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Effect of Sand Gradation on The Behavior of Sand-Clay Mixtures

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ABSTRACT: In contrast to the clean sands the behavior of sands with fines have complexities that has made it difficult for researchers to reach a general framework for their evaluation and this problem exist today for researchers. In this research work the effect of increasing clay content and also of the gradation of sand on the behavior and shear strength properties of over-consolidated mixtures of sand-clay was evaluated. one of the parameters of over-consolidated soils is the parameter m from the relationship of shear strength for over-consolidated clays. The compacted over-consolidated samples of mixtures of various sand gradations with 15, 20, 30 and 40 percent clay were tested in direct shear. the result showed that at a particular sand gradation, with the increase in clay content the shear strength and the value of m decreased. also at constant clay content with decrease in sand grains size the shear strength and vale of parameter m decreased.

Keywords: Sand-clay mixtures, Over consolidation, Shear strength, Ductile, Dilation

1. INTRODUCTION

The mechanical behavior of clean sands was investigated first by Coulomb in the 18th century [1]. Studies of the mechanical behavior of pure clays were reported only approximately 150 years later [2]. Studies of these soils continued over the years as clean sands and pure clays define distinct boundaries of a wide spectrum of natural soils and thus set limits on expected performance. Most of the studies concerning the stress–strain and shear strength behavior of granular soils mainly inspected the response of clean sands. However, field observations show that granular soils may contain a considerable amount of clay and/or silt. Therefore, these fines should be expected to influence the engineering behavior of sandy soils.

In the guidelines of earth retaining structures, soils with fines are disqualified as backfill materials. For example, according to the AASHTO (American Association of State Highway and Transportation Officials) specifications, the content of fines used in a reinforced soil retaining wall must be less than 15%. However, geotechnical engineers usually face practical concerns, like the availability of good quality backfill materials and the construction costs in meeting these criteria.

2. LITERATURE REVIEW

Wasti and Alyanak [2] have worked on sand-clay mixtures and stated that when clay content is just enough to fill the voids of the granular portion at its maximum porosity, the structure of the mixture changes and the linear relationship between the Atterberg limits (plastic and liquid limits) and the clay content is no more valid and soil changed its behavior from sand to clay. For mixture including kaolin clay at its liquid limit, they showed that this threshold value is about 25% kaolin content. Novais - Ferreira [3] performed consolidated-drained direct shear tests on artificial mixtures with increasing proportions of clay, including two types of sand (fine and coarse) and a montmorillonitic clay. They found that maximum and limiting shear stresses showed a tendency to decrease as the clay content increased. They also described the existence of three zones of behavior of the mixtures as a function of clay content (CF): 1) Noncohesive behavior ($CF \leq 28\%$) where the cohesion is negligible and the angle of friction is high (above 30^{o}) the effects of fluctuations in soil grain size variations are not significant. 2) ($28 < CF < 41$) where the soil is sensitive to grain size fluctuations. 3) Cohesive behavior ($CF \geq 41$) where the cohesion is high and the angle of friction is low.

Georgiannou [4], made an investigation on the behavior of clayey sands under monotonic and cyclic loading. He concluded that the fine content has a remarkable influence on the stress-strain response of the soil mass. As the fines content increases, the dilatant behavior of the soils is suppressed, and the response gradually becomes controlled by the fine matrix at about 40% fine content. Georgiannou, et al. [5] performed an experimental study about stress-strain behavior of anisotropically consolidated clayey sands using computer controlled triaxial cells. The specimens were prepared by sedimenting Ham River sand into a kaolin suspension. They observed the effects of variations in clay content and initial granular void ratio. They concluded that this method as compared with the same sand that is sedimented through clean water (i.e. contains no clay) creates a material which is markedly less stable, has a higher granular void ratio and exhibits a higher undrained brittleness behaviour. This is the engineering characteristic for ductile behaviour and it is determined by stress history, formative history, microstructure, rate of shearing, composition and fabric of clays.. Moreover they showed that a sand with 30% clay fraction in normally consolidated state is no longer dilatant and exhibits the response that would be expected in a sedimented clay. They also stated that for clay fractions up to 20%, the clay does not significantly reduce the angle of shearing resistance of the granular component.

Georgiannou etal. [6] have described the undrained behavior of natural clayey sand from the site of the Gulfoks C oil production platform in the North Sea. The behavior of a model soil formed from Ham River sand and kaolin was observed. This model soil was selected in order to display relatively closer response of the soil at the field. These reconstituted specimens have been subjected to Ko consolidation and undrained shear in the triaxial compression test under displacement control. They concluded that undrained brittleness in compression increases as the clay content increases from 4.5% to 11.5%, but reduces as the overconsolidation ratio, OCR, increases. They also showed that the clayey sand reaches its peak resistance at small axial strains: ε_a in compression increases from 0.1% to 0.3% as OCR increases from 1 to 2. Pitman, etal. [7] have carried out a study to investigate the influence of fines and gradation on the behavior of loosely prepared sand samples. Loose sand samples, formed by moist tamping and consolidated to the same effective stress level, were prepared with varying percentages of both plastic and nonplastic fines. Samples were isotropically consolidated and subjected to monotonic undrained triaxial compression. They stated that undrained brittleness decreased as the fines content, for both plastic and nonplastic type, increased. They also concluded that the undrained brittleness may not be controlled by the plasticity of the fines but more by the amount of fines (<74µm), at least for percentages greater than 10%. Bayoğlu [8], made an experimental study to study the effects of fine particles on the shear strength and compressibility properties of the soil mixtures. Soil mixtures having wide range of grain size from sand to silt-clay mixtures were studied. Direct shear and consolidated-undrained triaxial tests were performed on normally consolidated clay-sand mixtures to obtain strength and compressibility parameters. According to the results of drained direct shear tests on mixtures containing 5%, 15%, 35%, 50%, 75%, and 100% fines, the internal friction angles varied between 30-38 degrees until 50% fines and a slight decrease existed in the friction angle with increasing fine content. At fine contents higher than 50%, the reduction in the friction angle was significant and decreased to about 10 degrees. According to the results of consolidated-undrained triaxial tests on samples with 35%, 50%, 75%, and 100% fines, there was no relation between undrained friction angle and percentage of fines and the measured angle of shearing resistances were in the same order of magnitude irrespective of fine content.

S. Thevanayagam [9] carried out a series of experiments to obtain large strain undrained shear strength (s_{us}) in triaxial compression for particular host sand mixed with different amounts of nonplastic fines. Results indicated that the intergranular void ratio, e_s, which is the void of the sand-grain-matrix (given by $(e + f_c) / (1 - f_c)$, where f_c is the silt content fraction by weight plays an important role on s_{us} of silty sands. At the same void ratio, e, a silty sand shows low s_{us} compared to that of the host sand. However, when compared at the same e_s, provided that it is less than the maximum void ratio of the host sand, $e_{max,HS}$, both the silty sand and the host sand show similar s_{us} that is fairly independent of the initial confining stress. When e_s of the silty sand is in the vicinity of or exceeds $e_{max,HS}$, the s_{us} depends on the initial effective confining stress. At such "loose" states, initial consolidation stress is very low, s_{us} decreases with a further increase in e_s. At a fine content greater than about 30%, a silty sand is expected to behave as a silt at an interfine void ratio, e_f, defined as the void ratio of the silt-matrix (given by e/f_c), unless the silty sand or sandy silt is very dense. Salgado [10], made an experimental investigation about the effects of nonplastic fines on the shear strength of sands. A series of laboratory tests was performed on samples of Ottawa sand with fines content in the range of 5-20% by weight. He used triaxial tests that were conducted to axial strains in excess of 30%. He used the concept of the skeleton void ratio e_{sk} [11], which is the void ratio of the silty sand and calculated as if the fines were voids.

$$e_{sk} = \frac{1+e}{1-f} - 1$$

Where, e = overall void ratio of soil, f = ratio of weight of fines to total weight of solids. Whenever e_{sk} is greater than the maximum void ratio $(e_{max})_{f=0}$ of clean sands, the sand particles are not in contact and mechanical behavior is no longer controlled by the sand matrix. They suggested that silty sand with nonfloating fabric in the 5-20 % silt content range is more dilatant than clean sands; dilatancy appears to peak around 5 % silt content, but even at 20 % silt content it remains above that of clean sand. Mehmat salih olmez [11] performed three series of tests to observe the effect of kaolin in sand-clay mixtures on the shear strength behavior of mixtures. Undrained and drained triaxial tests and also direct shear tests were performed. He concluded that the shear strength properties and the stress-strain characteristics of mixtures of sand-kaolin showed a significant change at kaolin content of 20%. Pakbaz etal. [12] performed direct shear tests on natural overconsolidated clay samples as well as overconsolidated compacted mixtures of sand and bentonite. They concluded that the parameter m and the friction angle decreased with the increase in bentonite content. They also concluded that with with decrease in sand grain size with a constant bentonite content of 20% the shear strength increased and value of m decreased.

3. PROCEDURE AND MATERIALS

The soil mixtures were obtained by blending clayey silt and sandy soil mechanically. The clayey silt soil used here was natural clay and the sand was available commercially. Clean sand with three different grain size distributions has been mixed with 15, 20, 30 and 40 % natural clay to form different sand-clay soil mixtures. The index properties of clay, sand and soil mixtures are given in Tables 1 to 3. The grain size distributions of several different types of mixture, obtained by sieve and hydrometer analyses, are shown in Figures 1 to 3 The results of Atterberg limits tests showed that the liquid limit and plastic limit of the soil mixture was low respectively (plastic index of clay is 12). According to USCS (Unified Soil Classification System), mixtures were classified as SC and CL. Based on AASHTO; the amount of fines for a reinforced soil retaining wall must be less than 15% with PI less than 6%, whereas the amount of fines for reinforced slopes must be less than 50% with PI less than 20. Obviously, the blended soil did not meet the requirements of AASHTO in the construction of reinforced retaining walls and reinforced slopes. Standard Proctor compaction tests were conducted to find out the maximum dry unit weight and optimum water content of the soil mixtures. Based on the test results, the maximum dry unit weight $\gamma_{d,max}$ and the optimum water content W_{opt} were obtained and are given in the Tables 4 to 6. The purpose of making this study was to observe the effects of fraction of fine and sand gradation on the Mohr-Coulomb failure envelope and the parameter of m. the compacted samples of different sand-clay mixtures were prepared at the optimum water content and placed into the shear box of direct shear device. The samples were then consolidated to the stress of 1600 KPa incrementally before they were unloaded to the stress at which they were sheared. The samples were sheared under effective normal stresses of 50, 100, 150, 200, 300 and 400 KPa at speed of 0.1 mm/min.

Table 1. index properties for mixtures of sand between sieves No. 10-200 and clay

% fine	% clay	LL	PL	PI	G_s	Soil class
0	0	NP	NP	NP	2.64	SW
15	3.4	NP	NP	NP	2.64	SM
20	4.6	NP	NP	NP	2.65	SM
30	6.9	21.5	14	7.5	2.65	SC
40	9.2	27.3	18.2	9.1	2.66	SC
100	22.9	34.4	20.1	14.3	2.70	CL

Table 2. index properties for mixtures of sand between sieves No. 30-200 and clay

% fine	% clay	LL	PL	PI	G_s	Soil class
0	0	NP	NP	NP	2.64	SW
15	3.4	NP	NP	NP	2.64	SM
20	4.6	NP	NP	NP	2.65	SM
30	6.9	NP	NP	NP	2.65	SM
40	9.2	24.8	16.5	8.3	2.66	SC

Table 3. Index properties for mixtures of sand between sieves No. 50-200 and clay

% fine	% clay	LL	PL	PI	G_s	Soil class
0	0	NP	NP	NP	2.64	SW
15	3.4	NP	NP	NP	2.64	SM
20	4.6	NP	NP	NP	2.65	SM
30	6.9	NP	NP	NP	2.65	SM
40	9.2	22.4	14.6	7.8	2.66	SC

Table 4. Results of Proctor test on the mixtures of sand between sieves No.10-200 and clay

% fine	w_{opt} (%)	γ_{max}	γ_{dmax} (g/cm^3)	e_{min}
15	11.15	2.195	1.973	0.336
20	10.8	2.224	2.028	0.309
30	9.3	2.24	2.046	0.298
40	11.8	2.21	1.972	0.355

Table 5. Results of Proctor test on the mixtures of sand between sieve No. 30-200 and Clay

% fine	w_{opt} (%)	γ_{max}	γ_{dmax} (g/cm^3)	e_{min}
15	11.6	2.12	1.87	0.396
20	11.4	2.18	1.95	0.359
30	11.2	2.2	1.97	0.347
40	12	2.19	1.954	0.362

Table 6. Results of Proctor test on the mixtures of sand between sieve no. 50-200 and clay.

% fine	w_{opt} (%)	γ_{max}	γ_{dmax} (g/cm^3)	e_{min}
15	11.2	2.09	1.85	0.404
20	11	2.15	1.935	0.374
30	10.7	2.19	1.95	0.352
40	11.1	2.17	1.94	0.359

4. TEST RESULTS AND DISCUSSION

4-1. Shear stress – displacement relationship

Figures 4, 6 and 8 show the relationship of shear stress – displacement for samples tested. At all mixtures the increase in shear strength due to increase in effective normal stress and decrease in over consolidated ratio, OCR, is observed. The increase in strength is due to the increase in contact area of soil particle as a result of the increase in the effective normal stress.

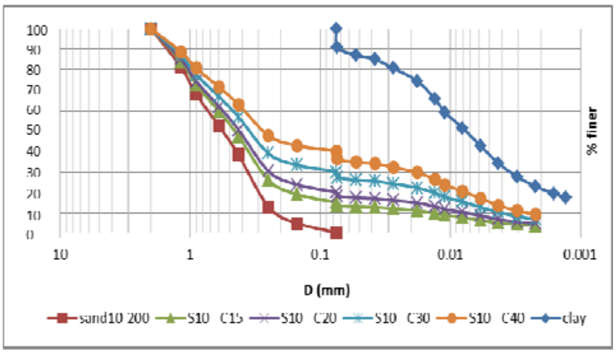

Fig.1 Grain size distribution of sand with gradation between sieves No. 10-200 and various clay content

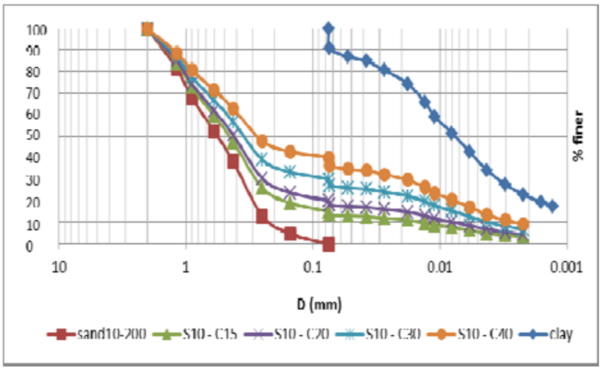

Fig. 2 Grain size distribution of sand with gradation between sieves No. 30-200 and various clay content

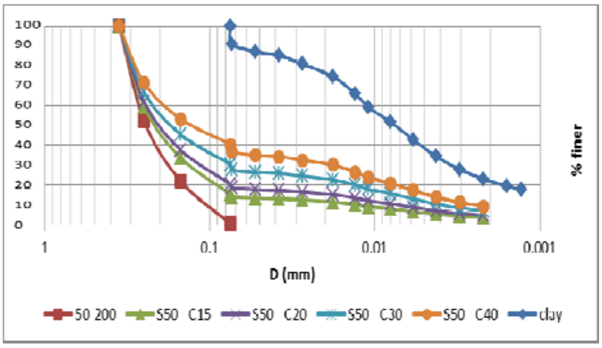

Fig. 3 Grain size distribution of sand with gradation between sieves No. 50-200 and various clay content

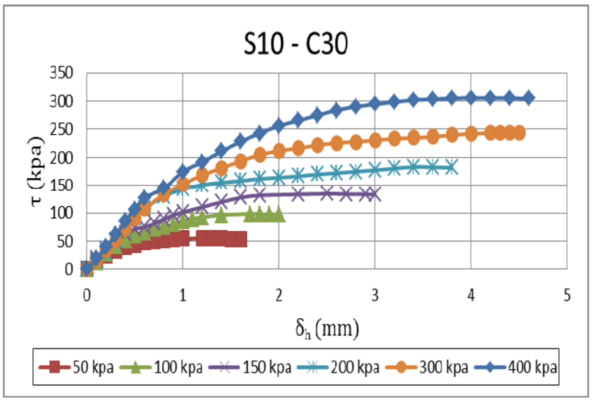

Fig.4 Shear stress-shear strain curve of sand between sieves No. 10-200 and 30% clay

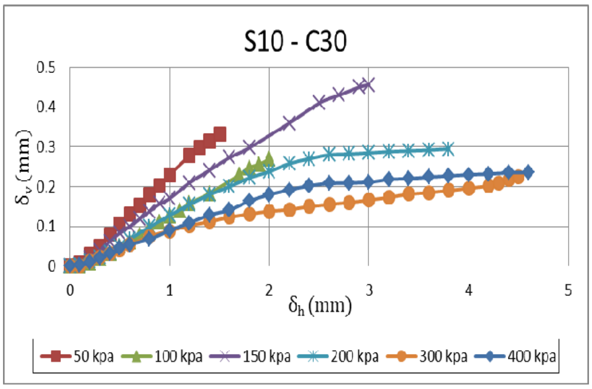

Fig.5 Vertical vs. horizontal displacement curve of sand between sieves No. 10-200 and 30% clay

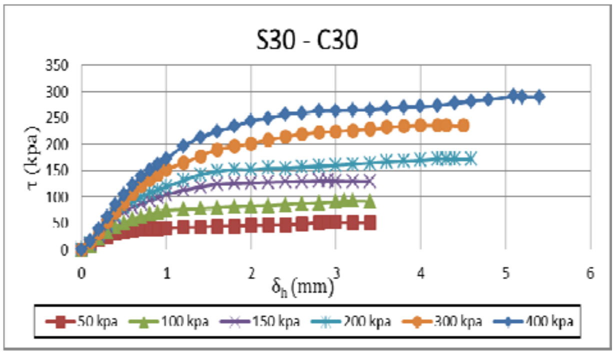

Fig.6 Shear stress-shear strain curve of sand between sieves No. 30-200 and 30% clay

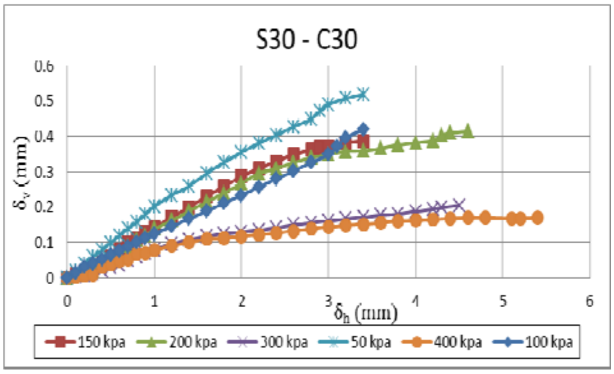

Fig.7 Vertical vs. horizontal displacement curve of sand between sieves No. 30-200 and 30% clay

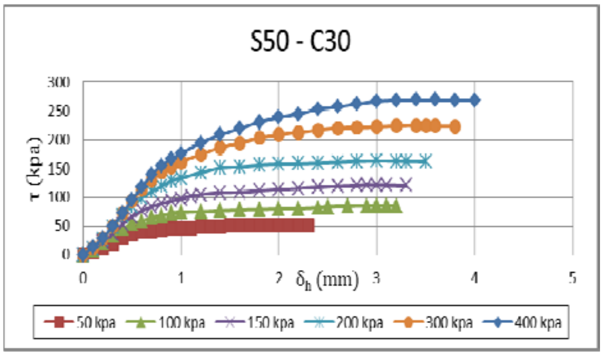

Fig.8 Shear stress-shear strain curve of sand between sieves No. 50-200 and 30% clay

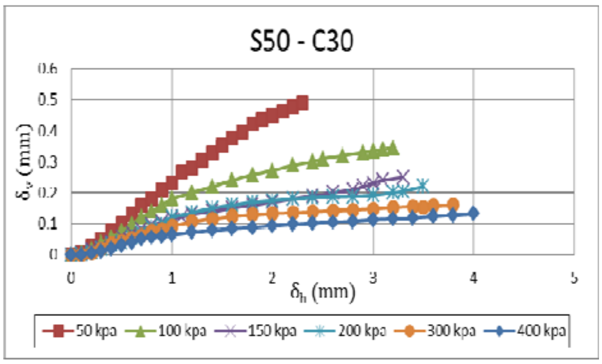

Fig.9 Vertical vs. horizontal displacement curve of sand between sieves No. 50-200 and 30% clay

4-2. Vertical vs horizontal displacement
The vertical vs horizontal displacement of samples tested are shown in figures 5, 7 and 9. In all samples dilated behavior was observed. According to these figures, at high normal stresses, the samples first showed constant value with shearing and then experienced increase in volume with further shearing. But at lower normal stresses the samples experienced the increase in volume from the beginning of shearing process. There are two reasons for dilative behavior of samples during shearing. First reason is due to swelling of clay particles at sliding plane formed in the samples during shearing. The second reason is the presence of coarser sand particles in compacted and dense fabric that climb over each other during shearing and create dilative behavior. Because of low plasticity of the clay, the first reason for dilative behavior of samples during shearing may not be significant in this case.

4-3. Mohr – coulomb failure envelope
In overconsolidated clay soil, the drained shear strength commonly is expressed in terms of a cohesion intercept c' and angle of friction ф' [13], Eq. (1).

$$\tau = C' + \sigma' \tan\varphi' (OCR)^{1-m} \qquad (1)$$

In order to obtain failure envelope for each test series, the maximum shear strength under each effective normal stress is taken from stress – strain relationship obtained and plotted against the corresponding effective normal stress as shown in Figs. 10-12. As it can be seen from these Figs that a curved relationship rather than a line defined best the failure envelopes obtained through data points.

4-4. Determination of parameter m
Because of the curvature in failure envelope of overconsolidated clay defining shear strength in terms the shear strength for overconsolidated material can be alternatively expressed in terms of normally consolidated shear strength of the same constituent as Eq. (2):

$$\tau(OC) = \tau(NC)\,(\mathrm{OCR})^{1-m} \qquad (2)$$

According to Eq. (2) the shear strength of overconsolidated soil τ_{oc} is higher than normally consolidated shear strength τ_{nc} of the same constituent by a factor of $(OCR)^{1-m}$. The OCR in Eq. (2) is expressed in terms of the stress at which overconsolidated portion of failure envelope joins the normally consolidated portion.
The value of coefficient m in Eq. (2) is for every soil independent of the effective normal stress and it is the slope of the logτ – logσ relationship. Figures 13-15 show the procedure for determining the value of m for each test series.

4-5. Effect of clay fraction on the value of m
According to Table 8 with the increase in CF (clay fraction) and decrease in sand fraction the value of C' is increased but because of low plasticity of the clay used, the amount of increase in C' insignificant. But according to Figure 16 the relationship between C' and m is reversed which means with the increase in C' the value of m is decreased. The explanation for this behavior can be stated as follow: with the increase in clay fraction physical and chemical bonds between soil grains is increased and the cohesion within the samples is improved and its behavior approaches that for intact clay and with a decrease in fine content, the intergrain bonds is decreased and the behavior of the soil approaches a sandy soil and therefore a higher value of m is obtained**.**

4-6. Effect of gradation
Figure 17 shows the effect of sand gradation of mixtures on the minimum void ratio of samples. According to this figure e_{min} is first decreased with the increase in fine content and then it is increased. In table 7 the fine content at which this change in e_{min} behavior is observed for different sand gradation is listed. The amount of threshold fine content is increased as sand gradation becomes finer, a difference of 2 to 3% in threshold fine content is observed. With finer sand gradation, uniform gradation is obtained therefor the void between sand grains is increases and more amount of clay is required to fill the voids.

Table 7. Threshold value of fine content for different gradation

gradation	#10 - 200	#30 - 200	#50 - 200
Threshold value	%27	%30	%32

4-6-1. Effect of sand gradation on the failure envelope
In Figure 18 over consolidated failure envelopes for different sand gradation for fine content of 30% are shown. As it can be seen as sand gradation becomes finer the curvature of the envelope is increased. Also at any effective normal stress the shear strength for finer gradation is lower than that for coarser gradation. With sand size becoming finer the mixture becomes more uniform that causes a higher void ratio and a lower interlocking between sand particles that results a decrease in shear strength.

4-6-2. Effect of sand gradation on m, C' and φ'
In order to evaluate the effect of sand gradation of the values of m, C' and φ' the results of all tests performed are tabulated in table 8. With increase in gradation the value of m is increased and with decrease in sand gradation as expected the φ' decreased and C' increased.

5. CONCLUSIONS

According to consolidated – drained direct shear tests results performed on mixtures of sand – clay with different sand gradation and fine content following conclusions are obtained:

1. Shear strength and stress – strain characteristics of mixtures show significant changes at fine content of about 30%.
2. The drained angle of friction and therefore drained shear strength is decreased with increase in fine content.
3. The threshold value of clay fraction at which the trend in changing e_{min} of mixtures with fine content is changed is about 27-30% depending on sand gradation. The higher range belongs to finer gradation.
4. The value of m is increased with sand fraction and vice versa it is decreased with fine content.

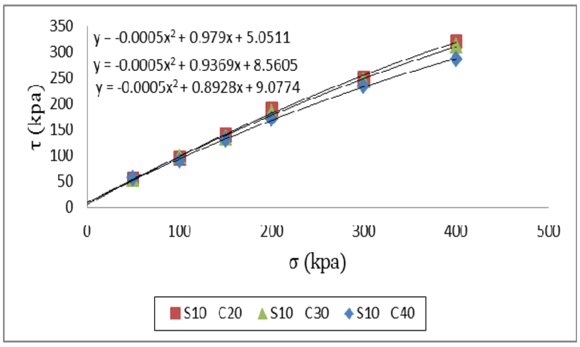

Fig.10 Over consolidated failure envelope for sand between sieves No. 10-200 content of various percent of clay

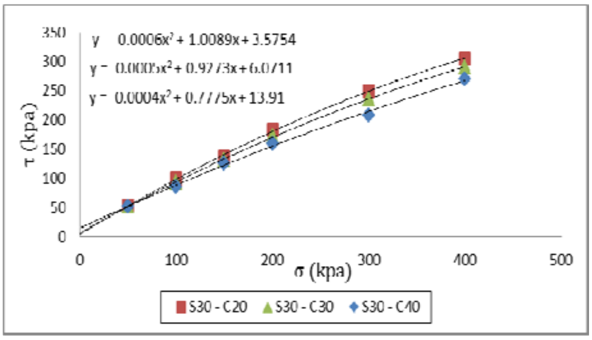

Fig.11 Over consolidated failure envelope for sand between sieves No. 30-200 content of various percent of clay

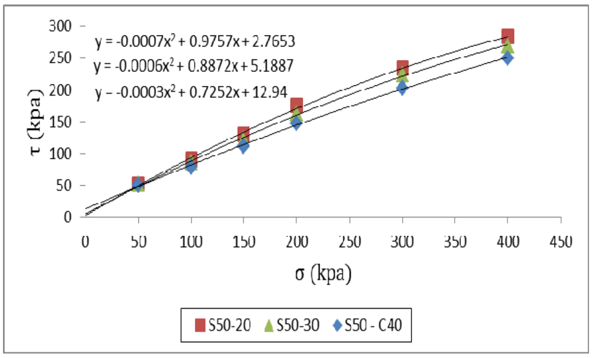

Fig. 12 Over consolidated failure envelope for sand between sieves 50-200 content of various percent of clay

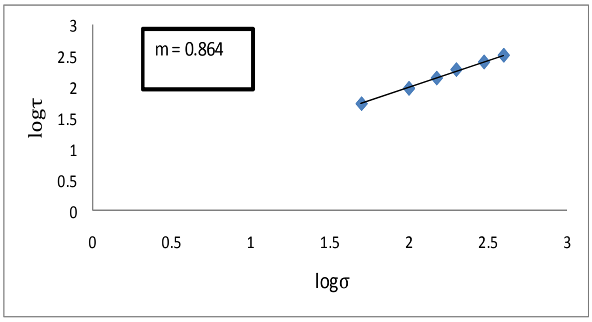

Fig.13 Parameter m for mixture of sand between sieves No. 10-200 and 20% clay

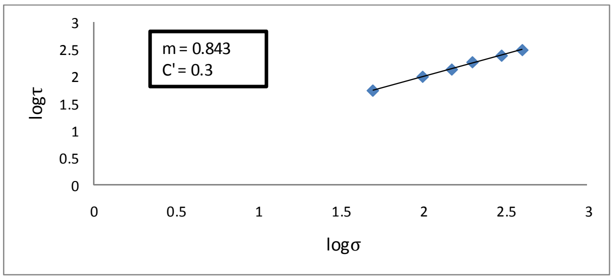

Fig. 14 Parameter m for mixture of sand between sieves No. 10-200 and 30% clay

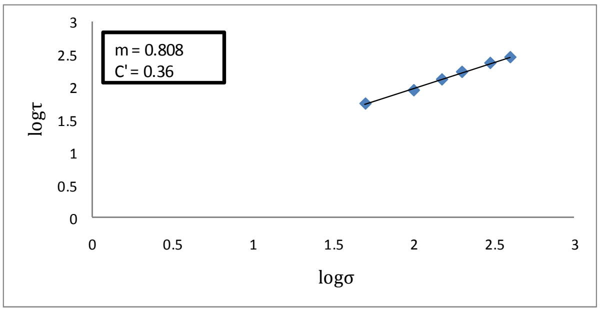

Fig. 15 Parameter m for mixture of sand between sievesNo. 10-200 and 40% clay

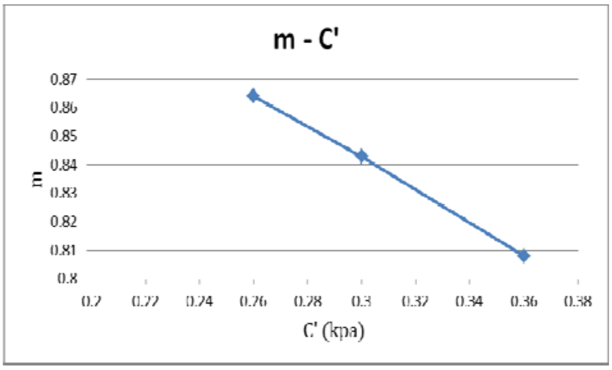

Fig. 16 Parameter m versus C'

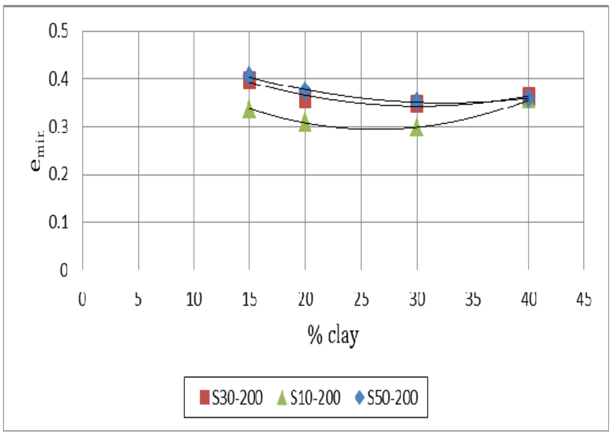

Fig. 17 Minimum void Ratio versus % clay

Table 8. Values of m, C' and φ'

Clay percent	20%			30%			40%		
gradation	m	φ'(∘)	C' (kpa)	m	φ'(∘)	C'(kpa)	m	φ'(∘)	C'(kpa)
#10 - 200	0.86	33.8	0.26	0.843	32.23	0.3	0.808	29	0.36
#30 - 200	0.85	32.3	0.29	0.837	30.52	0.32	0.799	26.86	0.37
#50 - 200	0.83	30.3	0.3	0.818	28.64	0.33	0.784	24.84	0.38

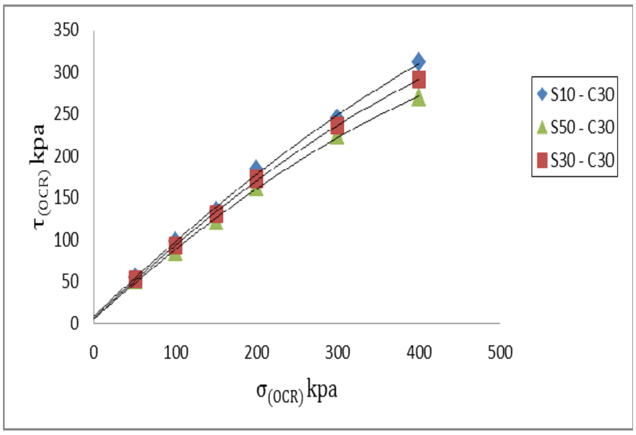

Fig. 18 Over consolidated failure envelope for various sand gradation with 30% clay

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