Shear strength of the interface between soil and cement grout under different suctions and net normal Stresses

Jian-Hua Yin & Md. Akhtar Hossain
Department of Civil and Structural Engineering – The Hong Kong Polytechnic University, Hung Hom, Kowloon, Hong Kong, China

ABSTRACT
Soil-structure interaction is an important phenomenon encountered in various geotechnical engineering projects. The interaction behavior depends on many factors such as way of forming the interface, soil type, void ratio and normal stress. Besides these, soil-structure interface behavior may be influenced by the matric suction of the soil. In this paper, the interface behavior between compacted completely decomposed granite (CDG) soil and cement grout is presented. A series of interface direct shear tests are performed under different matric suctions and two net normal stresses by using a modified direct shear apparatus. Axis-translation technique was applied to control the pore-water and pore-air pressure. Similar to soil, the interface shear stress increases with matric suction and net stress. Matric suction has significant influence on the hardening-softening behavior of interface. The interface-dilation is also greatly influenced by matric suction. No interface-dilation is observed at lower suction range with higher net stress and at saturated condition. However, interface-dilation is obvious at higher suction range, and the interface-dilation value is lower compared to soil-dilation. The interface strength is higher than the soil shear strength within lower suction range. On the contrary, the interface strength becomes lower than the soil shear strength at higher suction range.

RÉSUMÉ
l'interaction de Sol-structure est un phénomène important produit dans divers projets géotechniques de technologie. Le comportement d'interaction dépend de beaucoup de facteurs tels que la manière de former l'interface, le type de sol, le rapport vide et l'effort normal. Sans compter que ces derniers, le comportement d'interface de sol-structure peut être influencé par l'aspiration inscrite du sol. En cet article, le comportement d'interface entre le sol complètement décomposé compact du granit (CDG) et le coulis de ciment est présenté. Une série d'essais directs de cisaillement d'interface est réalisée sous différentes aspirations inscrites et deux efforts normaux nets en utilisant un appareillage direct modifié de cisaillement. la technique d'Axe-traduction a été appliquée pour commander la pression de la pore-eau et de pore-air. Semblable au sol, aux augmentations d'effort de cisaillement d'interface avec l'aspiration inscrite et à l'effort de filet. L'aspiration inscrite a l'influence significative sur le comportement durcir-se ramollissant de l'interface. L'interface-dilatation est également considérablement influencée par aspiration inscrite. On n'observe aucune interface-dilatation à la gamme inférieure d'aspiration avec un effort net plus élevé et à l'état saturé. Cependant, l'interface-dilatation est évidente à une gamme plus élevée d'aspiration, et la valeur d'interface-dilatation est inférieure comparée à la sol-dilatation. La force d'interface est plus haute que la résistance au cisaillement de sol dans la marge inférieure d'aspiration. Au contraire, la force d'interface devient plus bas que la résistance au cisaillement de sol à une gamme plus élevée d'aspiration.

1 INTRODUCTION

The ultimate shear strength of soil-structure interface is an important parameter for the design and safety assessment of geotechnical structures. Extensive studies have been performed on the interface behavior between soil and different construction materials using different apparatus or devices like direct shear (Potyondy 1961; Acar et al. 1982; Jewell and Wroth 1987; Boulon 1989; O'Rourke et al. 1990; Rao et al. 2000; Chu and Yin 2006), simple shear (Kishida and Uesugi 1987; Uesugi et al. 1990; Paikowsky et al. 1995; Shakir and Zhu 2009), torsional shear (Yoshima and Kishida 1981; Evans and Fennick 1995) or annular shear (Brumund and Leonards 1973). From these studies, it was found that interface shear strength depends on various factors such as relative density, normal stress, dilation angle, moisture content, surface roughness and particle diameter (Yoshima and Kishida 1981; Desai et al. 1985; Kishida and Uesugi 1987; Jewell and Wroth 1987; O'Rourke et al. 1990; Uesugi et al. 1990; Paikowsky et al. 1995; Fakharian and Evgin 1996; Rao et al. 2000; Chu and Yin 2006).

Moisture content, that is, soil suction has significant influence on the engineering behavior of unsaturated soil (Fredlund and Rahardjo 1993; Hossain and Yin 2010). Thus, shear strength at the interface between unsaturated soil and structures may be considered as one of the most important parameters in the design of many Civil Engineering projects. Shear resistance of earth structures, pullout resistance of soil nailing and shaft resistance of deep foundations depend on the shear strength of the concrete grout against the unsaturated soil. However, there is a lack of sufficient literatures regarding interface behavior of unsaturated soil and different construction materials (especially cement grout).

Miller and Hamid (2007) conducted Interface tests between unsaturated Minco silt and stainless steel. The test results show that the interface shear strength increases with the increase of net normal stress and matric suction. The failure envelope and suction envelope are quite linear. However, the shear strength of the soil was greater than the rough interface for similar stress conditions.

Soil-geomembrane interface laboratory tests were performed by Sharma et al. (2007), with provision for the measurement of pore pressures close to the soil-geomembrane interface during shearing process. The tests results suggest that soil suction contributes to shearing resistance at low normal stress values. At higher normal stress values, the interface shear behavior is appeared to govern only by the magnitude of total normal stress.

Hamid and Miller (2009) examined the interface behavior between unsaturated Minco silt and steel (smooth and rough surfaces). The test results indicate that matric suction contributes to the peak shear strength of unsaturated interfaces and post-peak shear strength does not vary with changes in matric suction. Net normal stress affects both peak and post-peak shear strength and the suction envelope for interface is nonlinear.

The main focus of present study is to investigate the interface behavior of unsaturated compacted completely decomposed granite (CDG) soil and cement grout under different matric suctions and two net normal stresses (50 and 300 kPa). A modified direct shear apparatus with a large shear box of 100.07 mm by 100.07 mm square was used to perform interface direct shear tests. The results of interface tests are compared with the shear strength results of the same CDG soil under the same suctions and net stresses.

2 INTERFACE SHEAR STRENGTH EQUATIONS

The interface shear strength is governed by the Mohr-Coulomb failure criteria for saturated case. Potyondy (1961) modified the Mohr-Coulomb's equation as follows with introduction of the coefficient f_a for the reduction of cohesion, and a coefficient f_ϕ for the reduction of the internal soil friction angle in the interface mode:

$$\tau_f = f_a c' + \sigma'_{nf} \tan(f_\phi \phi') \qquad [1]$$

where, τ_f is the interface shear strength at failure; $f_a = c'_a / c'$; $f_\phi = \delta' / \phi'$; σ'_{nf} is the effective normal stress at failure; c'_a is the effective soil adhesion; δ' is the effective interface friction angle; c' is the effective cohesion of soil; and ϕ' is the effective angle of internal friction of soil.

Miller and Hamid (2007) modified the shear strength equation for unsaturated soil proposed by Fredlund et al. (1978) to consider for interface between Minco silt and stainless steel. The equation is as follows:

$$\tau_f = c'_a + (\sigma_{nf} - u_{af}) \tan \delta' + (u_a - u_w)_f \tan \delta^b \qquad [2]$$

where τ_f is the interface shear strength at failure; c'_a is the effective adhesion intercept; $(\sigma_{nf} - u_{af})$ is the net normal stress variable on the failure plane at failure; u_{af} is the pore-air pressure at failure; δ' is the effective interface friction angle associated with the net normal stress state variable $(\sigma_{nf} - u_{af})$; $(u_a - u_w)_f$ is the matric suction at failure; and δ^b is the angle indicating the rate of increase in interface shear strength relative to matric suction $(u_a - u_w)_f$.

Sharma *et al.* (2007) used Bishop's (1959) effective stress equation for unsaturated soil to predict the interface strength of silty sand and geomembrane. The equation is given below:

$$\tau = \alpha + [(\sigma - u_a) + \chi(u_a - u_w)] \tan \delta \qquad [3]$$

where τ is the interface strength; α is the adhesion; σ is the total normal stress; u_a is the pore-air pressure; u_w is the pore-water pressure; δ is the angle of shearing resistance at the soil-geomembrane interface; and χ is a parameter whose value ranges from 0 to 1. Sharma *et al.* (2007) pointed out that Eq. 3 did not accurately predict the measured shear strength. At low normal stresses, it overestimated the shear stress relative to the measured values whereas the reverse was true for high normal stresses. Moreover, the resulting χ values ranged from 0.4 to 2.1 for the various series of tests which is not appropriate.

The shear strength equation for unsaturated soils proposed by Vanapalli et al. (1996) was modified by Hamid and Miller (2009) as follows to predict the shear strength of unsaturated Minco silt-steel interface:

$$\tau_f = c'_a + (\sigma_{nf} - u_{af}) \tan \delta' + (u_{af} - u_{wf}) * \tan \delta' \left(\frac{\theta - \theta_r}{\theta_s - \theta_r} \right) \qquad [4]$$

where θ is the current volumetric water content, θ_r is the residual volumetric water content, and θ_s is the saturated volumetric water content from a SWRC.

3 TESTING APPARATUS

Figure 1 shows a photographic view and Fig. 2 shows the schematic diagram of the modified direct shear apparatus (MDSA) used in present study for direct shear testing of unsaturated soil. The MDSA was manufactured according to Gan and Fredlund (1988) and set up in the Soil Mechanics Laboratory of The Hong Kong Polytechnic University.

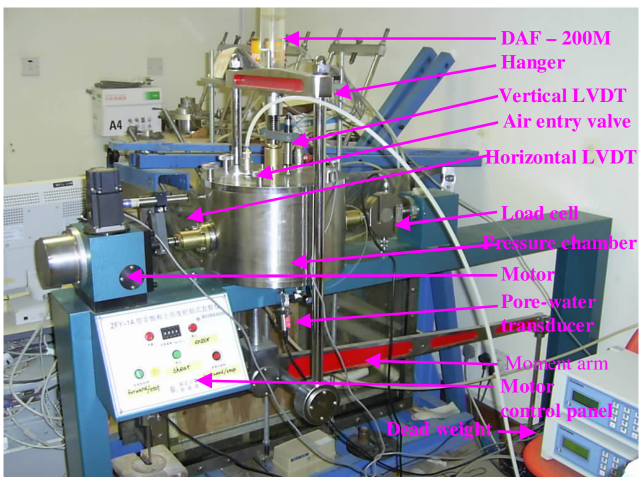

Figure 1. A photograph of the modified direct shear apparatus used in present study

A high air-entry value ceramic disk plate was set at the bottom of the shear box base over the water chamber. One end of the water chamber was connected with an auto volume change (AVC) device and the other end with a diffused air flushing device (model type: DAF 200M, Geotechnical Consulting & Testing System, LLC). Two LVDTs (Linear Variable Differential Transformer) were used for determining the horizontal and vertical displacement. A load cell, calibrated properly before starting the testing program, was used to determine the horizontal shear load. The vertical load was applied by a hanger having a moment arm with dead weights. Necessary corrections were made for calculation/control of the net normal stress and shear load for different air pressure in the chamber.

For interface direct shear testing, some modifications were made in the MDSA used for unsaturated soil. The water chamber was constructed inside the top steel platen instead of shear box base since the bottom part of specimen was cement grout material (shown in Fig. 3). High air entry ceramic disk was set below the water chamber at same level of the bottom of top steel platen.

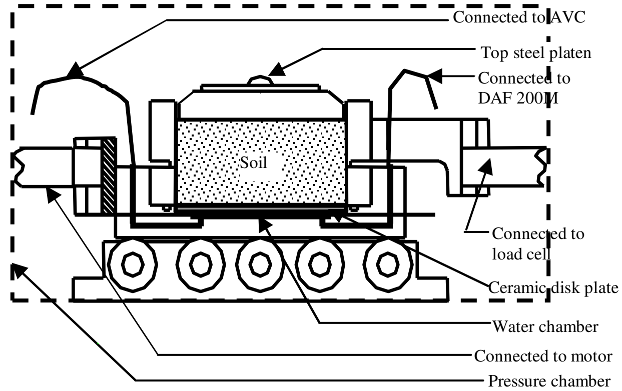

Figure 2. Schematic diagram of modified direct shear apparatus for soil-soil direct shear test

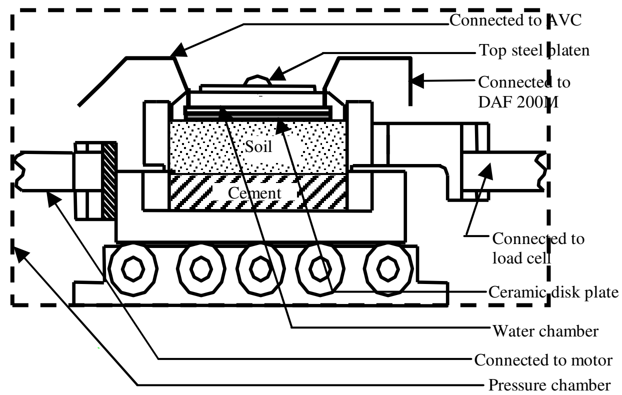

Figure 3. Schematic diagram of modified direct shear apparatus for soil-cement grout interface test

4 PROPERTIES OF MATERIALS

In present study, the soil used was a completely decomposed granite (CDG) soil, which is a typical in-situ soil in Hong Kong. This CDG was taken from a highway construction site at Tai Wai, Hong Kong. All the tests of the soil followed the procedures as described in BS 1377: 1990 to determine the basic properties. The tests included particle size distribution, compaction test, specific gravity, liquid and plastic limit, and permeability test. The particle size distribution of the soil was determined by wet sieving and hydrometer tests following the procedures in BS 1377-2 (1990) and GEO REPORT No. 36 (Chen 1992). According to the Unified Soil Classification System (ASTM D2487-90 1992), the studied CDG soil can be classified as silty sand or SM. The basic properties of the soil are tabulated in Table 1. Figure 4 shows the particle size distribution (PSD) of the soil.

Locally available Portland cement was used to prepare cement grout. The cement was mixed with water at a water/cement ratio of 0.42.The properties of the cement grout material are tabulated in Table 1. It should be noted that the properties of the cement grout material were determined at a curing period of 28 days.

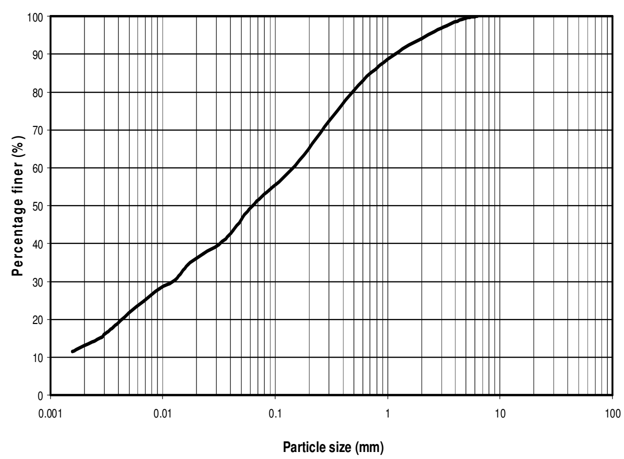

Figure 4. Particle size distribution of completely decomposed granite (after Hossain and Yin 2010)

Table 1. Properties of completely decomposed granite (CDG) and cement grout

Completely decomposed granite		
Specific gravity	-	2.599
Maximum dry density	Mg/m^3	1.75
Optimum moisture content	%	14.3
Gravel	%	5.8
Sand	%	44.1
Silt	%	36.8
Clay	%	13.3
Plastic limit	%	22.7
Liquid limit	%	32.8
Plasticity index	%	10.1
Permeability (k_{20})	m/s	2.36E-08
Cement grout		
Density	Mg/m^3	1.89
Uniaxial compressive strength	MPa	32.1
Secant Young's modulus (E_{50})	GPa	12.6
Poisson's ratio	-	0.21

5 PREPARATION OF INTERFACE SPECIMEN

In the literature of Hossain and Yin (2010), treatment of disturbed CDG soil and preparation procedures of soil specimen for direct shear tests are elaborately discussed. Preparation of soil-cement grout specimen is described in the followings. To simulate the cast in-situ installation, cement grout was poured on the compacted surface of CDG soil.

Before starting the compaction of treated soil in the shear box, the two parts of shear box were tightened together by using screws. The gap between the two part of shear box was filled with vacuum grease. A wooden block (wrapped with scotch tape) having a section of 100 mm by 100 mm and a height of 18 mm was placed at the bottom of the shear box. It should be noted that the height of bottom part of shear box is 20 mm.

The treated soil was compacted over the wooden block in two layers having a thickness of 10 mm each. Each layer was compacted at optimum moisture content of 14.3% to achieve a controlled dry density of 1.663 Mg/m^3, which was 95% of the maximum dry density of 1.75 Mg/m^3 obtained using a standard compaction test. The required mass of wet soil for a particular layer was calculated, placed inside the shear box and compacted. After completing the compaction, the weight of compacted soil was recorded, and the top part of the shear box was covered by another wooden block (wrapped with scotch tape) to prevent movement of moisture from or into the soil.

The amount of cement and water needed to fill a section of 100.07 mm by 100.07 mm and a height of 18 mm with cement grout was calculated before mixing. The cement and water was mixed in such a way that no cement particle could coagulate and no lumps could present in the grout. After the preparation of cement grout, the shear box with the soil was turned over (top part down and bottom part up) and the first wooden block was removed to pour cement grout on the prepared surface of soil. The cement grout was poured smoothly over the prepared surface so that the bottom part of shear box could be filled fully with no air voids.

The cement grout was kept open in atmosphere for about 12-14 hours to facilitate the setting. After setting, the surface of the cement grout was leveled carefully by using a leveler. The cement grout surface and shear box was wrapped with scotch tape to ensure self-curing of cement grout (to simulate the field condition) for a period of approximately 5 days. After completing the curing period, the wrapping tape was removed, and the shear box was turned over again (soil at top and cement grout at bottom) and set on the shear box base kept inside the pressure chamber.

Kulhawy and Peterson (1979) pointed out that when the concrete is poured directly onto compacted soil, a rough interface surface is developed and the shear surface is located in the soil away from the interface. Desai *et al.* (1985) considered that the interface action for many soil-structure interfaces occurs in a thin zone near the interface. In the present study, the interface layer thickness is considered as 2 mm from the grout surface.

6 TEST PROCEDURE

Single-staged consolidated drained direct shear tests were carried out to observe the interface shear strength of compacted CDG soil and cement grout under different matric suctions (0, 50, 100, 200 and 300 kPa) and two net normal stresses (50 and 300 kPa). The test procedure was consisted of three steps: saturation, equilibration of suction and drained shearing at constant suction.

After placing the soil-cement grout specimen on the shear box base inside the air pressure chamber, a porous disk plate was placed over the soil, ample amount of water was poured on the disk plate and the chamber cap was closed. The specimen was allowed to saturate by applying 200 kPa air pressure inside the chamber for about 10 hours. After saturation, the excess water and the disk were removed, the top steel platen fitted with ceramic disk was mounted and the water chamber was connected with AVC and DAF devices. The water chamber was flushed using a pressure controller (after Geotechnical Digital System) to drain out all the air bubbles from the connecting tubes and water chamber. After removing air bubbles, the connecting valves of AVC, DAF and pressure controller devices were closed.

The pre-calculated axial load, air pressure and water pressure were applied (by opening valve of AVC) sequentially to attain the desired matric suction. Vertical deformation and water movement were recorded during the equilibration process. Equilibration was assured when the flow of water essentially ceased. The duration of the equilibration stage depended on the target suction. The water chamber was flushed after the finishing of equilibration.

The specimen was sheared after matric suction equalization was attained. Single-staged shearing was carried out under a drained condition with a constant shearing rate of 0.004 mm/min (similar to soil-soil direct shear tests) until the horizontal displacement reached to 15 mm. The suction in the specimen was maintained constant throughout the shearing. During shearing, the horizontal shear load, horizontal displacement and vertical displacement were measured and recorded automatically in a computer at an interval of two minutes. Shearing was accomplished during a period of approximately 2.5 days. After the completion of shearing, water chamber was flushed finally to measure the diffused air volume. It should be noted that no diffused air bubble appeared after shearing under lower suctions and very negligible amount of air bubble appeared for higher suctions. The specimen was quickly dismantled from the shear box for the determination of wet weight of soil.

7 TEST RESULTS AND DISCUSSION

7.1 Direct shear tests on CDG soil

The failure envelop of CDG soil at saturated condition is shown in Fig. 5, and the effective angle of internal friction, $\phi' = 29.9°$ and effective cohesion, $c' = 0$ kPa are obtained.

The stress-displacement behavior of soil-soil direct shear test under different matric suctions and net normal stresses of 50 and 300 kPa are shown in Fig. 6(a) and Fig. 7(a) respectively. The experimental results indicate that the contribution of suction and net stress to shear strength is quite significant. Shear strength of CDG soil increases with the increase of matric suction and net stress. The stress-strain behavior is significantly influenced by matric suction. A strain-softening behavior is observed at higher suctions (100 to 300 kPa) for lower net stress of 50 kPa. On the other hand, a strain hardening-behavior is observed under entire suction range of 0 to 300 kPa for the higher net normal stress of 300 kPa.

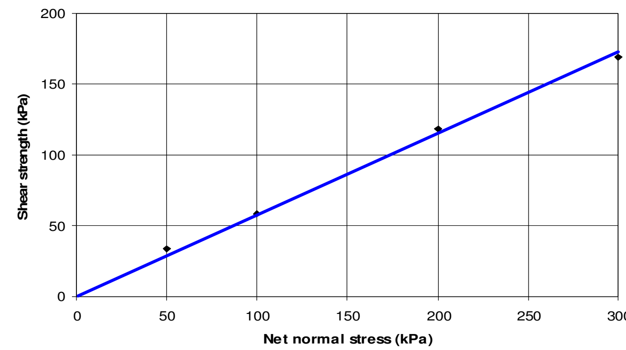

Figure 5. Failure envelope for compacted CDG soil at saturated condition (after Hossain and Yin 2010)

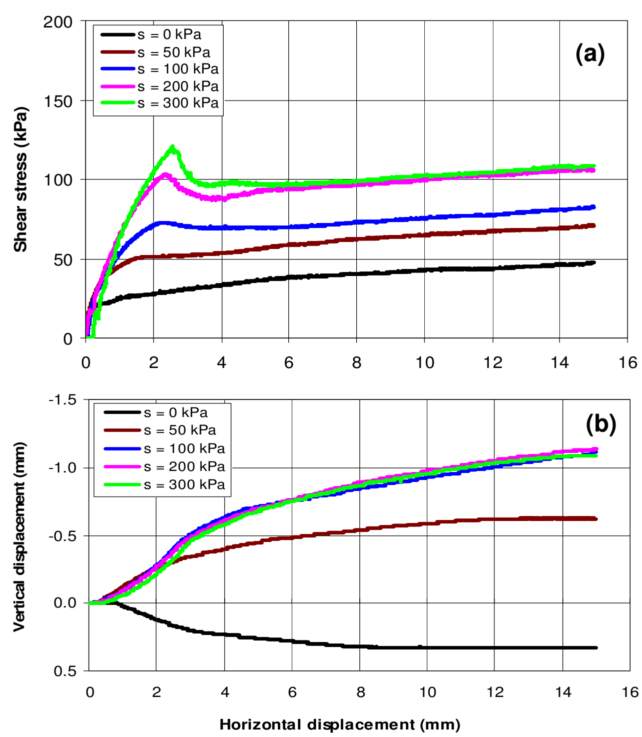

Figure 6. Stress-displacement curves of soil-soil direct shear tests under different matric suctions and 50 kPa net stress (after Hossain and Yin 2010)

The relationship between vertical displacement with horizontal displacement is shown in Fig. 6(b) and Fig. 7(b) under different suctions and net stresses of 50 and 300 kPa respectively. For 50 kPa net stress, a shear-dilation is obvious as the suction value is increased from saturated condition. The more the suction, the more the shear-dilation. In contrasts, for 300 kPa net stress, a shear-dilation is observed only for higher suctions of 200 and 300 kPa.

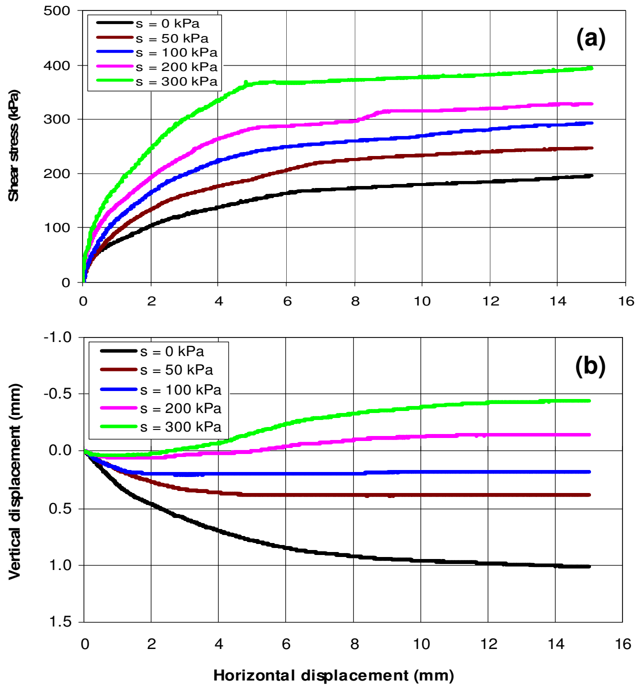

Figure 7. Stress-displacement curves of soil-soil direct shear tests under different matric suctions and 300 kPa net stress (after Hossain and Yin 2010)

The cause of increasing soil dilation with matric suction is explained by Hossain and Yin (2010). When the water content continues decreasing due to increase in suction and the soil is sheared, the soil particles may not move around each other but try to move up or over each other which cause dilation of the soil and this dilation rate depends on the rate of decreasing water content.

7.2 Soil-cement interface direct shear tests

The main test results from suction-controlled soil-cement grout interface tests are shown in Fig. 8 and Fig. 9. The relationship between interface shear stress and horizontal displacement under different suctions (0, 50, 100, 200 and 300 kPa) and net normal stresses of 50 and 300 kPa is shown in Fig. 8(a) and 9(a) respectively. Similar to soil-soil direct shear tests, the interface shear stress increases with increase of matric suction and net stress. For 50 kPa net stress, a strain-softening behavior with clear peak is observed for different suctions except saturated condition. However, a strain-softening behavior is observed within the suction range of 200 to 300 kPa for 300 kPa net stress, and a strain-hardening behavior is obvious within the suction range of 0 to 50 kPa. Comparing interface behavior with soil behavior, it can be pointed out that the

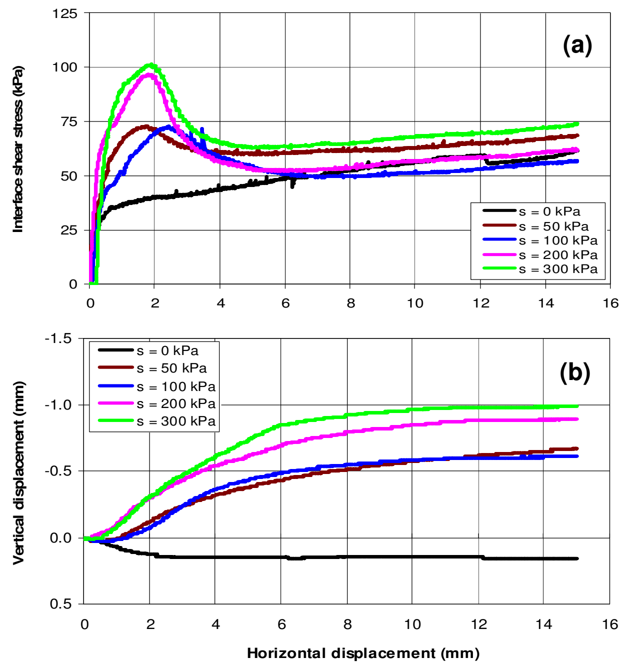

Figure 8. Stress-displacement curves of soil-cement grout interface direct shear tests under different matric suctions and 50 kPa net stress

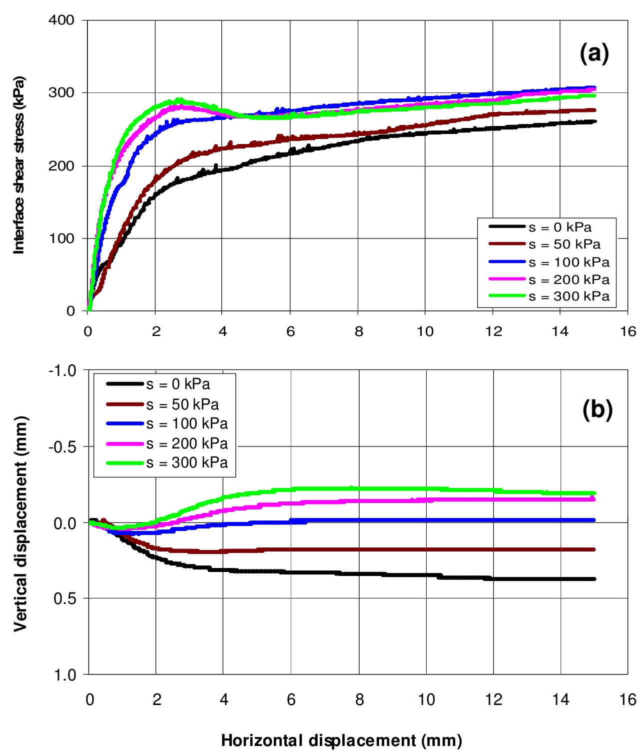

Figure 9. Stress-displacement curves of soil-cement grout interface direct shear tests under different matric suctions and 300 kPa net stress

interface failure plane becomes more compact (similar to overconsolidated soil) than that of soil-soil direct shear test because of infiltration of cement particles. Also, it can be concluded that matric suction and net stress has significant influence on the hardening-softening behavior of soil-cement grout interface.

Figure 8(b) and Fig. 9(b) describe the relationship between vertical displacement and horizontal displacement under different suction values and net stress of 50 and 300 kPa respectively. Similar to soil, for 50 kPa net stress, a shear-dilation is observed for interface as the suction value is increased from saturated condition. On the contrary, for 300 kPa net stress, no dilative behavior is observed at lower suction range (0 to 50 kPa). A little bit shear -dilation is found for 100 kPa suction, and a clear dilative behavior is observed for 200 and 300 kPa suctions. This indicates that contractive or dilative behavior of soil-cement grout interface depends on matric suction as well as net stress. However, the interface-dilation value is lower compared to soil-dilation which may be due to slippage provided by the presence of cement particles along the failure surface.

Figure 10 presents the comparison between the experimental shear strength of soil and interface shear strength of soil-cement grout under different suctions and net stresses of 50 and 300 kPa. Similar to soil-soil direct shear tests, the interface shear strength increases with matric suction and net stress and the suction envelope is nonlinear. It should be noted that the area correction for direct shear test was applied to calculate the shear stress, and the failure point is considered as the peak shear stress or at the point where the shear stress start to remain constant obtained from the raw test data.

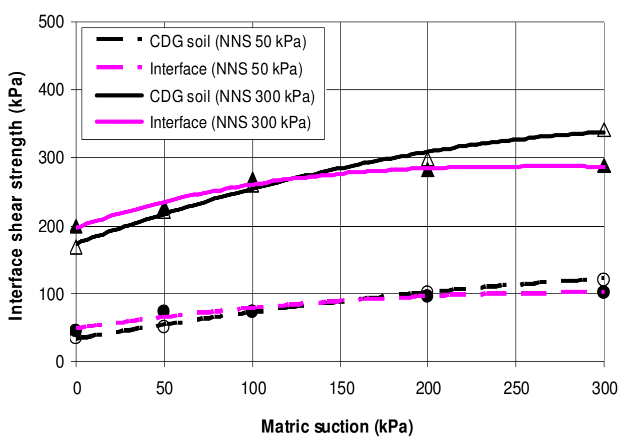

Figure 10. Comparison between interface shear strength and soil shear strength under different matric suctions and net stresses

Figure 10 indicates that the interface shear strength is greater than the shear strength of soil within the suction range of 0 to 100 kPa for lower and higher net stresses. However, the interface shear strength is lower than the shear strength of soil for the higher suction range of 200 to 300 kPa. This may be attributed to the breaking of bonding between soil and cement particles along the failure surface due to shortage of water at higher suctions. From Fig. 10, it is obvious that the breaking of bonding starts as the suction value is increased from saturated condition.

8 CONCLUSIONS

The interface direct shear test results and their interpretations are presented in this paper. The interface behavior is compared with the behavior of the same soil. The following conclusions can be drawn based on the discussion presented in the previous sections:

(a) The hardening-softening behavior of interface is significantly influenced by the matric suction and net stress. A strain-softening behavior is observed for different suctions except saturated condition for 50 kPa net stress. On the other hand, a strain-softening behavior is observed only at higher suction range of 200 to 300 kPa for 300 kPa net stress.

(b) Interface-dilation is obvious as the suction value is increased from saturated condition for 50 kPa net stress. In contrasts, Interface-dilation is observed at higher suction range of 100 to 300 kPa, and no dilation is observed at lower suctions of 0 to 50 kPa for 300 kPa net stress. The interface-dilation value is lower compared to soil-dilation.

(c) Similar to CDG soil, the interface shear strength increases with suction and net stress, and the suction envelope is nonlinear. However, the interface shear strength is higher than the soil shear strength within lower suction range of 0 to 100 kPa for both the net stresses. But, the shear strength is lower than the soil shear strength at higher suction range of 200 to 300 kPa.

9 ACKNOWLEDGEMENTS

Financial supports from The Hong Kong Polytechnic University and a grant from Research Grants Council (RGC) General Research Fund (GRF) (grant number: PolyU 5338/08E) of the Hong Kong Special Administrative Region Government of China are gratefully acknowledged.

10 REFERENCES

Acar, Y. B., Durgunoglu, H. T. and Tumay, M. T. 1982. Interface properties of sand, *Journal of Geotechnical Engineering,* ASCE, 108(4): 648-654.

ASTM D2487-90. (1992). Standard test method for classification of soils for engineering purposes. Annual Books of ASTM Standards, vol. 04.08, sec. 4, Philadelphia, 326-336.

Bishop, A. W. 1959. The principle of effective stress. Lecture delivered in Oslo, Norway, Tecknisk Ukeblad I Samarbeide Med Teknikk, 106(39): 859-863.

Bosscher, P. J. 1987. Frictional properties between sand and various construction materials, *Journal of Geotechnical Engineering,* ASCE, 113(9): 1035-1039.
Boulon, M. 1989. Basic features of soil structure interface behaviour, *Computer Geotechnics*, 7: 115-131.
Brumund, W. F. and Leonards, G. A. 1973. Experimental study of static and dynamic friction between sand and typical construction materials. *J. Test. Eval.*, JTEVA, 1(2): 162-165.
BS 1377. 1990. Methods of test for soils for civil engineering purposes. British Standards Institution (BSI). London, UK.
Chen P. Y. M. (1992): Methods of Test for Soils in Hong Kong for Civil Engineering Purposes (Phase 1 Tests), *GEO Report No.36*, Geotechnical Engineering office, Hong Kong.
Chu, L. M. and Yin, J. H. 2006. Study on soil-cement grout interface shear strength of soil nailing by direct shear box testing method, *Geomechanics and Geoengineering: An International Journal*, 1(4): 259-273.
Desai, C. S., Drumm, E. C. and Zaman, M. M. 1985. Cyclic testing and modeling of interface, *Journal of Geotechnical Engineering,* ASCE, 111(6): 793-815.
Evans, M. D. and Fennick, T. J. 1995. Geosynthetic/soil interface friction angles using a rotation shear device, *Geotechnical Testing Journal*, 18(2): 271-275.
Fakharian, K. and Evgin, E. 1996. An automated apparatus for three-dimensional monotonic and cyclic testing of interfaces, *Geotechnical Testing Journal*, 19(1): 22-31.
Ferdlund, D. G., and Morgenstern, N. R. 1977. Stress state variables for unsaturated soils, *Journal of Geotechnical Engineering,* ASCE, 103(GT5): 447-466.
Fredlund, D. G., Morgenstern, N. R. and Widger, R. A. 1978. The shear strength of unsaturated soils, *Canadian Geotechnical Journal*, 15(3): 313-321.
Fredlund, D. G. and Rahardjo, H. 1993. *Soil Mechanics for Unsaturated Soils*, Wiley, New York.
Frost, J. D. and Han, J. 1999. Behavior of interfaces between fiber-reinforced polymers and sands, *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, 125(8): 633-640.
Gan, J. K., Fredlund, D. G. and Rahardjo, H. 1988. Determination of the shear strength parameters of an unsaturated soil using the direct shear test, *Canadian Geotechnical Journal*, 25(8): 500-510.
Gan, J. K. and Fredlund, D. G. 1996. Direct shear and triaxial testing of a Hong Kong soil under saturated and unsaturated conditions, *GEO Report No. 46*, Geotechnical Engineering Office, Civil Engineering Department, Hong Kong.
Hamid, T. B. and Miller, G. A. 2009. Shear strength of unsaturated soil interfaces. *Canadian Geotechnical Journal*, 46(5): 595-606.
Hossain, M. A. and Yin, J. H. 2010. Behavior of a compacted completely decomposed granite soil from suction controlled direct shear tests, *Journal of Geotechnical and Geoenvironmental Engineering,* ASCE, 136(1): 189-198.
Jewell, R. A. and Wroth C. P. 1987. Direct shear tests on reinforced sand, *Géotechnique*, 37(1): 53-68.
Kishida, H. and Uesugi, M. 1987. Tests of the interface between sand and steel in the simple shear apparatus, *Géotechnique*, 37(1): 45-52.
Kulhawy, F. H. and Peterson, M. S. 1979. Behavior of sand-concrete interfaces, *Proceedings of the 6th Panamerican Conference on Soil mechanics and Foundation Engineering*, Lima. Peru, 7: 225-236.
Millere, G. A. and Hamid, T. B. 2007. Interface direct shear testing of unsaturated soil, *Geotechnical Testing Journal*, 30(3): 182-191.
O'Rourke, T. D., Druschel, S. J. and Netravali, A. N. 1990. Shear strength characteristics of sand-polymer interfaces, *Journal of Geotechnical Engineering,* ASCE, 116(3): 451-469.
Paikowsky, S. G., Player, C. M. and Connors, P. J. 1995. A dual interface apparatus for testing unrestricted friction of soil along solid surface. *Geotechnical Testing Journal*, 18(2): 168-193.
Potyondy, J. G. 1961. Skin friction between various soils and construction materials, *Géotechnique*, 11(4): 339-353.
Rao, K. S. S, Allam, M. M. and Robinson, R. G. 2000. Drained shear strength of fine-grained soil-solid surface interfaces, *Proceedings of Institute of Civil Engineers, Geotechnical Engineering*, 143: 75-81.
Shakir, R. R. and Zhu, J. 2009. Behavior of compacted clay-concrete interface. *Journal Frontiers of Architecture and Civil Engineering in China*, 3: 85-92.
Sharma, J. S., Fleming, I. R. and Jogi, M. B. 2007. Measurement of unsaturated soil-geomembrane interface shear-strength parameters, *Canadian Geotechnical Journal*, 44: 78-88.
Uesugi, M., Kishida, H. and Uchikawa, Y. 1990. Friction between dry sand and concrete under monotonic and repeated loading, *Soil and Foundations*, Japanese Society of Soil Mechanics and Foundation Engineering, 30(1): 115-128.
Vanapalli, S. K., Fredlund, D. G., Pufahl, D. E. and Clifton, A. W. 1996. Model for the prediction of shear strength with respect to soil suction. *Canadian Geotechnical Journal*, 33: 379-392.
Yoshimi, Y. and Kishida, T. 1981. Friction between sand and metal surface. *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, International Society of Soil Mechanics and Foundation Engineering, 1: 831-834.