

Undrained behavior and shear strength of clean sand containing low-plastic fines

Vu To-Anh Phan^{1*} Darn-Horng Hsiao²

¹Faculty of Civil Engineering, Ton Duc Thang University, Ho Chi Minh City, Vietnam

²Department of Civil Engineering, National Kaohsiung University of Applied Sciences, Taiwan

Corresponding author, E-mail address: phantoanhvu@tdt.edu.vn

Abstract. This study presents experimental tests to understand the undrained behavior of sand containing various fines contents. The specimens were prepared by the wet tamping method. The consolidated undrained triaxial shear tests were carried out by sands mixed with amounts of fines in ranging from 0 to 60%. The results showed that the deviator stress quickly reaches the peak value with an axial strain in a range of 0.5 to 2%, and then, the value drops significantly with further increases in the axial strain, the pore water pressure of all the sand-fines mixtures rapidly increases as the axial strain reaches a value in a range from 1 to 2% and then slowly increases and reaches a stable state when strain is greater than 8%. Peak deviator stress gradually decreases with an increasing fines content from 0 to 40%, thereafter, the peak deviator significantly increases with further increases in the fines content up to 60%, irrespective of confining pressure values using in these tests. Finally, the effective internal friction angles are remarkably greater than the total friction angles for various sand-fines mixtures.

1. Introduction

Natural sandy soil commonly contains sand particles and fines with different proportions. The engineering of these soils has been widely studied during the past few decades. Past studies have been demonstrated that the existence of fines significantly plays a major role on undrained behavior and on the engineering properties of sands [1, 2]. In fact, fine particles also cause to many complicated phenomena under undrained behavior, stress-strain relation, and liquefaction resistance [3, 4].

Some results showed that the liquefaction resistance either increased with increasing fines content in the mixture [5-7] or decreased with increasing fines content [8-10]. Other studies have found that the resistance of sand to liquefaction initially decreased as the fines content increased until some minimum resistance was reached and then increased as the fines content continued to increase [11-14]. Generally speaking, the existence of fines significantly affected the engineering properties as well as the behavior of a sand. The reason can be summarized, such as grain size distribution, the arrangement of fines in a mixture, the binary packing model, the plasticity index of fines, relative density, and the shear stress condition. Theoretically, the variation of void ratio with fines content is plotted in Figure 1. For 0 % fines content, the large grains produce a package with a global void ratio e_1 , as indicated on



the left side of Figure 1. Fine particles are then dropped within the void ratio among the large particles, as shown on the second inserted sketch in Figure 2. The global void ratio decreases when increasing the fines content. The minimum global void ratio e_{\min} is obtained when fine particles fill up the void ratio between large particles. As the fine particles increase to surpass the minimum global void ratio, the large particles in the primary fabric are pushed apart and no large particles exist till fine grains reach up 100% (point C in Figure 1). For the stage of BC, the global void ratio increases linearly with an increasing fines content; furthermore, large particles become to separate and do not exist at the end with 100% fines corresponding with the void ratio e_2 . Generally, e_2 is greater than e_1 .

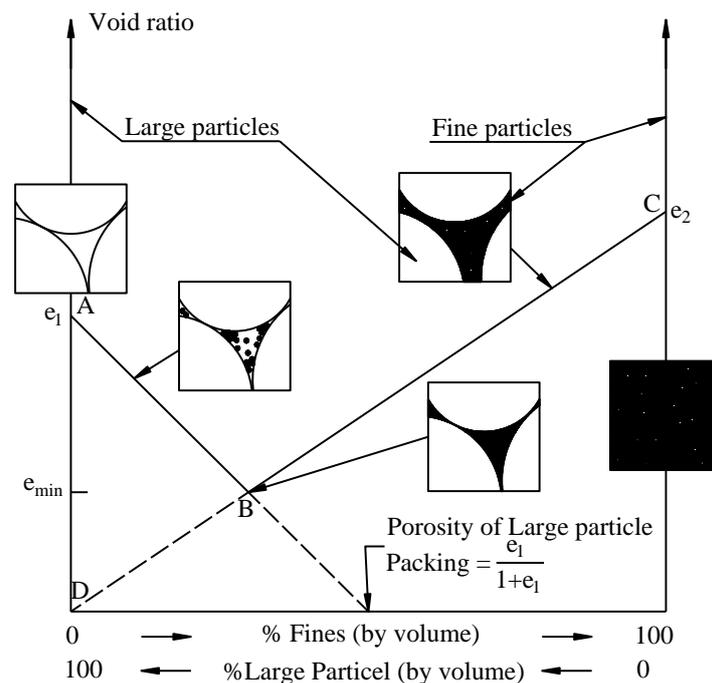


Figure 1. The global void ratio in relation to the fines content [15]

To answer the question whether the mixed soils behave sand-like or fines-like soil properties, a transitional fines content (TFC) is proposed and determined by certain studies [16, 17]. For instance, the TFC can be determined by Chang and Hong (2008) [17]:

$$TFC = \frac{e_1}{1 + e_1 + e_2} \quad (1)$$

where e_1 and e_2 are global void ratio of sand and fines, respectively.

Considering the portion of the fine particles, the active intergrain contacts are considered, TFC can be determined as follows [17]:

$$TFC = \frac{1}{(1-b)} \frac{e_1}{(1 + e_1 + e_2)} \quad (2)$$

The microstructure of a soil mixture, which can be composed of in various manners with various types of intrinsic contacts causes various undrained shear behaviors. For instance, considering a granular mixture consisting of two grain sizes, including fine particle (d) and large particle (D), which have different proportions. For this variation, Thevanayagam et al. (2002) [16] have already postulated

three types of limiting categories of microstructure, including (a) primarily the coarse particles are in contact, (b) primarily the fine particles are in contact with each other. For type (a), three subsets are including: the fines are confined within the void spaces between the coarse grains with little contribution to supporting the coarse grain skeleton and it is called as case (i); the fines are partially supporting the coarse grain skeleton, case (ii); or they partially separate the coarse grains, case (iii). In type (b), for case (iv) the coarse grains are found completely to be separated the fine grain matrix. As indicated in Figure 2, some definitions are listed as follows:

$$e_s = \frac{1+e}{1-f_c} \quad (3)$$

$$e_f = \frac{e}{f_c} \quad (4)$$

Where b is a portion of the fine grains that contributes to the active intergrain contacts; e is global void ratio; FC is fine grains content; FC_{th} is threshold fine grains content; FC_L is limit fines content; m is reinforcement factor; $R_d=D/d$ is a particle size disparity ratio

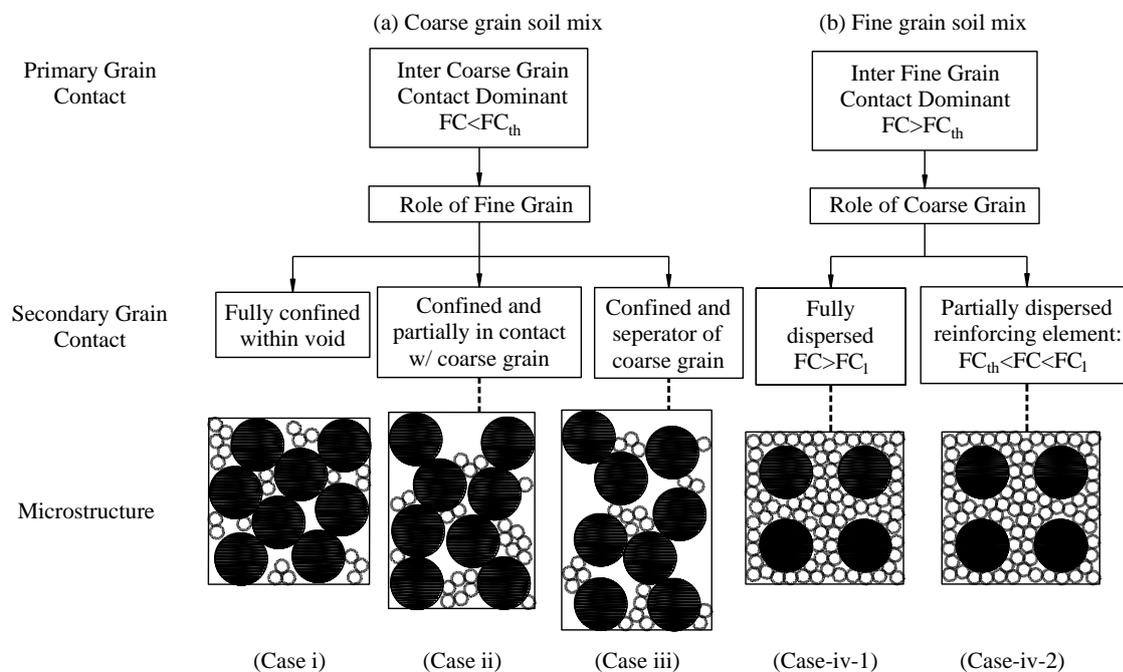


Figure 2. Framework concept of intergranular soil mix classification [16]

In this study, the test program employed triaxial shear tests on various sand-fines mixtures. The specimens were prepared at the same peak deviator stress of 290 kPa with a confining pressure of 100 kPa to understand the undrained behavior of sand containing various fines contents in ranging from 0% to 60%. The behavior of stress-strain relation, pore water pressure-strain relation, shear strength parameter (c , ϕ) were obtained and discussed. In addition, the relationship between the intergranular void ratio and the peak deviator stress, fines content, and confining pressure is also discussed. The experimental data is expected to have an insight about the behavior of sand containing various fines contents.

2. Materials tested sample preparation

Soil specimens were taken from Liouguei District, located in Kaohsiung city, Taiwan. A quantity of natural sandy soil was carefully sieved to separately obtain clean sand and pure fines. The fines particles are defined as the grain size of soil that is able to pass through a No. 200 (0.075 mm) sieve. The sand-fines mixtures were prepared from these two materials for the following combinations.

All tests were conducted with six sand-fines mixtures defined by dry weight: 100% sand plus 0% fines (0% FC), 85% sand plus 15% fines (15% FC), 70% sand plus 30% fines (30% FC), 60% sand plus 40% fines (40% FC), 50% sand plus 50% fines (50% FC), 40% sand plus 60% fines (60% FC). Based on the ASTM D422 [18] method, the specimen was carried out on specimens that had the same peak deviator stress of 290 kPa when tested in the triaxial test. To obtain the same peak deviator stress of 290 kPa, a series of trial mixes from 1 to 6 were gradually constructed with various void ratios and dry unit weights, and thereafter, each sample was tested with a consolidated drained triaxial shear test with a confining pressure of 100 kPa to obtain the peak deviator stress. Selected specimens had a peak deviator stress of 290 kPa, as explained in Hisao and Phan [19].

3. Results and discussion

A series of consolidated undrained triaxial tests were performed at an effective confining pressure of 50, 100, and 200 kPa using a strain-controlled system. The specimens were sheared in undrained compression at a constant strain rate of 0.5 mm/min. Past studies by Sladen et al. (1985) [20] concluded that there is no difference the results of either load-controlled or strain-controlled tests.

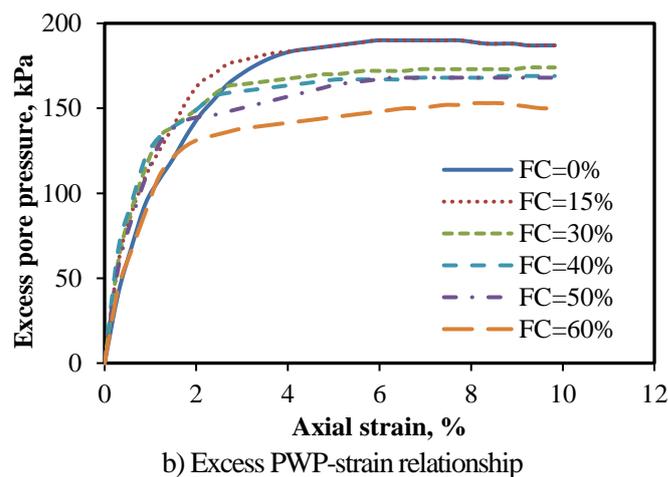
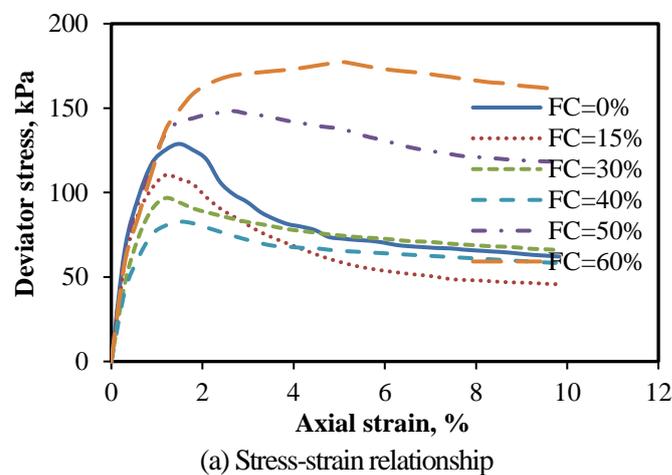


Figure 3. Undrained behaviors of sand-fines mixture ($\sigma_3 = 200$ kPa)

Figure 3 typically presents the behaviors of stress and strain, the pore water pressure (PWP) and the strain of different fines contents at confining pressure of 200 kPa. The test results from a confining pressure of 50, 100 kPa produced the same tendency as the result from confining pressures of 200 kPa and thus are not shown. In general, the deviator stress quickly reaches the peak value, with an axial strain in the range of 0.5 to 2%, and then, the value drops significantly with further increases in the axial strain.

Figure 3(a) indicates that peak deviator stresses only decrease when the fines content increases from 0 to 40%, and then, the peak deviator stresses significantly increase with further increases in fines content. The peak deviator stress can reach a maximum deviator stress at a fines content of 60%. The obtained results are somewhat different from the observations of Pitman et al. (1994) [5] who studied the various types of fines (kaolinite and crushed silica fines) and different fines contents and indicated a clear decreasing trend of a strength-sortening response with increasing fines content of up to 30% and noted that at 40% fines, there no defined peak and deviator stress continues to climb, flattening only at large strains.

Additionally, as shown in Figure 3(b), the PWP of all the sand-fines mixtures rapidly increases as the axial strain reaches a value in the range of 1 to 2% and then slowly increases and reaches a stable state with a strain greater than 8%. However, it is very difficult to distinguish the increasing tendency of the PWP among specimens contained various fines contents.

To understand the relationship between the peak deviator stress (PDS) and the fines contents, Figure 4 plots the variations of the peak deviator stress with various fines contents. In general, the PDS decreases with an increasing fines content of up to 40%. Figure 4 shows that the PDS gradually decreases with an increasing fines content from 0 to 40%, thereafter, the PDS significantly increases with further increases in the fines content of up to 60%, irrespective of confining pressure values using in this test.

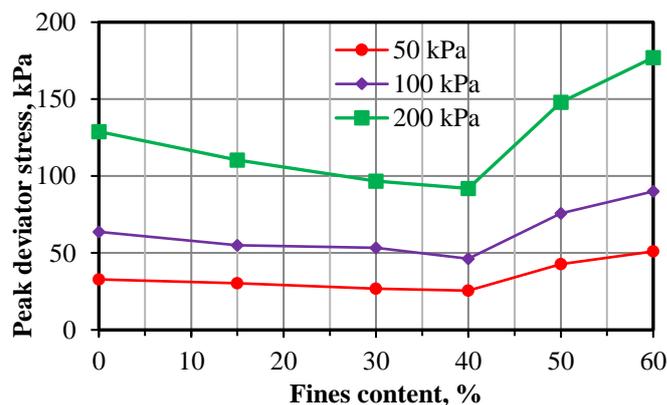


Figure 4. Peak deviator stress - fines content relation in the CU test

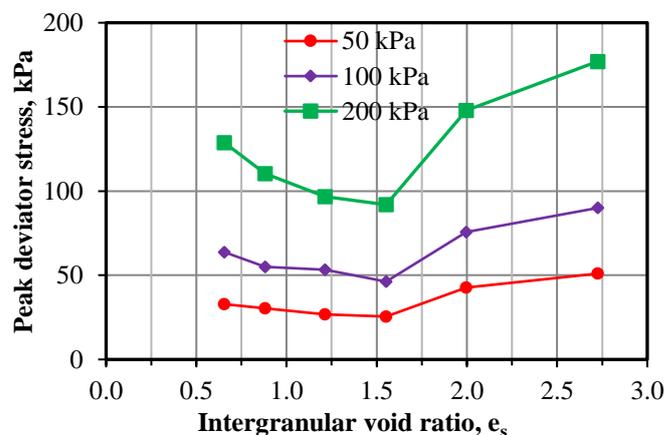


Figure 5. Peak deviator stress-intergranular void ratio relation in the CU test

In addition, the relationships between PDS and intergranular void ratio e_s of sand-fines mixtures are plotted in Figure 5. As a similar tendency presented in Figure 4, the intergranular void ratio seems to

yield the same tendency with the fines contents, regardless of applied confining pressure. Figure 5 shows that the peak deviator stress is obtained with the intergranular void ratio of 1.55.

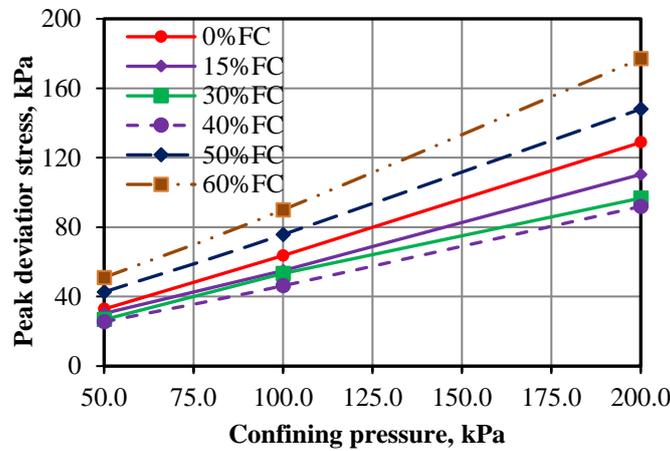


Figure 6. Various peak deviator stress and confining pressure in the CU test

Figure 6 plots the various peak deviator stresses with the confining pressures for various fines contents. In general, a higher confining pressure has a greater deviator stress, irrespective of the initial condition of the specimen. An increasing PDS with confining pressure is to be found. Moreover, the PDS line first moves downward until the fines content of 50% is reached and then PDS lines significantly moves above that of clean sand when added more fines.

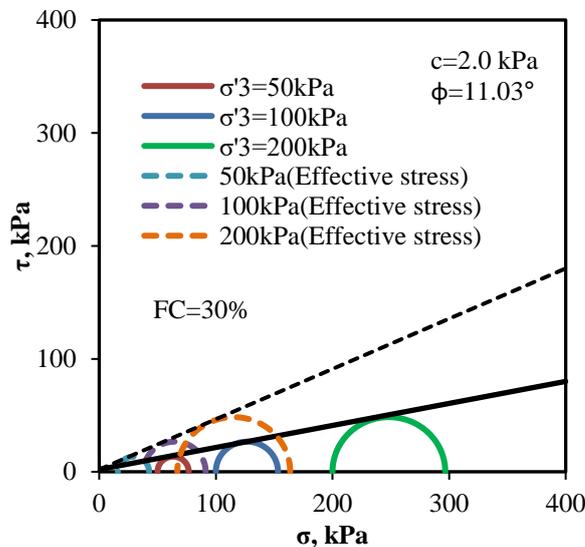


Figure 7. Mohr's circle of total and effective stress

Figure 7 presents the typical Mohr's circles of sand containing 30 % fines content. The Mohr's circles of total stress are presented by continuous lines, and the Mohr's circles of effective stress are shown by the dash lines. It is easy to recognize that the effective stress circle has the same diameter as the total stress circle and separates from it by the pore water pressure. In addition, the failure line of effective stress circle becomes steeper and causes the greater internal friction angle as comparing with its total stress circle. Finally, the results of internal friction angle and cohesion are also presented in Table 1. By comparing the results presented in Table 1, it is very obvious that the effective internal friction angles are remarkably greater than the total friction angles with different fines content. The reason can be explained by the role of excess PWP, the existence of PWP significantly causes a

decreasing tendency in friction force between grain particles. This finding is well-fitted with the principle of geotechnical engineering.

Table 1 Test results in the consolidated undrained triaxial shear test

(a) Total shear stress parameters

0% FC		15% FC		30% FC		40% FC		50% FC		60% FC	
c_u (kPa)	ϕ_u (°)										
0	14.04	1.2	12.24	2.0	11.03	2.4	10.15	2.7	15.29	3.0	17.22

c_u : total cohesion; ϕ_u : total friction angle.

(b) Effective shear stress parameters

0% FC		15% FC		30% FC		40% FC		50% FC		60% FC	
c' (kPa)	ϕ' (°)										
0	31.5	1.7	30.4	2.6	31.2	2.9	30.5	3.7	29.3	3.0	28.9

c' : effective cohesion; ϕ' : effective friction angle.

4. Conclusions

Based on the obtained data from consolidated undrained triaxial shear tests of six sand-fines mixtures. Conclusions can be drawn as follows:

- The deviator stress quickly reaches the peak value, with an axial strain in the range of 0.5 to 2%, and then, the value drops significantly with further increases in the axial strain. The PWP of all the sand-fines mixtures rapidly increases as the axial strain reaches a value in the range of 1 to 2% and then slowly increases and reaches a stable state with a strain greater than 8%.
- Peak deviator stress gradually decreases with an increasing fines content from 0 to 40%, thereafter, the PDS significantly increases with further increases in the fines content of up to 60%, irrespective of confining pressure values using in this test.
- The intergranular void ratio seems to yield the same tendency with the fines contents, regardless of applied confining pressure.
- The effective internal friction angles are remarkably greater than the total friction angles with different fines content. This finding is well-fitted with the principle of geotechnical engineering.

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