFECTING SHEAR STRENGTH OF SILTY SAND

Research has been carried out during the last few decades to study the behavior of silty sands. These works indicated that the mechanical response of silty sands is different from those of clean sands and that the amount of non-plastic fines and the void ratio of the samples affect silty sand behavior (Thevanayagam, 1998, Sitharam et al., 2004).

Many factors were found to affect shear strength of silty sands such as confining stress (in triaxial) or normal stress (in direct shear), void ratio, relative density, fines content and soil state (whether dry or wet). Thevanayagam (1998) and Sitharam et al. (2004) observed from experiments on silty sands that the residual strength decreases with increase in the void ratio. Furthermore, the residual strength increases with increasing effective confining pressure. Yamamuro and Lade (1998) mentioned that for loose silty sands, the friction angle indicates the lowest value at the lowest initial confining pressure. The friction angle then increases to its peak value at the highest confining pressure. According to Sitharam et al (2004), increase in relative density results in an increase in the residual strength at a given confining pressure. Thevanayagam et al. (2002) from their experimental studies on undrained strength of silty sand (12% to 32% fines) at a confining pressure of 100 kPa, reported similar behavior of increasing residual strength with increasing relative density.

Sitharam et al., (2004) found that silty sands in the void ratio range of 0.607 to 0.656 show drastic reduction in strength at lower confining stresses and exhibit more dilative behavior at higher confining stresses. Thevanayagam (1998) conducted series of undrained triaxial compression tests to investigate the effect of fines, intergranular void ratio, defined as the void ratio of the original coarser-grain matrix structure if the fines were removed from the structure, and initial confining stress, and to quantify their impact on undrained shear strength of silty sand. Results indicated that; the intergranular void ratio plays an important role on undrained shear strength S_{us} of silty sand. Salgado et al., (2000) performed a series of triaxial and bender element tests to find how the shear strength and small-strain stiffness of Ottawa sand change as an increasing percentage of nonplastic fines are added to it. The tests were conducted on isotropically consolidated sand samples with 0, 5, 10, 15 and 20% nonplastic fines. The results were analyzed to assess both the peak and the critical-state friction angles of clean and silty Ottawa sands. It was observed that the addition of even small percentage of silt to clean sand considerably increases both the peak friction angle at a given initial relative density and the critical-state friction angle. They noted that although small-strain stiffness drops, peak and critical-state strengths increase with increasing fines content.

MATERIALS

The materials used in this study were, coarse sand with sub-rounded quartz particles brought from Nile river bed in the vicinity of Merowi Dam in Northern Sudan, known as Marwa sand and Nile silt from the flood plain of the Blue Nile at Soba area South of Khartoum, Sudan. The physical properties of the sand and the silt are summarized in Table 1. Specific gravity and relative density tests were performed to find the minimum density by ASTM D 4254 and maximum density by ASTM D 4253 for Marwa sand. Based on the Unified Soil Classification System (USCS) the sand is classified as poorly graded sand SP, and the silt is classified as low plastic clayey silt ML.

Table 1. Properties of the Nile silt and Marwa sand

Soil Parameter	Plastic Silt	Marwa Sand
Specific Gravity	2.79	2.81
Liquid Limit	43	non-plastic
Plastic Limit	31	non-plastic
Sand content	20%	100%
Silt Content	70%	0
Clay Content	10%	0
Minimum void ratio	-	0.9
Maximum void ratio	-	0.47
Classification (USCS)	ML	SP

TEST PROGRAM AND METHODS

The test program comprised performing direct shear tests on pure sand and sand mixed with different quantities of fines. The tests were carried out at two density states (loose and dense) and for dry and wetted conditions. Since Marwa sand and the Nile silt consist of different sand, silt and clay contents (Table 1), specific quantities of the two soils were air dried and then mixed to get soil with overall fines content equals (Marwa sand 0%, 4%, 8%, 12%, 16%, 25%, 40% and 60%) of the total dry weight. The sand with 60% silt content represents fined grained soil according to the Unified soil Classification system (USCS).

The samples were prepared by manual tamping in the shear box at two different dry densities. Sufficient quantities of the mixes were weighed to guarantee that all the specimens being prepared for each percentage were identical. The specimens were prepared at the two target densities (void ratios) of the silty sand specimen (ρ_1 = 1.59

g/cm^3 and ρ_2 = 1.80 g/cm^3) (e_1 = 0.77 and e_2 = 0.56) equivalent to relative densities equal to 30% and 79.4%. It was not possible to prepare the dense sand+60% silt. The test was carried out on three similar (identical) specimens having the same fines content and dry density. The vertical normal pressures (50, 100 and 150 Kpa) were applied on the specimens.

As for the wetted tests, wetting was performed by adding distilled water to the specimen in the shear box. It was intended to create or simulate submerged conditions in the field. The specimens with low percentages of fines (0, 4, 8, 12 and 16%) were inundated for two to six hours, whereas the specimens with relatively high percentages of fines (25%, 40% and 60%) were inundated for one day. Compression of the sample was accomplished by applying the vertical stress on it for a time span ranging from 5 minutes for all dry samples to 120 minutes for wet samples with low fines content (0, 4, 8, 12 and 16%) and 24 hours for wet samples with relatively high fines content (25%, 40% and 60%). The samples were then sheared at a suitable horizontal rate of displacement for drained conditions; 0.1 mm/min for pure sand and low fines content (less than 12%) and 0.06 mm/min for high fines content ($\geq$ 16%). These rates were determined from the results of one dimensional consolidation tests following the procedure described in Head (1994). The test data was plotted in two formats, shear stress versus horizontal displacement and vertical displacement versus horizontal displacement (Saad, 2010). The test results were then placed in three groups based on their similarity in response to shearing. The first group covers the stress-strain and stress-volume change relations for dry and wetted pure Marwa sand; the second presents the same relations for sand mixed with 4%, 8%, 16% and 25% silt content and this one could be represented by the sand+8% silt; the third presents the data for sand + 40% and 60% silt.

ANALYSIS AND DISCUSSION

The results from the tests will be analyzed and discussed to gain information on the influences of fines content and wetting "submergence" on the stress-strain relationships and volume change during shear, i.e., dilation and compression of the tested samples for loose and dense states.

Shear Stress versus Displacement Relationship

The shear stresses versus displacement curves for pure sand (Figure 1) are typical of those of other types of sands. The dense sand showed increase in stress with strain up to a peak shear resistance, after which the shear resistance dropped displaying work-softening to ultimate values. The drop was sharper for the higher vertical loads. The loose samples showed a small peak at very low strain followed by a drop in shear resistance, a sign of a slip, then increase in shear resistance to ultimate values for dry and wetted samples. The shape of the wetted dense samples was similar to that of the corresponding dry samples; however, the resistance to shearing was lower than those of the dry samples. The shear stress-strain response of the loose wet samples is similar to that of the loose dry samples except that the peaks are not displayed.

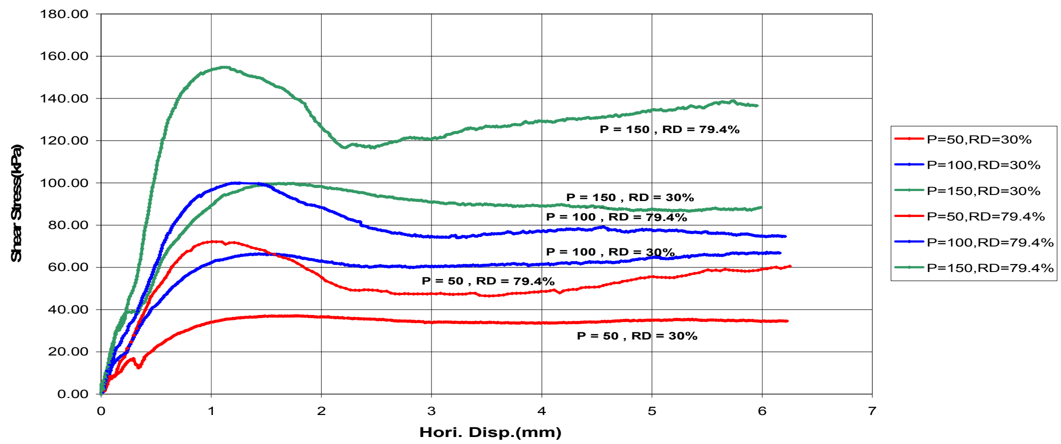


FIG. 1. Shear stress - horizontal displacement curves for dry pure sand at relative densities 30% & 79.4%

The shapes of the shear stress versus displacement curves for dry and wetted sand containing 8% silt samples, which are representative of 4% to 25% silt content, look similar to those of the pure sand, except that the magnitudes of the maximum shear stress and the corresponding displacement are different.

The stress displacement curves for dry sand + 40% silt and 60% (loose) look about similar to those of sand+8%, sand+12% sand+16% and sand+25% ones "for dry samples" with slightly flatter curves around the peak values of stress. However, for the wet sand+40% and 60% silt the stress increased with strain to ultimate value beyond which the resistance to shear was constant (Figure 2). The peak was experienced at a horizontal displacement of 1.0 to 1.5 mm. An important observation is that, the stress strain relationships for wetted dense and loose sand+ 40% silt are similar to those of saturated fine grained soils as represented by sand+60% silt. Therefore, the shape of the stress-displacement curves is affected by the fines content and wetting conditions, combined. As for the dry sand+silt samples, the shapes of the stress-displacement curves look about similar irrespective of the silt content, but the influence of the silt becomes more noticeable for the wetted specimens. When the silt content is 40%, the peak stress is not displayed for all the wetted samples (loose and dense) and the shapes of the stress-displacement curves become similar to those of sand+60% silt or those of fine-grained soils.