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Effect of Mixing Fine Sand on the Drained Shear Strength of Completely Decomposed Granite Soil

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ABSTRACT: Experimental test results presented in this paper were from a series of triaxial compression tests studied under drained conditions for Completely Decomposed Granite (CDG) soil mixed with fine sand content of (0, 10, 20, 30, and 40%). The CDG soil showed high compressibility during isotropic consolidation, probably due to the use of the moist tamping method and the effect of weathering degree on the soil structure. The tests results produced a unique Critical State Line (CSL) in the e-lnp′ plane, and these lines were parallel for each mixture and moved downward with increasing fine sand content. The fine sand content, at which the intergranular void ratio of the CDG-fine sand mixture became equal to e_{max} for plain CDG soil, was named as Transition Fine Sand Content (TFSC), which occurred at 20-30% fine sand content. Normalization of the critical state stresses showed that for the samples with low P′/P′c between 0.58 and 0.65 (i.e. the CDG soil mixed with fine sand), the stress paths moved directly towards the critical state without passing through the boundary surface of the soil mixture, which revealed the impact of the fine sand addition to the CDG soil structure, reflecting an improvement in the soil strength behavior by developing a strong interlocking among the particles of the mixture. It was also observed that a small portion of stress paths could pass through the boundary of Hvorslev surface in the case of low fine sand content (≤ 10 %) and the boundary of Hvorslev surface observed clearly in the case of plain CDG soil. The friction angle increased at steady state from 28°- 32.6°, and the cohesion decreased from 15 to 8.3 kN/m^2 with increasing fine sand content. A comparison of critical state parameters and strength properties between weathered granite CDG soil from Malaysia and Hong Kong were also made and summarized in this study.

Keywords: Critical state; completely decomposed granite soil; triaxial compression; transition fine sand content.

1. INTRODUCTION

Weathering, which is nature's inevitable process, gradually alters a rock from its original hard state to a soil material and as a consequence changes its engineering behavior. Most rocks are weathered to some extent, and it is recognized that this process will affect many of its engineering properties. Although rocks will progressively lose their strength, little attempts have been made to compare the index properties and strength of different weathered rocks in different weathering conditions.

Weathering materials usually result in the creation of voids due to the preferential dissolution of certain mineral phases. As the weathering process continues, and a particular mineral leaches out, the original structure of the rock is changed, producing altered grains with increasing numbers of macro and micro fractures. At the advanced stage of weathering, the remnant bonds collapse, resulting in the formation of residual soil, Bjerrum [1] , Chandler [2].

In general, a rock loses its strength and becomes more plastic and permeable with weathering; although, the extent of weathering will depend on the nature of the rock, the presence and types of weathering product, and the stage of weathering Anon [3]. The degree of weathering may be reflected by the changes in the index properties, such as density, void ratio, and silt - clay ratio content.

Previous studies were mostly concerned with the stress-strain and shear strength behavior of granular sandy and /or clayey soil. However, field observations showed that sandy soil might contain considerable amount of clay and/or silt, and vice versa if the silty or clayey soil matrix contained considerable amount of fine sand, which should be expected to affect the soils behavior. There are a few available researches on the Critical State Line (CSL) of the Completely Decomposed Granite (CDG) soil mixed with fine sand. Been and Jefferies [4] observed that the slope of the Steady State Line (SSL) for sand increased with increasing silt content (0 to 10%), and the SSL rotated clockwise around a pivot point in the e-lnp′ plane. Bouckovalas et al., [5] proposed an ideal effect of the non-plastic silt content and effective stress on the SSL of sandy soil and observed the same effect as observed by Been and Jefferies [4]. Thevanayagam [6] investigated the effects of non-plastic fines (kaolin silt and silica fines), intergranular void ratio, and initial confining stress. He also quantified their impact on the undrained shear strength of silty sand and host sand with fine content. His test results denoted that similar silty sand may be considered to behave as silt if the fines content is greater than about 30%. A silty sand or sandy silt is expected to behave as a silt at an interfine void ratio unless the silty sand or the sandy silt is very dense.

Yang et al., [7] investigated the Steady State Lines (SSL) for sand-silt mixture with various fines contents (0, 5, 10, 15, 20, 30, 50, 70, and 94%) from drained and undrained compression tests. It was observed that the location of the SSL in the e-lnp′space was different for each mixture and depended on the fines content in the mixture. The slopes of SSL are similar with the fines content that was less than the Transition Fine Content (TFC). By using the intergranular and interfine void ratios principle proposed by Thevanaragan and Martin [8], the location of the Transitional Fine Contents

(TFC) could be observed at 30% of fine content in the soil matrix.

The identification of the TFC location in soil is useful in understanding the behavior of soil, which is based on the principle of intergranular and/or interfine void ratio of plain soil. This paper presents the laboratory investigation of the geotechnical properties of four different CDG soils from Malaysia and Hong Kong.

The current study was not intended to exactly mirror the field conditions but to identify and quantify the changes in shear strength, which occurred as the fine sand contents increased in the matrix of plain CDG soil. The CSL parameters of the CDG soil mixed with the fine sand along investigation of the Transion Fine Sand Content (TFSC) range by using the intergranular void ratio concept was carried out.

2. MATERIALUSED

The CDG soil samples were obtained from a construction site in Putrajaya, Malaysia. The samples were collected from a depth of 5 to 6 m below the ground surface. The CDG soil was classified as grade V weathering soil according to Geotechnical Engineering Office (GEO) classification system [9]. A free-fall method proposed by the Japanese Society of Soil Mechanics and Foundation engineering (JSSMFE) [10] was used to ensure that the small lumps of the soil samples were consistently broken up. Following this method, the soil was oven-dried for 24 hours; then each 500 g specimen contained in a plastic bag was allowed to free-fall 30 times onto a rigid floor from a height of 1.5 m. After this treatment, soil particles larger than 4.75 mm were discarded by dry sieving (Sieve No.#4). This process was used to obtain soil specimens with similar grading and to avoid the use of excessive large soil particles for triaxial tests. The grading curve of the treated soil was obtained in accordance with (ASTM D 422-98) [11]. The grading curve suggested that the CDG soil was sand with silt mixture and similar to the coarse decomposed granite from Hong Kong (HK soil) described by Wang [12]as shown in Fig.1. The soil had an average particle size (D_{50}) of 0.97 mm with silt contents (less than 75 μm) of about 2-4%, which was obtained from dry sieving, and the silt percentage increased to 52-65% from wet sieving. The specific gravity (G_s) of the CDG soil obtained by using (ASTM D854-02) was 2.69, and the liquid and plastic limits were 61% and 34%, respectively (ASTM D 4318-00). The silt content in the CDG soil can be classified as MH according to USCS Chart, as shown in Fig.2. Compaction tests were carried out using the standard Proctor method (ASTM 698-00a), and it was found that the maximum dry density was 1.46 g/cm^3, and the optimum moisture content was 29.6%. From consolidation test (ASTM D2435-96), the compression index C_c, swelling index C_s, coefficient of consolidation C_v, and pre-consolidation pressure were found to be 0.25, 0.013, 20.64 mm^2/min, and 1.2 kg/cm^2, respectively. The geotechnical soil properties of plain CDG and /or CDG soil mixed with fine sand are presented in Table I.

The fine sand used in this study was taken from a mining sand project. It is classified as a poorly graded, medium to fine sand, having over 98 percent passing the No. 40 sieve (0.425 mm) and retained on the No. 200 sieve (0.075 mm), as shown in Fig.1. It had an average particle size (D_{50}) of 0.23 mm. It was grayish in color and the uniformity and curvature coefficients were 2.64 and 0.96, respectively. It had a specific gravity of 2.66, and its grains were sub-angular to sub-round in shape. The angle of friction obtained from consolidated drained tests under different relative densities of 30, 60, and 80% were 28°, 31°, and 37°, respectively. The maximum, minimum dry densities, the minimum and maximum void ratios of the fine sand, which were determined according to (ASTM D 4253) and (ASTM D4254), were 1.57 g/cm^3, 1.45 g/cm^3, 0.746, and 0.822, respectively. A comparison of index and strength properties of weathered granite (CDG) from Malaysia and Hong Kong are summarized in Table II.

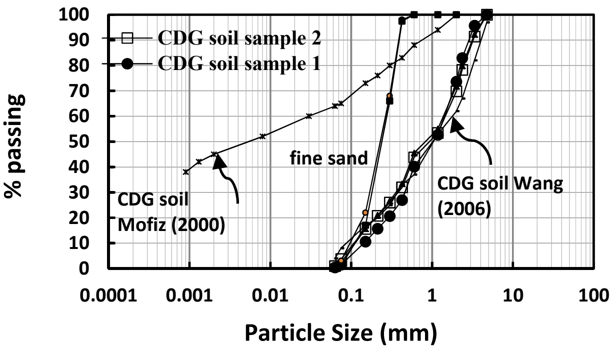

Fig.1. Grading curves of different CDG soil and fine sand samples

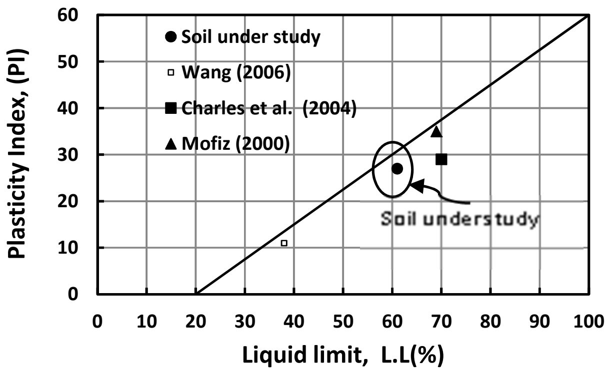

Fig.2. Plasticity chart for different CDG soils.

3. TESTING PROGRAM

Two series of triaxial compression tests were carried out to investigate the behavior of CDG soil mixed with fine sand under isotropic consolidation. The first series investigated the drained responses of medium dense plain CDG soil at confining stresses of 100 to 300 kPa.

Table I. Physical geotechnical soil properties of CDG soil mixed with fine sand

Mixing Ratio	L.L (%)	P.L (%)	P.I (%)	$\gamma_{d\,max}$ kN/m³	o.m.c (%)	$\gamma_{d\,min}$ kN/m³	ωi (%)
Plain CDG soil	61	34.0	27.0	14.3	29.6	12.2	28.5
+10% fine sand	54	31.2	22.8	15.7	26.6	12.8	25.5
+20% fine sand	52	30.3	21.7	16.0	25.0	12.95	24.0
+30% fine sand	45	27.5	17.5	16.2	23.5	13.24	21.5
+40% fine sand	40	26.5	13.5	16.5	20.0	13.64	18.5

Note: The plain soil has 0% of fine sand added to it.

Table II. Critical state parameters of different plain CDG soils from triaxial tests

properties	Mofiz [19] Dr=100%	Malaysian soil under study Dr=45%		Wang [12] HK soil Dr=45%	Charles *et al*. [20] Loose HK CDG	
	CID	CIU	CID	CID	CIU	CID
Γ	2.01	2.16	2.05	1.25	1.13	1.13
λ	0.039	0.060	0.041	0.0961	0.115	0.115
v	1.65-1.79	1.80-1.882	1.815-1.85	1.662-1.822	1.71-1.91	1.56-1.69
M=q/p′	1.11	1.107	1.1	1.54	1.58	-
e_{min}	0.823	0.80	0.82	0.556	0.71	0.56
e_{max}	0.966	0.90	0.86	1.122	0.91	0.69
c′ kN/m²	27.42	15	15	10	0	0
ϕ′ (deg)	28.02	27.9	27.7	35.2	28.1	28.1

Note: Γ, λ: represents the specific volume (v) of istropically consolidated soil at P′=1 kPa and the slope of the critical state line in the v-p′ space, respectively.

The main objective in this series is to provide a reference for interpreting results from other series of tests. The second series of CID tests were to investigate the stress-strain behavior, and the critical state interpretation for the CDG soil mixed with fine sand. The specimens after fully saturation were isotropically consolidated at different confining pressures of 100, 200, and 300 kPa (Table III) and different fine sand contents (0, 10, 20, 30, and 40%).

Table III. Test program for isotropically consolidated drained tests on CDG soil mixed with fine sand

% of Fine Sand	σ_3=100 kN/m²	σ_3=200 kN/m²	σ_3=300 kN/m²
Plain CDG soil	PCID100	PCID200	PCID300
+10%	10CID100	10CID200	10CID300
+20%	20CID100	20CID200	20CID300
+30%	30CID100	30CID200	30CID300
+40%	40CID100	40CID200	40CID300

Note: PCID100, 40CID300: soils are isotropically consolidated (i.e consolidated drained test) under 100 kPa and 300 kPa confining stress with 0% and 40% fine sand content respectively.

4. SPECIMEN PREPARATION FOR TRI-AXIAL TEST

The specimens of 50.44 mm in diameter and 100.13 mm in average height were prepared by using the moist-tamping technique, which was used for testing a triaxial test setup that was fully controlled by computer using GDS-system [13]. A predetermined amount of oven dried CDG soil and fine sand were well-mixed with deaired water for about 30 minutes to achieve sample uniformity. The soil was then sealed inside plastic bags for 24 hours to establish its equilibrium conditions. In order to achieve the same initial structure, all the specimens were prepared at initial water content, as shown in Table I. The soil specimens were divided into five layers, and each layer was determined by the weight and relative density of a similar volume of the soil. To eliminate the consolidation effects of the soil layer, the relative density for each layer was different. The method proposed by Chien and Oh [14] was used to determine the relative density of soil specimen in which from the top to the bottom of the soil sample, the relative density of each layer was Dr+2, Dr+1, Dr (45%), Dr-1, and Dr-2. Each specimen was compacted in five layers inside split mold, and each soil layer was given 20 blows using a steel tamping rod, which was 40 mm in diameter and dropped from a height of 5 cm. Before the specimen was set up in the tri-axial apparatus, all the tubes were flushed with deaired water. A porous stone that had been soaked overnight was slid over a layer of water on the base pedestal without trapping any air. A filter paper and the specimen were then placed on the porous stone.

The weight of water added to each layer was determined by the water content required to achieve 80 to 85 percent saturation in the layer being tamped (i.e. preliminary saturation). This percentage of preliminary saturation was chosen because it allowed for the preparation of specimens over a greater range of densities, and the samples were easy to saturate under a low back pressure in the range of (45 to 50 kPa). Some specimens were cut into five sections to check the uniformity of density in the height of 100.13 mm, and it was observed that the maximum density variation was no more than 3%.

The tamping method does not mimic the deposition processes of natural deposits, but it offers several advantages over either pluviation or vibration methods. Moist tamping eliminates the problems of particle segregation associated with pluviation through either air or water, and it is capable of producing specimens with consistent void ratios over a fairly wide range of densities. These factors make it ideal for a parametric study such as this in which the two main factors being evaluated are fines content and density.

After each specimen was set up in the cell, the de-aired water was first flushed through the top of the specimen, and then a back pressure of controlled average value of 45 kPa was applied for saturation. The water volume that was required to saturate the soil, which was measured from the computer software, was used to calculate the degree of saturation at the end of saturation stage. Skempton's B-values greater than 0.95 were obtained for all the specimens after saturation and B-check stages were achieved. The specimen was consolidated istropically under the desired effective radial stress of 100, 200, and 300 kPa, as mentioned in the testing program (Table III). During consolidation, the volume change was measured in order to obtain the initial void ratio for the condition from which the shearing stage started. After consolidation at the various mean effective stresses, CID shearing stage test was carried out at an axial strain of low rate 0.05 % per min, to allow full dissipation of pore water during drained shearing stage.

After each test, the final void ratio of the entire specimen was obtained by weighting the soil specimen (after drying in the oven for 24 hours) to determine the water content, and later the water content and the void ratio of the tested specimen were calculated. The intermediate void ratios during consolidation were determined by using measured volume of water expelled from the specimen during the consolidation stage. All these volumes of water conducted from Advanced Volume-Pressure Controller (ADVPC) details were described by Menzies [15]. Errors in total volume measurement due to membrane penetration during the consolidation stage were evaluated using the method suggested by Baldi and Nova [16]. The maximum possible error in total volumetric strain was about 0.1%, and the average error was found to be about 0.05%, which may have some effects on the location of the instability line in p′-q space. However, the error was considered to be insignificant for the determination of the critical state line since ε_v measured at the critical state was in the order of 2% or less, and the specimens were prepared in medium dense state.

5. RESULTS AND DISCUSSIONS

5.1 Isotropically compression tests for plain CDG soil

Tests results for isotropical compression of plain CDG soil were presented in Fig. 3. The q-ε_a relationships show strain hardening behavior for CID tests; till the soil reached the ultimate or critical states where they continued to distort at a constant deviatoric state Fig. 3 (a) and (b). The slope of the critical line (M) obtained from the drained tests was 1.112. This behavior was consistent with the data reported by Taha et al., [17] and Wang [12]. The cohesion was 15 kPa, and the internal friction angles were 28° for the CID tests. A study on Malaysian granite soil revealed a similar angle of friction but higher cohesion (Ting and Ooi [18]). The critical state parameters for different CDG soils are presented in Table II. Fig. 3(b) shows the axial - volumetric strains relationship and similar behavior was observed in the study of Mofiz [19] and Wang [12]. It can be concluded that the failure stresses and the volume changes are stress-dependent of confining stress. As shown in Fig. 3 (a) and (b), this is consistent with the study by Charles et al., [20] for the loose decomposed granite.

5.2 Isotropically consolidated drained tests for the CDG soil mixed with fine sand

The deviatoric stress-axial strain curves are presented in Fig. 4. The CDG soil mixed with fine sand was observed to be highly contractive, and the volumetric strains were generally very similar for the three confining stresses. The high level of volumetric contractions occurred at low confining stress were the characteristics of decomposition and weathering effects on the granite parent rock. These volumetric strains are shown in Fig. 5.

The volumetric strains were reduced by 7.7, 30.0, 38.8, and 46.1 % from the volumetric strains of the plain CDG soil with 10, 20, 30, and 40 % fine sand content, respectively. The volumetric strain of the specimens approached maximum volumetric contraction at an axial strain level between 20 and 25% for the plain CDG while these strains started early when fine sand was added to the CDG soil.

The q-p′ plots relationship are shown in Fig. 6 and the impact of the fine sand addition to the CDG soil structure, reflecting an improvement in the soil strength (q) behavior by developing a strong interlocking among the particles of the mixture. The stress ratios measured at 20 to 25% axial strain were found to be 1.112 to 1.314 for the CID tests.

5.3 Intergranular contact friction particles interpretation

The fine sand added to the plain CDG occupied the voids, causing a reduction in the void ratio with increasing in sand percentage (Fig. 7). It was found that the maximum and minimum void ratios of the CDG soils mixed with fine sand decreased as the fine sand content increased from 0 to 40%. Similar results were observed by Lade and Yamauro [21] for Nevada and Ottawa sand mixed with non-plastic silt. The extra percentage of fine sand might end up between the surfaces of adjacent fine sand particles; such particles would tend to cause an increase in void ratio, as they do not occupy any void in the CDG soil matrix.

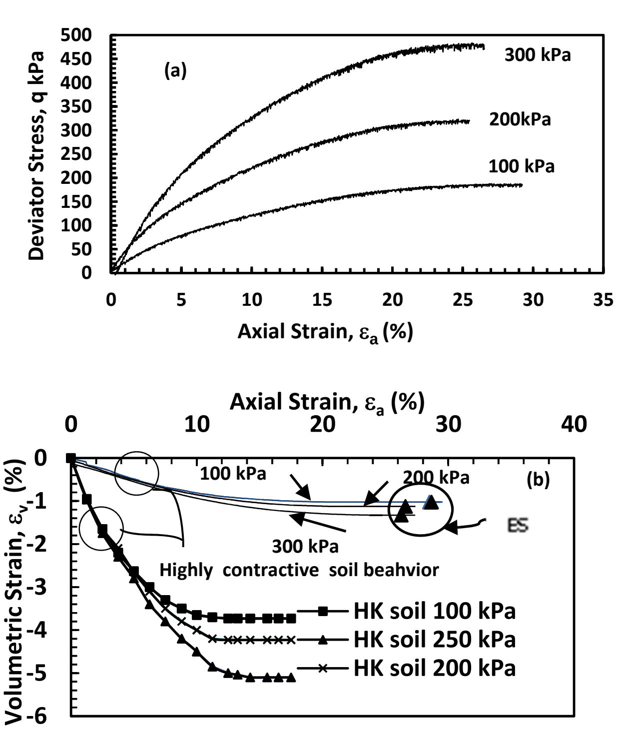

Fig. 3 Results of triaxial tests on plain CDG soil: (a) CID test results, and (b) (ε_a vs. ε_v) for CID test.

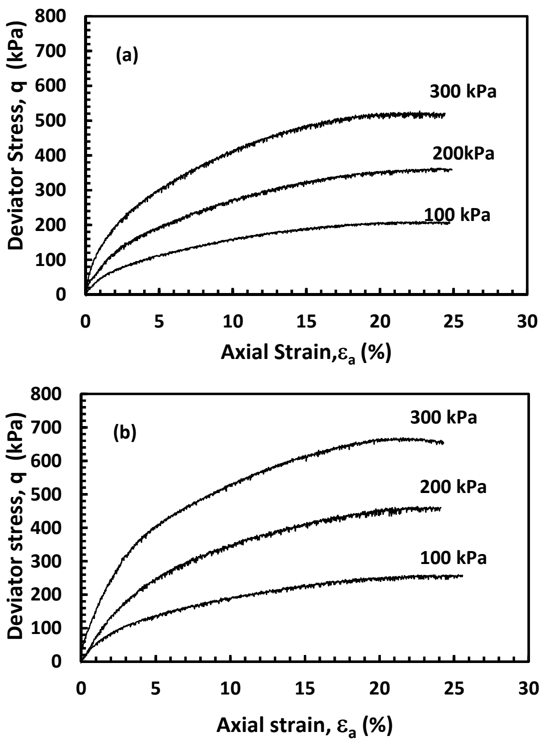

Fig. 4. Deviatoric stress vs. axial strain curves for CID tests: (a) with 10% fine sand, (b) with 40% fine sand.

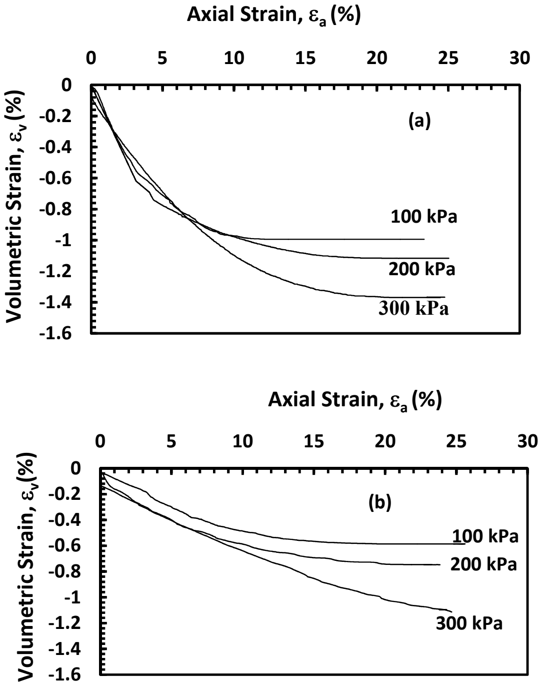

Fig. 5. Volumetric strain vs axial strain CID test: (a) with 10% fine sand, (b) with 40% fine sand.

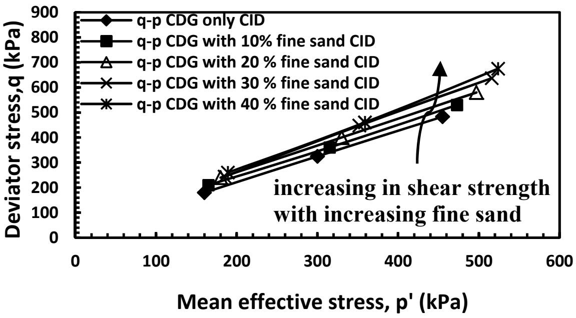

Fig. 6. Comparison between drained strength envelopes for CDG soil with different fine sand content of CID test.

The shear strength of mixture will be governed mainly by friction resistance (i.e. fine sand) and the cohesion (i.e. soil plasticity) of silt encountered in the CDG soil matrix.

For a given overall void ratio, there is a fine sand content where the fine sand separates the adjacent CDG soil particles. Fig. 8 shows the intergranular void ratios (e_{int}) as a function of void ratio for 10, 20, 30, and 40% fine sand content for CID tests by using the method proposed by Kuerbis et al., [22] to determine the intergranular void ratios (Equation1).

$$e_{int} = \frac{1+e}{1-fc} - 1 \qquad (1)$$

Where e = the overall void ratios of the CDG-fine sand mixture; and fc = ratio of weight of fine sand to the total weight of the CDG soil solids. It was found that whenever (e_{int}) is greater than e_{max} of (CDG) (i.e. plain CDG soil, fc =0%), the CDG soil matrix exists with a void ratio higher than this could be achieved in the absence of fine sand, which implied that the CDG particles were, on average not in contact, and the mechanical behavior was no longer controlled by the CDG soil matrix but by both CDG and fine sand particles, as indicated by Fig.8.

Therefore, the Transition Fine Sand Content (TFSC) in the CDG soil occurred at 20 to 30% of fine sand content. Thevanayagam and Martin [8] suggested different contributions of the fines on the sand grains to the strength of the soil matrix with various fines content, and proposed that transition fine content TFC existed when sand-dominated behavior passed to fines-dominated behavior (i.e fc > TFC).

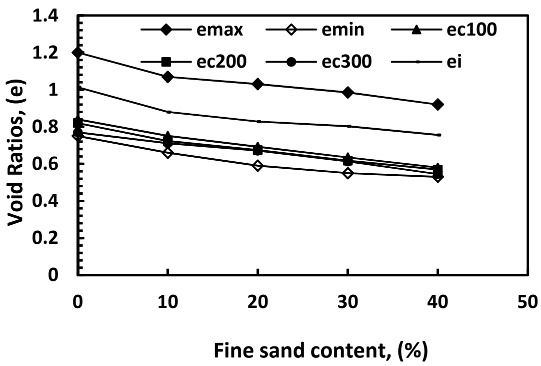

Fig. 7. Void ratio for samples of different fine sand content for: CID tests, Note:- e_{max}= maximum void ratio, e_i= initial void ratio, ec_{100}, ec_{200}, and ec_{300} represents void ratio after consolidation at confining pressure of 100, 200, and 300 kPa respectively, e_{min} = minimum void ratio.

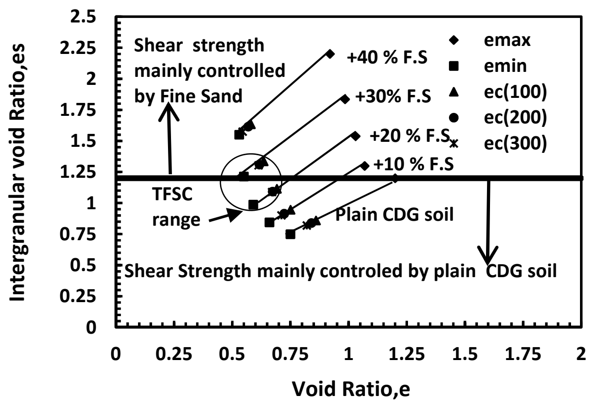

Fig. 8. Limit void ratio for CID tests for: Plain CDG soil , +10 % fine sand , +20% fine sand, +30% fine sand, and +40% fine sand under undrained conditions.

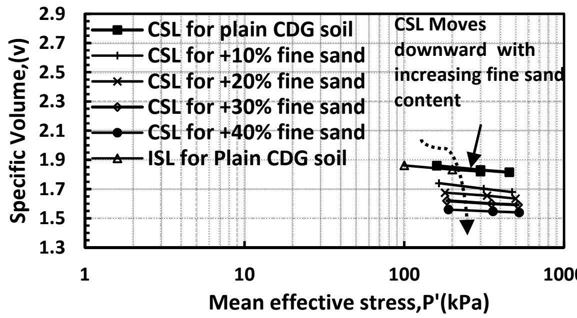

Fig. 9. Steady state lines for CDG soil mixed with different fine sand content for CID test.

5.4 Critical state interpretation and factors controlling CSL

The other objectives of this study were to examine the CSL of CDG - fines mixture, and to determine the effect of fines on the CSL line of CDG soil. The critical state (CS) is identified as a constant shear stress and volume change with increasing shear strain. The critical-state friction angles (ϕ_{crit}) were obtained from those tests. The volumetric strain versus axial strain was used to check if the CS was reached. The ϕ_{crit} was determined at the axial strain at which the volumetric strain versus axial strain plot becomes horizontal (i.e. the dilatancy angle becomes zero $(\Delta\varepsilon_v / \Delta\varepsilon_a = 0)_{crit}$, at the points "ES" represents the end of shearing stage (Fig. 3 (b)). The same points can be observed in Fig. 5 (a) and (b). The whole sample states at these points were analogous to that the converging between these lines, indicating the high contractive behavior of CDG soil, and a very thin cracks on the sample surface was observed in the case of higher fine sand percentage (i.e. fc>30%).

The slopes of CSL lines of the CDG-sand mixture are controlled by many factors, such as the plasticity of the CDG-sand structure. Fig. 9 shows the steady state-lines for the CDG soil mixed with fines ranging from 0 to 40 %. It is observed that the CSL moves downward in the e-lnp′ plane and parallel for each mixture, and similar behavior was observed by Zlatovic and Ishihara [23], Naeini and Baziar [24], and Yang et al., [7]. The improvements in the shear strength due to inclusion of fines in the CDG soil are shown in Fig. 6. The slope of CSL (M) obtained from q-p′ plane was increased for CID tests with increasing fines content. This reflected the frictional behavior of sand particles in the structure of the CDG soil, where the friction angle increased from plain soil condition of 28° to 32.6° for the CDG soil mixed with fine sand, and the cohesion reduced from 15 to -8.3 kPa]. To examine the effect of the initial confining stress on the state parameters (Fig. 9), the converging between the ISL line (i.e. initial state line) and the CSL line of the CDG (i.e. ISL below CSL, denser than critical with negative state parameter) indicated a decrease in the void ratio with increasing confining stress for the specific fine sand content. This reflected the contractive tendency of the CDG soil, that each material has a unique line.Consequently, the ISL for all CDG mixed with the fine sand lies below its CSL.

5.6 Critical state normalization

By using the method proposed by Sladen and Oswell [25] to normalize the deviatoric stress, as q / (M*P′c) and the mean effective stress P′, defined as P′/P′c (i.e. P′c represents the mean effective stress at critical state), it followed that all normalized stress path end at the (1,1) point which represented the critical state of the CDG soil.

The normalized stresses for plain CDG gave consistent contractive behavior (Fig. 10). The samples with higher P′/P′c were between 0.65 to 0.68 for the CID tests.

The stress paths reached a unique boundary surface of the Hvorslev surface, and were followed to reach the critical state. While for the samples with low P′/P′c between 0.58 and 0.65 (i.e. the CDG soil mixed with fine sand), the stress paths tended to move directly towards the critical state without passing the boundary surface of the soil mixture (Fig. 11 (b)).

This showed that when the fine sand was added to CDG soil, the soil behavior was improved due to the production of strong interlocking between the particles of mixture, with a small portion of stress paths passed through the boundary surface found in the case of low fine sand content ≤ 10 % (Fig. 11 (a)).

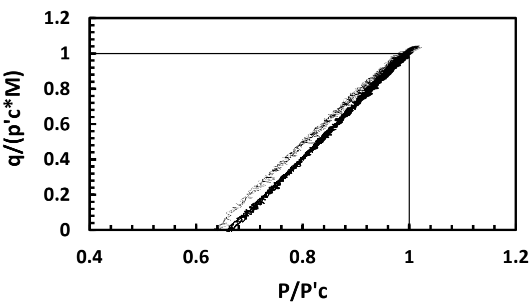

Fig.10. Critical state normalizing for the plain CDG soil under drained conditions.

6. CONCLUSIONS

This study investigated the critical state interpretation and the transition fine sand content (TFSC) of the CGD soil mixed with fine sand. The results indicated that the mechanical behavior of soil mixture was dependent on the inter-granular contact density. In addition, at low fine sand content (i.e. 10%) (*fc* < TFSC), the dominant mechanisms that affected the shear response of the CDG soil mixed with fine sand were mainly controlled by the interfine contact density between the silt particles in the CDG soil and the friction of low fine sand content.

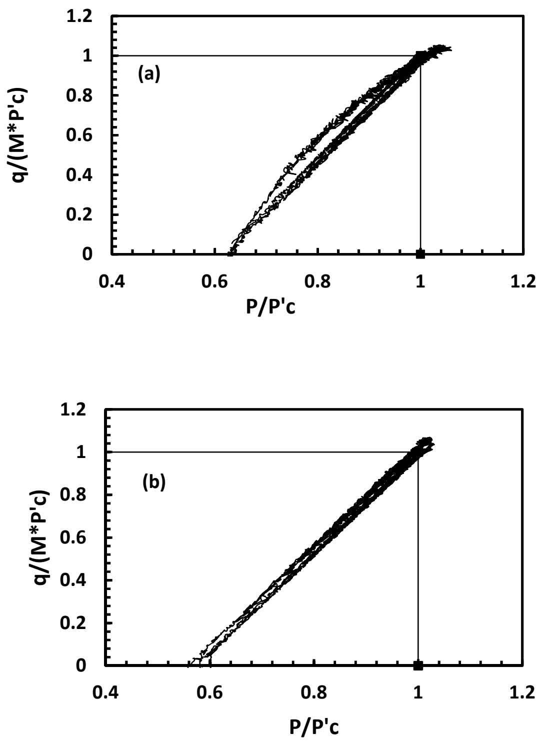

Fig.11. Critical state normalizing for CID tests: (a) with 10 % fine sand, (b with 40 % fine sand.

In contrary, at high fine sand content (*fc*>20%), the dominant mechanisms were mainly controlled by the friction of fine sand and secondly by silt particles in the CDG soil.

The triaxial test revealed that the critical state strengths increased with the increasing fine sand content. The possible explanation would be the fine sand modified the strength and provide interlocking between the soil mixtures. As the shear stage progressed, the fine sand reached more stable arrangements, and ultimately increased the interlocking. Furthermore, the shear strength parameters, ϕ increased from 28° to 32.6°, and the cohesion intercept reduced from 15 to 8.3 kPa. The silt:sand ratio (50:50) concept was insufficient for real understanding of soil behavior in any soil mixture. It was found that the TFSC occurred at 20 to 30 % over the fine sand content obtained from the grading curve of the sand-silt mixture in the plain CDG soil. This concept is shown to be dependent on the initial condition of the test specimen before shearing, and this finding in consistent with the previous studies.

The CSL lines of the CDG soil mixed with fine sand determined from the CID tests indicated their locations in the e-lnp′ plane which dependent on the fine sand content, and it moved downward when fine sand content increased. In the plots of q-p′ plane, the CSL lines showed a unique line for each mixture. This indicated improvements in the shear strength (i.e. increasing deviator stress) with increasing fine sand content.

7. SYMBOLS AND ACRONYM

Below list contains the definitions of symbols and acronym that can be traced in the study

PI=Plasticity index (%)
L.L=Liquid limit (%)
P.L=Plastic limit (%)
γ_{max} = Maximum soil unit weight(kN/m^3)
γ_{min} = Minimum soil unit weight(kN/m^3)
o.m.c=Optimum moisture content (%)
w_i=initial water content (%)
σ = Minor principal confining stress (kN/m^2)
D_r = Relative density (%)
Γ = Intersection of the CSL with p' = 1 kPa line in the ν –ln p' space
λ = Slope of the critical state line in the ν -lnp' space
ν = Specific volume
e_{max} = Maximum void ratio
e_{min} = Minimum void ratio
e_{int}= Intergranular void ratio
ec= Void ratio after consolidation
C′=Effective cohesion of soil (kN/m^2)
ϕ'= Effective friction angle (degree)
M = Slope of the CSL in the q–p' space
p' = Mean effective stress[Cambridge]
p'c = Mean effective stress at critical state
q = Deviatoric stress[Cambridge]
ε_a= Axial strain (%)
ε_v = Volumetric strain (%)
ADVDPC = Advanced digital pressure controller
ASTM = American standard for testing material
CSL = Critical state line
CDG=Completely decomposed granite
GDS = Geotechnical digital systems instruments Ltd
GEO= Geotechnical engineering office
ISL=Initial state line
TFSC= Transition fine sand content

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