Evaluation of Undrained Shear Strength of Loose Silty Sands Using CPT Results

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Abstract: *Series of undrained monotonic triaxial tests and cone penetration tests were conducted on loose silty sand samples to study correlation between undrained shear strength of silty sands (S_{us}) and piezocone test results. CPT tests were conducted at 27 silty sand samples in calibration chamber. The results indicate that, in low percent of silt (0-30%), as the silt content increases, the undrained shear strength (S_{us}) and cone tip resistance (q_c) decreases. It is shown that, fines content affects undrained shear strength (S_{us}) and cone tip resistance (q_c) similarly. On the basis of obtained results, equations were proposed to determine the normalized cone tip resistance (q_{c1n}) and undrained shear strength (S_{us}) of silty sand in term of fines content. Finally based on those equations, a correlation between normalized cone tip resistance and undrained shear strength of silty sand is presented. It is shown that the normalized undrained shear strength and normalized cone tip resistance of loose silty sands (F.C. <30%) decreases with increase of silt contents.*

Keywords*: shear strength, piezocone test, triaxial test, cone tip resistance, fines content.*

1. Introduction

The shear strength of a liquefied soil is an important component in seismic geotechnical engineering evaluation. Determination of soil strength parameters for stability analyses remains a challenging task. At present, there are three methods for assessing the residual shear strength of loose silty sand. 1) Correlations of residual field strength (S_r) versus normalized standard penetration resistant $(N)_{60\text{-}cs}$ [1]. 2) Evaluation of residual strength (S_{us}) obtained from monotonic consolidated undrained triaxial tests on reconstituted samples [2]. And 3) the normalized strength ratio approach [3, 4, 5], which shear strengths of liquefied soil back-calculated from case histories of liquefaction failure are normalized with respect to preliquefaction effective overburden pressure to determine critical strength ratios.

The cone penetration test is becoming a useful tool for geotechnical site characterization. The wide use of the cone penetration test in geotechnical engineering practice has resulted in a great demand to validated correlations between cone tip resistance and engineering properties of soil. Many field studies on the CPT results in silty sand soils reported by Robertson and Campanella [6], Seed and DeAlba [7], Stark and Olson [8] , Robertson and Wride [9], Olson [10], Rahardjo et al [11], Baziar and Ziaie_Moayed [12,13], and Ziaie_Moayed [14], indicated that the fines content of non-clean sand influences the cone resistance.

Experimental data of silty sand samples reported by Pitman et. al. [15], Zelatovic and Ishihara [16], Thevanayagam et. al. [17], Yamamuro and Lade [18], Vaid et. al. [19], Naeini [20], Naeini and Baziar [21], indicated that, deformation characteristics and pore pressure build up in silty samples were quite different from clean sand and as the fines content increases up to 30 percentages, the steady state strength decreases.

Most of the S_{us} values determined from field case histories of failures pertain to silty sands [17,22]. These observations suggest a possible correlation between the laboratory and field S_{us} values and fines content in silty sands. Further understanding of the factors contributing to the low shear strengths observed for silty sands both in the laboratory and in the field. The role of fines on the reduction in S_{us} would help in the formulation of a consistent method for strength characterization of sandy soils containing fines.

The objective of this research is to evaluate the influence of silt contents on cone tip resistance (q_c) and undrained shear strength (S_{us}) of loose silty sand mixtures and correlation between these values.

2. Test Method and Equipments

2.1 Calibration Chamber

A schematic illustration of the calibration chamber assembled for cone penetration testing, used in this study, is shown in Figure.1. The testing chamber consisted basically of a rigid thick walled steel cylinder of 0.76-m internal diameter,1.50-m height and10 mm thickness, with removable top and bottom plates [14]. The 10 mm thickness is determined using strength material calculation to obtain the zero lateral displacement of chamber against the maximum soil pressure in the test [14].

2.2 Cone Penetrometer

The piezocone was inserted into the chamber by a hydraulic system. Standard piezocone used in this investigation has 10 cm^2 projected tip area and a 150-cm^2-friction sleeve area. In this penetrometer, friction sleeve was sited immediately behind the cone tip. The filter element to record pore water pressure was located immediately behind the cone tip. The piezocone was advanced through soil at a constant rate of 20 mm/sec. Three sets of data including cone tip resistance, friction resistance and pore water pressure can be recorded continuously during sounding in each 1 cm of depth.

2.3 Triaxial Appratus

The triaxial test on cylindrical specimen is presently the most widely used procedure to evaluate several important properties of the soil behavior such as strength, stress-strain relationship and dilatancy among others. For the present study, a classical strain-controlled triaxial test apparatus was used. Electrical transducers (pore pressure, force, volume change and displacement) were used, allowing an automatic data acquisition during the test for subsequent processing of data. The procedure involves the classical steps needed to perform a triaxial test (saturation, consolidation and loading).

3. Material Properties

To accomplish the objectives of this study, approximately 60 tons of Tello clean fine sand was acquired. This alluvial soil is fine clean sand without any clay or silt particles that contain sub-angular fine-grained quartz sand. In order to determine the influence of silt content on cone tip resistance and residual shear strength, the pure silt was obtained from grinding of Tello fine sand. Approximately 10 tons of silt material was obtained to prepare different mixture of silt and sand. The properties of the soil used during this study are shown in Table 1. The silty sand soils were prepared by mixing

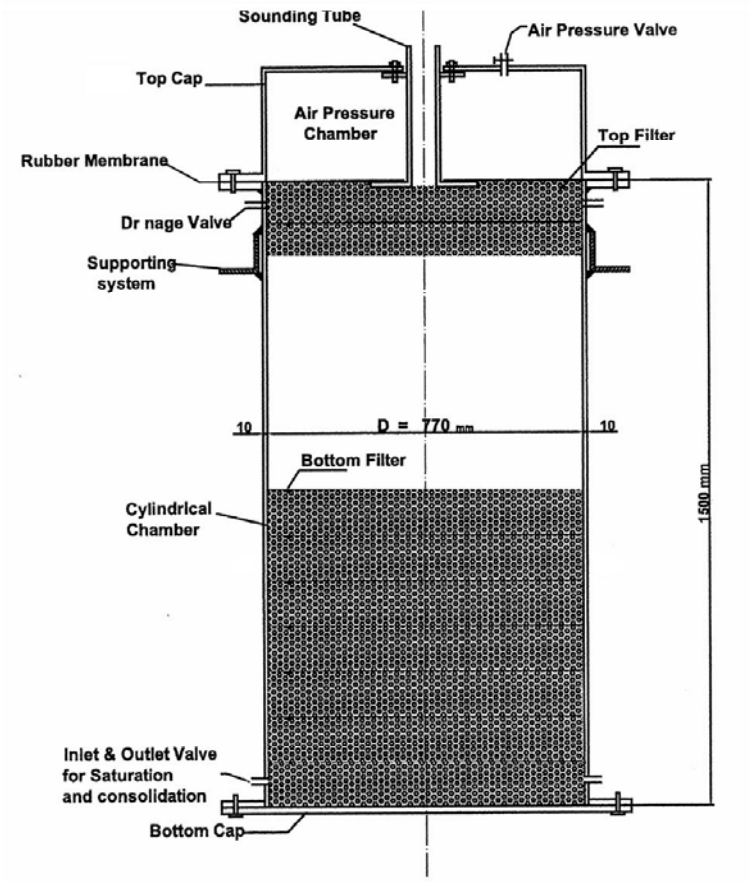

Fig. 1. IUST Calibration Chamber

Table 1 Properties of tested materials

Material	D_{50} (mm)	C_U	F.C.* (%)	e_{min}	e_{max}	D_r (%)
Tello clean sand	0.40	3.0	0	0.746	1.09	29
Tcs-10	0.38	5.60	10	0.625	1.16	37
Tcs-15	0.35	7.10	15	0.608	1.20	47
Tcs-20	0.34	7.50	20	0.594	1.24	59
Tcs-25	0.33	7.45	25	0.584	1.28	67
Tcs-30	0.32	7.40	30	0.578	1.33	74

- *Content of grains smaller than 0.075 mm

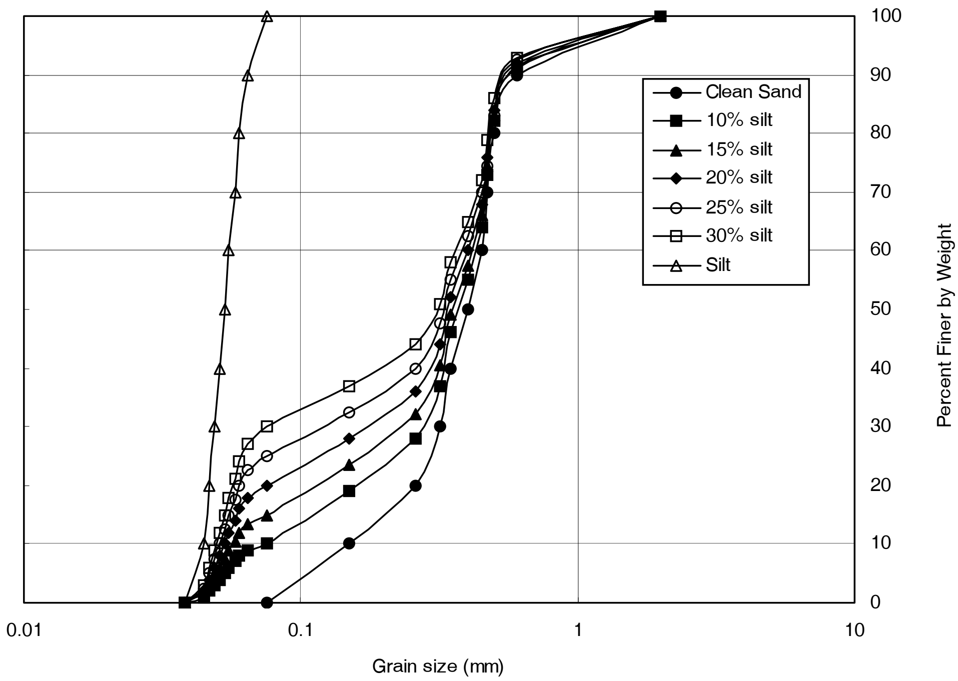

Fig. 2. Grain size distribution curves of tested materials

appropriate amounts of Tello sand with silt. The fines content ranged from 0 to 30 % of the specimen. The grain size distribution curves for the soils are shown in Figure 2. The difference in particle arrangement of the specimens is because of different in fines content. In geotechnical engineering there are two terms concerned with the arrangement of soil particles, these are "fabric" and "structure" of a soil mass. These terms are used interchangeably to describe degree of packing, orientation of particles, heterogeneities, cementation and in general any feature of the arrangement of soil particles. The term "fabric" is proposed to describe the initial arrangement of soil particles that can be completely broken down under large deformation. On the other hand, the term "structure" is preferred to be used to describe all those particle configurations that can be fully erased by straining independently of how large this might be (Verdugo [23]). Therefore, because the steady state is achieved under a large level of deformation, the initial particle arrangement is strongly modified and in the case of homogeneous soil mass, the initial arrangement of particles is completely erased when the steady state of deformation takes place. Thus the steady state is unaffected by the initial particle arrangement. But it can be strongly affected by the initial structure of the soil mass.

4. Maximum and Minimum Void Ratio

The role of fines content on minimum and maximum void ratios of sand and the relationships between these quantities are important. Many engineering properties of sands relate well to relative density (D_r). This quantity has therefore often been used as an indicator for those properties. When silt is present in sand, it mostly fills the pores among the sand and sand-grain to sand-grain contact decreases. Thus changing the structure of sand, decreasing the void ratio and increasing relative density.

Fines content plays an important role in determining sand structure and the consequent extreme void ratios. These in turn, have significant influences on undrained strength of a sand deposit. Therefore, the use of relative density as sole indicator for liquefaction potential is questionable (Thevanayagam [17], Lade et al [24]). In this study as indicated by Castro [25], Ishihara [26] and Koester [27], because of present of silt contents and changed sand structure, the relative density of samples are different. So the state of stress of silty sand deformations is determined uniquely by the void ratios. The maximum and minimum void ratios determined by method of JSSMFE and relative density of samples are shown in Table 1.

5. Method of Sample Preparation

Samples were prepared in the loosest state using dry deposition method. Sand and silty sand were spread in the forming mould (triaxial test) and calibration chamber (CPT test) with zero height of fall at a constant speed until the mould and chamber filled with the dry sand and silty sand. Tapping energy was applied by hitting the side of the mould and chamber to obtain a desired density. After the sample were encased in the membrane with the top cap, a vacuum of 10-20 kPa were applied and carbon dioxide gas percolated through the sample, which were then flushed with de-aired water, making it saturated. Ensuring attainment of a B value greater than 0.97 by means of back pressure application checked full saturation. After saturation stage in CPT tests, the vertical pressure is applied at the top of sample using the air pressure chamber and the sample is consolided vertically at the zero laterally strained condition.

6. CPT Test Results

A total of eighteen cone penetration tests were performed in calibration chamber including three samples of clean sand and fifteen samples of silty sand. All the tests reported in this research were conducted in four stages including sample preparation, saturation, consolidation, CPT sounding [14]. Table 2 presents the summary of CPT results, obtained in this study.

Figure. 3 show typical results from continuous cone penetration with pore water pressure measurement in normally consolidated clean sand samples at three different consolidation pressures. Figure. 4 indicate the same values corresponding to a sand sample containing 15 percent of silt. It should be noted that in each sample, there is a 20-cm top and 80 cm bottom filters and the total length of soil sample is about 50 cm (Figure 1). This height of tested sample (50-cm) was obtained from test results to achieve the homogeny condition of tip resistance distribution along the sample depths [14]. Due to obtained results it can be concluded that, the consolidation stresses are reduced in depths greather than 50 cm and therefore, the homogeneity of sample is disturbed [14]. It can be seen that from the top of specimen, the q_c value increases with depth and reaches to a maximum value in top filter zone area, then reduces and remains constant along the soil sample. This pattern is observed in all clean and silty sand samples. At the end, when the cone tip reaches to bottom filter (depth 70 cm), the cone tip resistance

Table 2 Summery of CPT results

Test No.	Type of Material	F.C (%)	σ´c =100 (kPa)	σ´c =200 (kPa)	σ´c =300 (kPa)
			q_c (MPa)	q_c (MPa)	q_c (MPa)
1	Clean Sand	0	1.6	3.5	4.0
2	Silty Sand	10	1.4	3.0	3.4
3		15	1.3	2.6	3.0
4		20	1.2	2.2	2.8
5		25	0.95	1.75	2.6
6		30	0.80	1.45	2.5

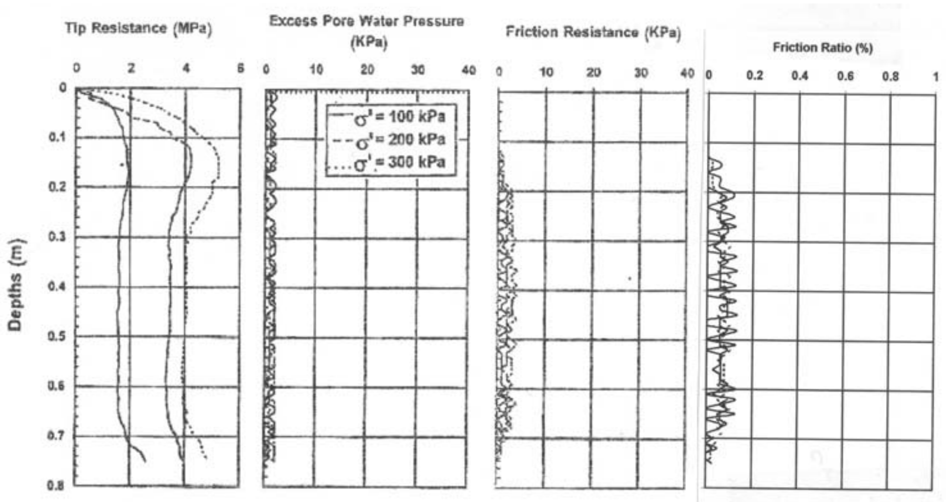

Fig.3. Typical CPT Results in Clean Sand Specimen

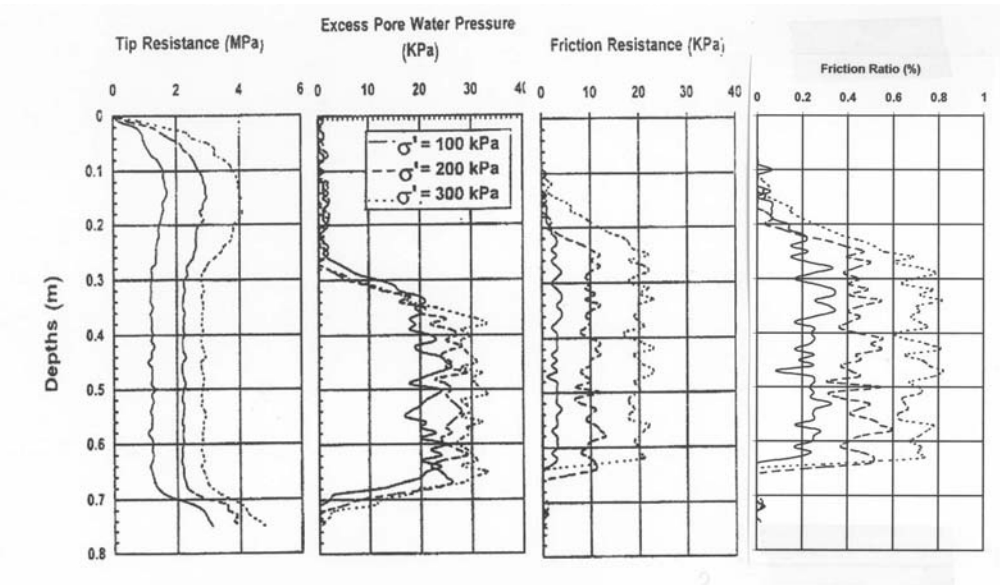

Fig. 4. Typical CPT Results in Silty sand Specimen (15% silt content)

increases again.

Excess pore water pressure is approximately zero in the top and bottom filter zone (20 cm of up and down part of each sample). In clean sand samples at different consolidation pressures (100,200,300 kPa), the excess pore water pressure is almost zero (Figure 3), while in silty sand samples (Figure 4), excess pore water pressure grows up and reaches to a constant value in the main part of sample and then reduces to zero in bottom filter zone (depth 70 cm).

The friction resistance and firction ratio values values are neigliable in clean sand specimens (Figure 3). However, a large amount of friction resistance and friction ratio is recorded during sounding in silty sand samples (Figure 4). This phenomenon is related to fine grained soil behavior of sandy silt to silty sand. It seems that the friction resistance and also the friction ratio are important parametere for classification of these type of soils.

Results of cone penetration tests show the influence of silt content on excess pore water pressure. In clean sand samples in order to existence of full drainage condition, the excess pore water pressure readings are negliable. But in silty sand samples, excess pore water pressure grows up.

7. Effect of Silt Content on Cone Tip Resistance

Robertson and Wride [9] found that the CPT penetration resistance in silty sand is smaller due to greater compressibility and decreased permeability of silty sands. They suggested the following correlations for correction of tip resistance based on fine content:

$$(\mathbf{q_{c1n}})_{\mathbf{cs}} = \mathbf{q_{c1n}} + \mathbf{\Delta q_{c1n}} \quad (1)$$

Where:

$(q_{c1n})_{cs}$= Normalized cone tip resistance, corrected for fine content

q_{c1n} = Measured CPT tip resistance, corrected for overburden and normalized of silty sand

$$\mathbf{\Delta q_{c1n} = [K_{cpt}/(1-K_{cpt})] * q_{c1n}} \quad (2)$$

Where:

$$\mathbf{K_{cpt}=0 \quad for \quad FC<5\%} \quad (3a)$$

$$\mathbf{K_{cpt}= 0.0267(FC-5) \quad for \quad 5\%<FC<35\%} \quad (3b)$$

$$\mathbf{K_{cpt}= 0.8 \quad for \quad FC>35\%} \quad (3c)$$

FC = Fines content, in percentage

The recommended procedure is to determine the fines content of soil directly and apply a correction factor to the measured q_{c1n} value using Equations 2 and 3, then uses the Equation 1, to estimate the equivalent clean sand normalized CPT tip resistance.

Based on obtained results, values of normalized cone tip resistance (q_{c1n}) versus fines content is presented in Figure 5.

The relation shown in Figure 5, which is proposed to determine the normalized cone tip resistance of loose silty sand in term of fines content, can be represented as following equation:

$$\mathbf{q_{c1n} = -0.0243\ (FC) + 1.6} \quad (4)$$

For **0 < FC (%) < 30%**

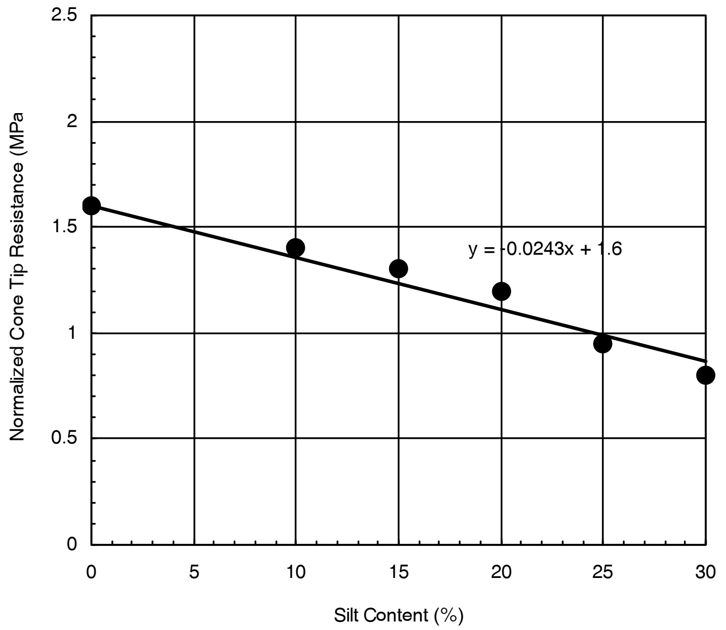

Fig. 5. Normalized cone tip resistance versus silt content

In the range of 0-30% of silt, system acts similar to a coarse granular matrix. Behavior of soil sample in this case is related to contacts of coarse grain and quantified by inter-granular void ratio. By increase of silt content in range of 0% to 30%, the contact between sand particle decreases and hence the q_c decreases.

8. Triaxial TestsResults

Several series of undrained monotonic triaxial compression tests were performed on reconstituted saturated samples of Tello sand with different fines content, in order to clarify the effect of fines content on steady state strength.

Samples were made up of sand with silt content ranging from 0-30%. Specimens were prepared in the loosest state using dry deposition method. The specimens were 50.8 mm (2 in) in diameter and 101.6 mm (4 in) in height. Properties of the tested samples are shown in Table1 and grading curves are shown in Figure.2. Specimens were consolidated isotropically at mean effective pressure between 100-300 kPa and subjected to undrained monotonic triaxial loading with a constant strain rate of 1% per minutes.

In order to compare the behavior of sands with different amount of silt, which causes differences in void ratio, it is very useful to compare the steady state lines of sands with different amount of silt contents. Figure 6 clearly shows that, the position of the steady state line is influenced by the silt contents. As the silt contents increase, the steady state line moves downward in the log σ'_{3us}-e diagram and the contractiveness is increased [20]. This is accordance with the decrease in strength with an increase in silt content. This phenomenon is explained by the fact that fine grains of silt positioned around and among the sand grains acting as filling as well as lubricant. This means that the silt not only

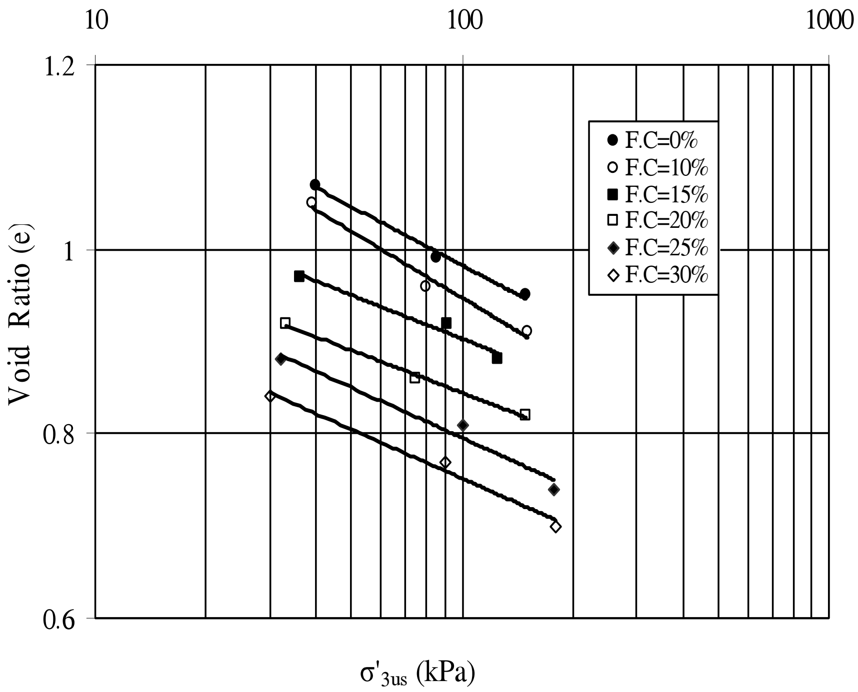

Fig. 6. Steady state lines for monotonic triaxial tests

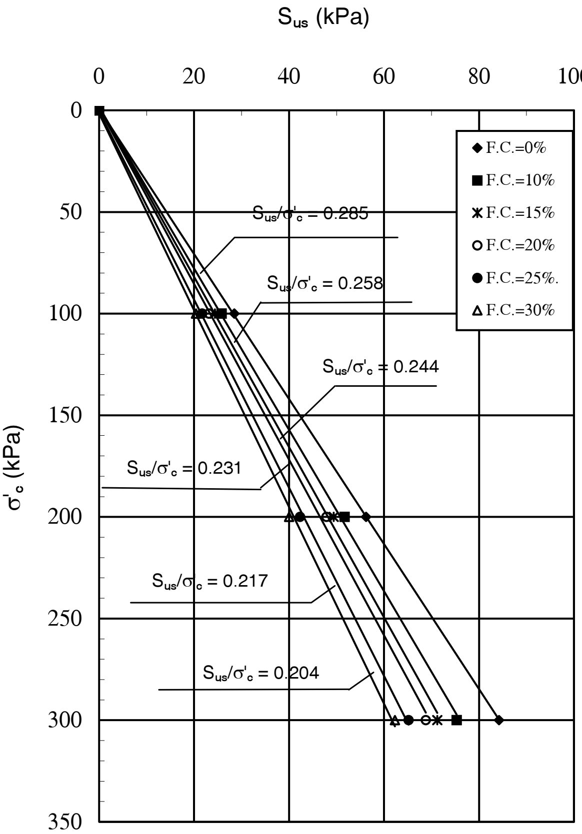

Fig.7. Undrained shear strength versus initial confining pressure

Table 3 Summery of triaxial test results

Test No.	Type of Material	Sample Preparation	F.C (%)	σ'_c (kPa)	e*	S_{us} (kPa)
1	Clean Sand	Dry Deposition Method	0	100	1.07	28.5
2				200	0.99	56.2
3				300	0.95	84.2
4	Silty Sand		10	100	1.05	25.8
5				200	0.96	51.7
6				300	0.91	75.3
7			15	100	0.95	24.4
8				200	0.92	49.4
9				300	0.88	71.2
10			20	100	0.92	23.1
11				200	0.89	47.9
12				300	0.82	68.8
13			25	100	0.88	21.7
14				200	0.81	42.3
15				300	0.74	65.2
16			30	100	0.86	20.4
17				200	0.79	40.0
18				300	0.71	62.3

*e- is the void ratio at the end of the consolidation.

filled the pores among the sand grains, but also made the sand skeleton looser as the silt content increased up to 30 percentages.

The results of monotonic triaxial tests shown in Figure 7 indicated that, in these series of samples, with increase in silt content up to 30%, the residual strength steadily decreases. It is also concluded that at the same void ratio, the clean sand samples have the highest S_{us} compared with silty sand. This is in accordance with increase in excess pore water pressures by increase in silt contents. This phenomenon is explained by the fact that, by increase of silt contents in the range of 0-30%, the fine grains of silt positioned among the sand grains, changing the structure of sand, and decreasing the void ratios, therefore excess pore water pressures increases and S_{us} decreases. The results of triaxial test obtained in this study are shown in Table 3.

Based on the test results as indicated in Figure 7, it can be concluded that, the ratio of S_{us} / σ'_c is constant and reasonably independent of σ'_v. Based on this finding, we can use such a linear relationship to estimate the residual strength instead of in-situ measurements of void ratio. On the basis of obtained results, values of normalized residual strength versus fines content are presented in Figure 7. In the current study, for the range of 0-30% fines content for Tello sand in normally consolidated undrained triaxial compression tests, the following expression is proposed from Figure 8, to determine the normalized undrained shear strength in terms of fines content.

$$\mathbf{S_{us} / \sigma'_c = 0.285 - 0.0027(FC)} \qquad (5)$$

For $0 < FC\ (\%) < 30\%$

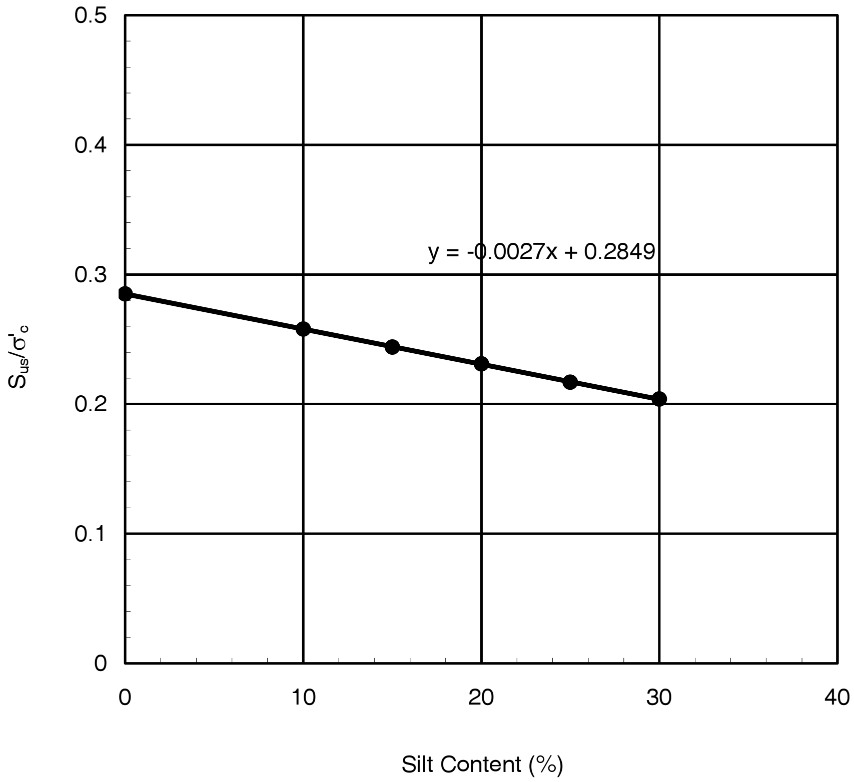

Fig. 8. Normalized residual shear strength versus fines content

9. Correlation Between S_{us} and q_{c1n}

The results indicating that, fines content play important roles affecting undrained shear strength (S_{us}) and cone tip resistance (q_c) similarly. It is shown that, in low percent of silt (0-30%), as the silt content increases, the cone tip resistance (q_c) and undrained shear strength (S_{us}) decrease. Based on the obtained results, equations were proposed to determine the normalized cone tip resistance (q_{c1n}) and undrained shear strength (S_{us}) of silty sand in term of fines content (Equations 4 and 5). Finally based on those equations, a correlation between normalized cone tip resistance (q_{c1n}) and undrained shear strength of silty sand is presented as the below equation:

$$\mathbf{S_{us} / \sigma'_c = 0.107 + 0.111 * (q_{c1n})} \qquad (6)$$

For **0 < FC (%) < 30%**

It can be resulted that the normalized undrained shear strength of loose silty sand (FC < 30%) increases linearly by increase of normalized cone tip resistance.

10. Conclusions

The objective of this paper was to study the steady state shear strength of silty sand using cone penetration test results. In the light of the experimental evidence for Tello silty sand samples prepared in the loosest state using dry deposing method, the following conclusions drawn:

1- The amount of silt content in sand is an important parameter affecting cone tip resistance. As the silt content increase, the cone tip resistance decrease.

2- The normalized cone tip resistance of loose silty sand in terms of fines content can be epresented by:

$$q_{c1n} = -0.0243\ (FC) + 1.6$$

It is shown that the normalized cone tip resistance of loose silty sands (FC <30%) decreases with increase of silt content.

3- Pore water presuures increase with increase in silt contents. This is accordance with present of silt contents, changing the structure of sand and decreasing the void ratios.

4- The results of monotonic triaxial tests shown that the ratio of S_{us} / σ'_c is constant and reasonably independent of σ'_v. On the basis of this finding, we can use such a linear relationship to estimate the residual strength instead of in-situ measurements of void ratio.

5- The normalized undrained shear strength in terms of fines content can be determined by:

$$S_{us} / \sigma'_c = 0.285 - 0.0027(FC)$$

It is shown that the normalized undrained shear strength of loose silty sands (FC <30%) decreases with increase of silt content.

6- Correlation between normalized undrained shear strength and cone tip resistance of Tello loose silty sand can be presented as:

$$S_{us} /\sigma'_c = 0.107 + 0.111 * (q_{c1n})$$

It can be concluded that the normalized undrained shear strength of Tello loose silty sand (FC < 30%) increases linearly by increase of normalized cone tip resistance.

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