Strength-Flow Parameters of Loose Silty Sands From Piezocone Tests

Reza Ziaie_Moayed[i]; Seyed Abolhassan Naeini[ii]; Mohammad Hassan Baziar[iii]

ABSTRACT

Piezocone penetration tests with dissipation phases are particularly useful for geotechnical site characterization. They also provide three independent readings with depth from a single sounding as well as time-rate information. In past investigations, only silty sands with specific silt content have been tested and complete sets of tests have not been conducted to evaluate the influence of different silt contents on CPTU results. In the present research, the complimentary CPTU tests with pore pressure dissipation phase and soil laboratory tests (including consolidation, permeability and triaxial tests) are also performed on silty sand samples with different silt content. In this study, six piezocone tests are performed in saturated silty sand samples with several different silt contents ranging from zero to 50 percent. The pore pressure dissipation tests are also carried out in samples and t_{50} (the time for 50% pore pressure dissipation) is evaluated. Laboratory tests including consolidation, permeability and triaxial tests are also performed for the soil parameter determination. Based on the obtained results, the interrelationships among “K- t_{50}”, “C_v - t_{50}” and “ϕ'-q_c” are finally presented.

KEYWORDS

Silty Sand, Pore Pressure Dissipation, CPTU, Coefficient of Consolidation, Permeability

1. INTRODUCTION

An increased concern is noticed recently toward the evaluation of different engineering soil parameters (e.g. geotechnical parameters, flow characteristics, etc.) using in situ testing. The piezocone penetrometer is one of the most widely equipments used for in situ investigations and soil exploration. The piezocone penetrometer is capable of measuring simultaneously the cone tip resistance, sleeve friction, and pore pressure. These measurements can be effectively utilized for soil profiling and identification. The dissipation data can be used to evaluate the flow and consolidation characteristics of soils [5].

Today the piezocone test is generally regarded as a standard cone penetration test (CPT) with pore pressure measurements (CPTU). The main advantages of the CPTU over the conventional CPT are [5]:

- Ability to distinguish between drained, partially drained and undrained penetration.
- Ability to correct measured cone data to account for unbalanced water forces due to unequal end areas in cone design.
- Ability to evaluate flow and consolidation characteristics.
- Ability to assess hydrostatic conditions.
- Ability to improve soil profiling and identification.
- Ability to improve evaluation of geotechnical parameters.

Due to these capabilities, the CPTU has a wide range of applications in geotechnical engineering. Correlation between CPTU data and soil properties are generally used in two different ways including direct and indirect methods. First, for a given set of input soil properties, they can be used to calculate cone resistance for such purposes as liquefaction assessment or prediction of the end-bearing capacity of piles (direct method). Secondly, they are often used to back-calculate soil properties from measured CPTU records (indirect method) [5].

In direct approaches, the measured CPTU data are directly input into empirical formulas to provide estimates of foundation capacity and settlement [1], driven pile

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capacity [2] and drilled shafts or bored pile capacities [3], [4].

Considerable efforts have been made to drive soil-engineering properties from the results of CPT and CPTU data. Methodologies have been developed using empirical and statistical methods, back calculation, analytical studies, and numerical simulation. Various interpretation methods have been grouped into one of two basis categories: those specifically addressing a) Sands and cohesionless materials and b) Clays and cohesive materials [5].

Consequently, almost all correlations developed between *in situ* test measurements and soil design parameters have been concentrated on sands and clays. A thorough literature survey revealed that good quality piezocone data in silty soils together with good quality laboratory tests on obtained samples are not available in the geotechnical literature. For soils such as silty sands, limited studies have been accomplished toward improving the interpretation of CPT test results ([6], [7], [8]). In the pervious investigations, only silty sands with specific silt content have been tested. For example, Filho (1982) examined a well-graded medium to fine silty sand obtained from North Sea. Rahardijo et al. (1995) used Yatesville silty sand from the site of the Yatesville Lake – dam on Blaine Creek, which contained approximately 40% non-plastic fines. In other words, in previous studies, a complete set of laboratory tests have not been conducted to evaluate the influence of different silt contents on CPT results.

Experimental research project was begun at 2000 in advanced geotechnical center of Iran University of Science and Technology (IUST) to investigate the effect of silt content on CPT results in sandy soils [9]. The obtained results including the effect of silt content on the cone tip (q_c) and friction resistance (f_s) are presented [10], [11].

In the present research, the complimentary CPTU tests with pore pressure dissipation phase and soil laboratory tests (including consolidation, permeability and triaxial tests) are also performed on silty sand samples with different silt content. In this study, six piezocone tests are performed in saturated silty sand samples with several different silt contents ranging from zero to 50 percent. The pore pressure dissipation tests are also carried out in samples and t_{50} (the time for 50% pore pressure dissipation) is evaluated. Laboratory tests including consolidation, permeability and triaxial tests are also performed for the soil parameter determination. Based on the obtained results, the interrelationships among "K- t_{50}", "C_v - t_{50}" and "ϕ'-q_c" are finally presented.

2-EXPERIMENTAL PROCEDURES

A. *Piezocone Test*

The standard piezocone (according to ASTM D5778 [13]) is 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, the friction sleeve is located immediately behind the cone tip. The filter element to record pore water pressure is located immediately behind the cone tip. The piezocone is 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.

The testing chamber consists of a rigid thick walled steel cylinder of 0.76-m internal diameter and 1.50-m height, with removable top and bottom plates (Figure 1).

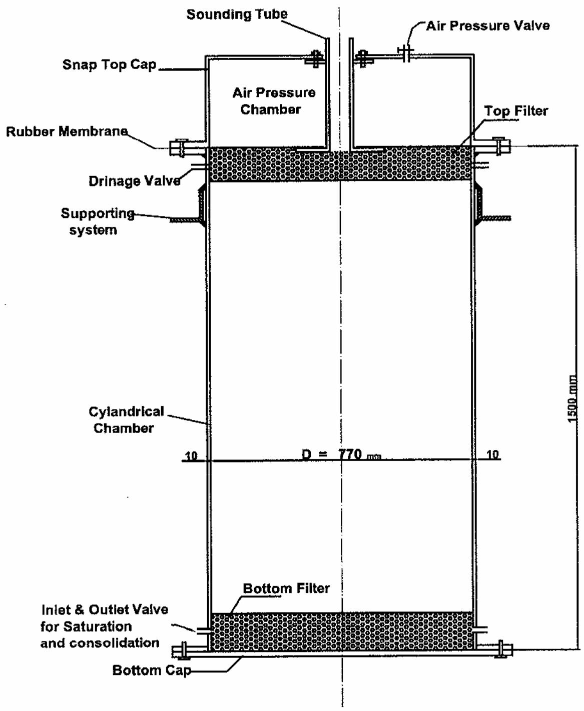
Given the diameter of a standard cone, 3.57 cm (ASTM D5778), the Rd (Chamber diameter/Cone diameter) is about 21. Due to the past studies [14], it can be resulted that with using a standard cone penetrometer in loose sandy soils, there will be no effect of boundary conditions for this calibration chamber. Therefore, the laboratory obtained results are compatible with field conditions.

All the CPTU tests reported in this research were conducted in six stages including: sample preparation, saturation, consolidation, CPT sounding, pore pressure dissipation test and evacuation [10]. Before filling the testing cylinder with dry sand, a soil filter grading from coarse sand to fine gravel is placed at the bottom. After the soil sample is setup, another filter layer is placed at the top of the soil.

The normal testing procedure consists of sample saturation, consolidation and cone penetration sounding. When the piezocone is pushed and reached at the midpoint of the sample, the pore pressure dissipation test.

To saturate the soil specimen, the top plate is fixed on the chamber and vacuum is applied inside the chamber for 30 minutes. Then, the bottom water supply is opened and as a result, the filter is flooded quickly, and a uniform slow upward flow is followed.

The normal testing procedure consists of sample saturation, consolidation and cone penetration sounding. When the piezocone is pushed and reached at the midpoint of the sample, the pore pressure dissipation test is performed. A dissipation test consists of stopping cone penetration and monitoring the decay of excess pore pressures (Δu) with time. From these data, an approximate value of the coefficient of consolidation (C_v) and permeability of soil (K) can be obtained.



Fig(1)- IUST Calibration Chamber

B. Laboratory Tests

To evaluate the flow parameters of soil specimens, the standard falling head permeability and odometer tests are performed at Imam Khomeini International University (IKIU) soil laboratory. Strain-controlled triaxial tests are also used to determine the internal friction angle of soil specimens.

C. Soil Types

To accomplish the objective of this study, soils with variable fines content (percent passing No. 200 sieve) were used. The fines content ranged from 10 to 50%. The silty sandy soils were prepared by mixing appropriate amounts of Tello (Eastern part of Tehran City) clean fine sand with low plasticity silt, obtained from grinding of this sand. This alluvial soil was fine clean sand without any clay or silt particle and had specific gravity of 2.6. The properties for the soils used during this study are shown in Table 1. It should be noted that the presented void ratio in Table 1 (e_f) and corresponding Dr are determined after consolidation. The grain size distribution curves for the soils are shown in Figure 2. The sand is a rounded to sub-angular fine-grained quartz sand with D_{50}=0.4 mm and C_u=3.0.

D. Samples 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. After the sample was encased in the membrane with the top cap, a vacuum of 10-20 kPa was applied and carbon dioxide gas percolated through the sample, which are then flushed with de-aired water, making it saturated. Degree of saturation was also checked by a B value greater than 0.98. After saturation stage in CPT tests, the vertical pressure was applied at the top of sample using the air pressure chamber and the sample was consolidated vertically at the zero laterally strained condition.

TABLE 1

PROPERTIES OF TESTED MATERIAL

Soil	F.C (%)	D_{50} (m)	C_U	e_{min}	e_{max}	e_f	Dr (%)
Clean Sand	0	0.40	30	0.746	1.05	0.989	20
Ts-10	10	0.38	5.6	0.625	1.0	0.931	18.4
Ts-20	20	0.34	7.5	0.594	0.97	0.897	19.3
Ts-30	30	0.32	7.4	0.572	0.92	0.855	18.7
Ts-40	40	0.24	6.9	0.52	0.895	0.825	18.6
Ts-50	50	0.075	5.5	0.485	0.875	0.798	19.5

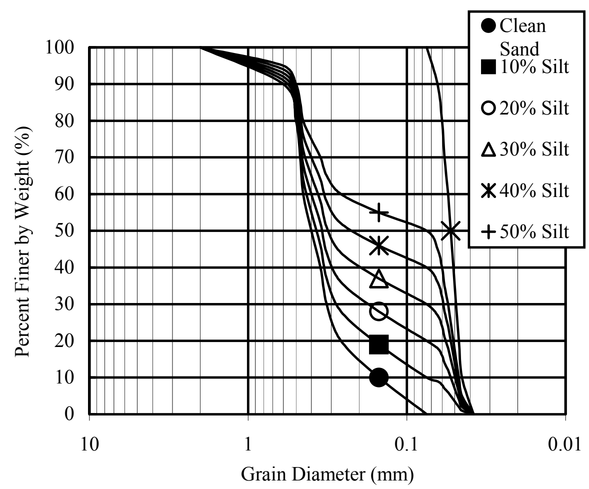


Figure 2: Grain size distribution curves

2. TEST RESULTS

A. Piezocone Test Results

Table 2 presents the summary of pore pressure dissipation (PPD) test results. Pore pressure dissipation curves for different silt contents are presented in Figure 3. As it is shown, the normalized excess pore water pressure ratio (u_e/u_{ei}, u_e = measured excess pore water pressure during time "t", u_{ei} = initial excess pore water pressure at "t=0") decreases with time elapsed in different silt content. The rate of decay depends on the coefficient of consolidation (C_v) and permeability (K) of the soil. Drainage conditions during a CPTU probe can be characterized by the parameter t_{50}, the time required for dissipation of 50 percent of initial excess pore pressure in a dissipation test made when piezocone penetration stopped. The value of t_{50} depends upon the coefficient of consolidation, which, in turn, depends on the compressibility, and permeability of the soil.

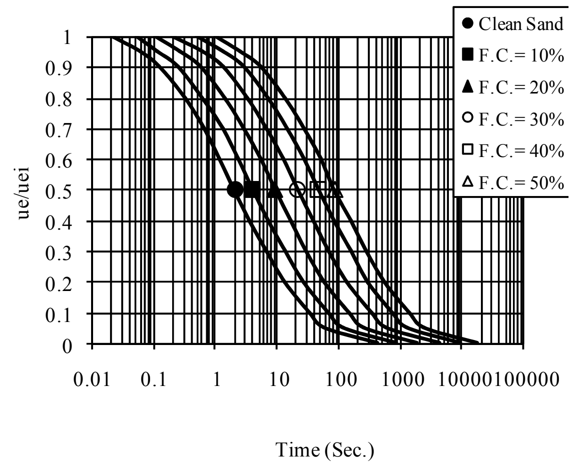


Figure 3: Pore pressure dissipation curves

In this research, the t_{50} parameter is determined for each sample containing different silt content.

The CPTU recordings at midpoint of each sample including cone tip resistance (q_c), excess pore water pressure (u_e) and friction resistance (f_s) are presented in Table 2 with the obtained t_{50} (the time for 50% pore pressure dissipation).The repeatability of test results was also controlled. Due to obtained results the maximum variation of test data are about 5% (Figure 4) and are acceptable.

TABLE 2
CPTU TEST RESULTS

Test No.	Type of Material	Silt Content	q_c (mPa)	U_e (kPa)	f_s (kPa)	t_{50} (sec)
1	Clean Sand	0	1.6	6	2	2
2	Silty Sand	10	1.4	14	1	4.
3		20	1.2	24	3	9
4		30	0.8	30	4	21
5		40	0.65	35	10	46
6		50	0.6	60	7	85

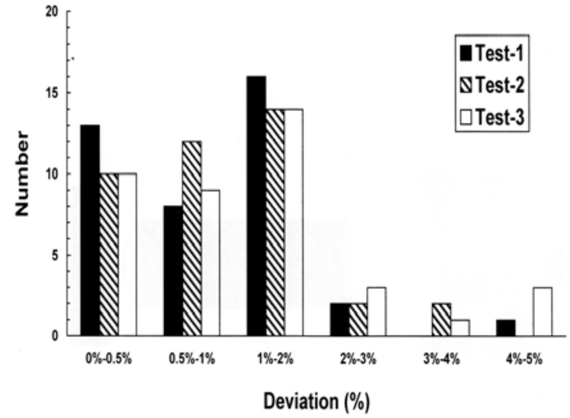


Figure 4: Repeatability of CPT test results

B. *Laboratory Test Results*

Coefficient of consolidation values (C_v) can be determined by comparing the characteristics of the experimental and theoretical curves in odometer tests. Permeability (K) of silty sand specimens was also determined using the falling head test method. Finally, the effective shear strength parameter of soil specimens (ϕ') were obtained from monotonic compression triaxial tests. The laboratory test results including falling head permeability, odometer and monotonic compression triaxial are presented in Table 3. The obtained laboratory results are explained and discussed completely in[15].

TABLE 3
LABORATORY TEST RESULTS

Test No.	Soil Type	Silt Content	C (cm^2/sec)	K (cm/sec)	ϕ'°
1	Clean Sand	0%	48.33	5×10^{-4}	32.2
2	Silty Sand	10%	17.77	2×10^{-4}	30.7
3		20%	8.11	1×10^{-4}	28.5
4		30%	3.30	4.5×10^{-5}	26.6
5		40%	1.31	2×10^{-5}	24.5
6		50%	0.69	8×10^{-6}	25.9

3. DETERMINATION OF PEAK ϕ'

The drained strength of sands can be expressed in terms of the Mohr-Coulomb criterion as a peak friction angle (ϕ'). Numerous methods for assessing ϕ' from cone resistance have been published. The methods fall into one of the following three categories:

- Empirical or semi-empirical correlations (mostly based on calibration chamber tests)
- Bearing capacity theory
- Cavity expansion theory

Empirical correlations are usually consisting of two methods including Dr-approach and Calibration chamber data. It is possible to estimate the peak friction angle (ϕ') based on knowledge of the relative density (D_r), soil gradation characteristics and the *in situ* stress level. Schmertmann (1978) proposed a relationship between the peak secant friction angle and relative density for different grain-size characteristics. The weakness with this approach is the approximate nature to which D_r can be estimated from CPT data. Experience has shown that the D_r – q_c correlations are sensitive to soil compressibility and in situ horizontal stresses.

A review of calibration chamber test results was made independently by Lunne and Christophersen (1983) and Robertson and Campanella (1983) to compare measured cone resistance (q_c) to measured peak friction angle (ϕ'). The peak friction angle values were obtained from drained triaxial compression tests performed at confining stresses approximately equal to horizontal stresses in the calibration chamber before cone penetration. Several separate theories of bearing capacity and wedge plasticity were evaluated in the light of calibration chamber test data from several quartz sands that compiled by Robertson & Campanella(1983). The expression for peak ϕ' from CPT is given by:

$$\phi' = \arctan [0.1 + 0.38 \log (q_c/\sigma'_{v0})]$$

An alternative expression has been proposed by Kulhawy & Mayne (1990):

$$\phi' = 17.6 + 11 \log (q_{c1})$$

where $q_{c1} = (q_c) / (\sigma'_{v0} / \sigma_{atm})^{0.5}$ (normalized cone tip resistance)

Bearing capacity solutions are generally based on assumed failure mechanisms, incompressible material, linear strength envelopes and plane strain conditions. The two main available bearing capacity solutions were developed by Janbo and Senneset (1982), Durgunoglu, and Mitchell (1975). Research by Mitchell and Keavany (1986) showed that the bearing capacity method provides reasonable prediction of ϕ' for most sands but underestimates for highly compressible sands.

The cavity expansion method developed by Vesic (1975), accounts for soil compressibility and volume change characteristics. Baligh (1976) developed this method further to incorporate the curvature of the strength envelope. Research by Mitchell and Keaveny (1986) has shown that the cavity expansion method appears to model the measured response extremely well and could predict the response in highly compressible sands. Unfortunately, the cavity expansion approach requires knowledge of soil stiffness and volumetric stain in the plastic region, both of which are difficult parameters to estimate or derive. Hence, the cavity expansion methods have not been used extensively.

Based on the above discussion, it is recommended that ϕ' for sands should be estimated empirically using calibration chamber method. This method is used at present research. For silty sand samples with different fine contents (0 to 50 %) the ϕ' values obtained by running monotonic drained compression triaxial tests. The cone tip resistance is also determined by using CPTU sounding data. Variation of peak ϕ' with silt content is presented in Figure 5.

As it is shown in Figure 5, in low percent of silt (0 to 40%), as the silt content increases, the peak ϕ' decreases. But in 50% silt content the peak ϕ' increases.

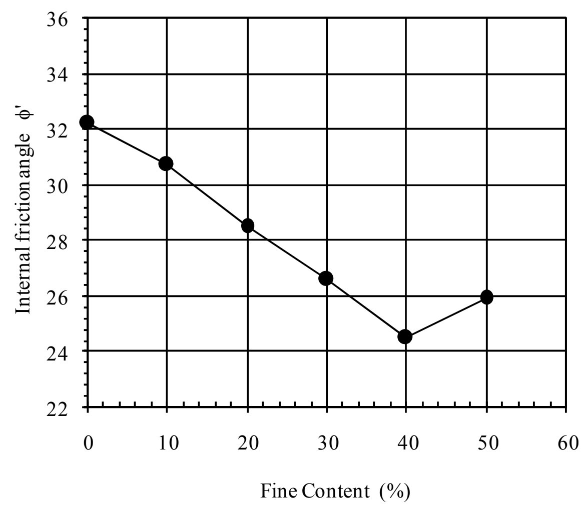


Figure 5: Peak ϕ' versus silt content

This phenomenon is observed and studied with many researchers in the past years such as Thevanayagam [24]. He found that the shear strength behavior of the silty sand specimens may be explained with the variation of intergranular (e_c) and interfine (e_f) void ratios. The variation of (e_f) and (e_c) against silt content which are obtained in this study are presented in Figure6.

In the low percent of silt (0 to 40%), the (e_f) has the big amount and the fine grains have the low effect on the shear strength of the soil. Therefore, the shear strength of silty sands is influenced by coarse grain contacts. As it is shown in Figure 6, as the silt content increases the (e_c) decreases and therefore the contacts of coarse grains and also the shear strength of soil are decreases. In high percent of silt (greater than 40%) the e_f becomes equal and smaller than e_c. Therefore, the contacts of fine grains and also the shear strength of soil increases.

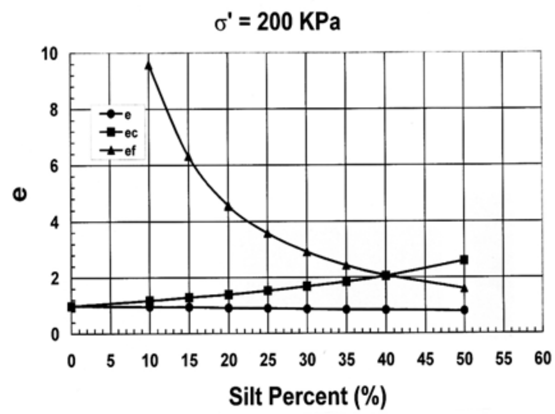


Figure 6: Variation of "e_c", "e_f" and "e" versus silt content

Due to obtained results it seems that the direct relationship between peak ϕ' and q_c can be evaluated only in low percent of silt. Hence, the correlation between (tan ϕ' – q_c) for 0 to 40% fine contents is presented in Figure 7.

The expression for peak ϕ' from CPTU in silty sands samples is given by:

$$\phi' = \arctan [0.12 + 0.412 \log (q_c/\sigma'_{v0})]$$

Figure 7 also shows that the peak ϕ' increases when the q_c increases. The effect of silt content on ϕ' determination is also considered in q_c parameters. As it is shown, the amount of silt content in sand is an important parameter affecting cone tip resistance. In low percent of silt (0 to 40%), as the silt content increases, both the cone tip resistance and the peak ϕ' decreases.

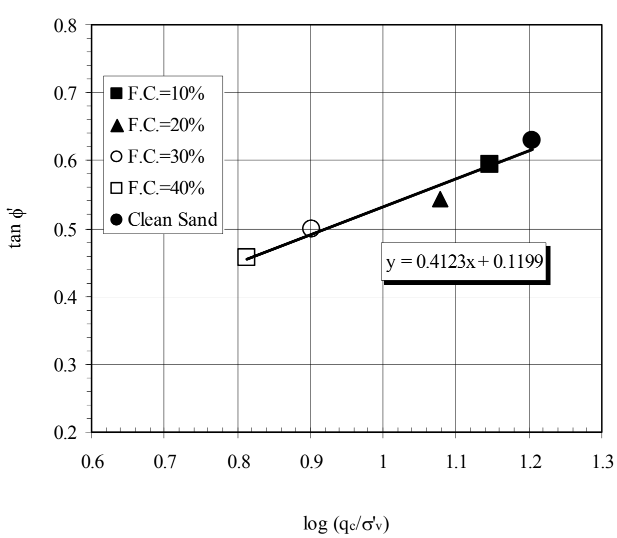


Figure. 7. Relationship between peak ϕ' and q_c

4. FLOW PARAMETER DETERMINATION

Flow and consolidation characteristics of soil are normally expressed in terms of the coefficient of consolidation, C_v, and hydraulic conductivity or permeability, K. These parameters vary over many orders of magnitude and are some of the most difficult parameters to measure in geotechnical engineering. Therefore, to avoid uncertainties, the field measurement techniques are recommended to determine these parameters. Results of pore pressure dissipation readings from piezocone tests can be used to determine permeability and coefficient of consolidation (Jamiolkowski et al., (1985)). Several methods of interpreting piezocone dissipation tests have been available for this purpose (e.g. [26]).

A. Coefficient of Consolidation

Rate of consolidation parameters may be assessed from the piezocone test by measuring the dissipation or decay of pore pressure with time after a stop in penetration. For interpretation, it is best to normalize the pore pressure relative to the initial pore pressure at the beginning of dissipation, u_i, and the equilibrium *in situ* pore pressure, u_0. The normalized excess pore pressure, U, at time t, is thus expressed as:

$$U = (u_t-u_0)/ (u_i-u_0)$$

Where:

u_t = the pore pressure at time t

u_i = initial pore pressure at time t = 0

u_0 = *in situ* pore pressure before penetration

Over the last decade, theoretical and semi-empirical solutions were developed for deriving the coefficient of consolidation from pore pressure dissipation data. Torstensson (1975) developed an interpretation model based on cavity expansion theories. He suggested that the coefficient of consolidation should be interpreted at 50% dissipation from the following formula:

$$C = (T_{50} / t_{50}).\ r_0^2$$

Where the time factor T_{50} is found from the theoretical solutions, t_{50} is the measured time for 50% dissipation and r_0 is the penetrometer radius.

Levadoux and Baligh (1986) who proposed an interpretation method after evaluating predictions of dissipation tests in Boston Blue Clay have performed a comprehensive study on pore pressure dissipation. They used the strain path method to predict the initial pore pressure distribution. A finite element method is used for the subsequent consolidation analysis.

Houlsby and Teh (1975) proposed an interpretation method based on the results of large strain finite element analysis of the penetration pore pressures, and a finite difference analysis of the dissipation pore pressures. They proposed the following equation for determination of C_v:

$$T^* = (C_v.t)/ (r^2.\sqrt{I_r})$$

Where:
T^* = modified time factor
C = coefficient of consolidation
r = radius of cone
I_r = rigidity index

Robertson et al. (1992) reviewed dissipation data from piezocone tests to predict coefficient of consolidation using Houlsby and Teh (1988) solutions with reference values from laboratory tests and field observations. The review showed that the Houlsby and Teh (1988) solution provided reasonable estimates of C_v.

Based on available experience, the Houlsby and Teh (1988) procedure is recommended to estimate the coefficient of consolidation. At present research, the coefficient of consolidation of tested samples with different silt contents were evaluated from laboratory consolidation tests. Then the t_{50} values were determined for each sample from pore pressure dissipation curves. Finally, the correlation between t_{50} and C_v are presented in Figure 8.

The expression for C from CPTU in silty sands samples is given by:

$$C_v = 94.32 / (t_{50}^{1.11})$$

The theoretical curve of C_v which is obtained from Houlsby and Teh method (Ir =500 and T_{50} = 0.245) is also compared with experimental data in Figure 9. The good agreement is shown between experimental data and theoretical solution. Only the in accordance exists in clean sand sample due to rapid drainage condition.

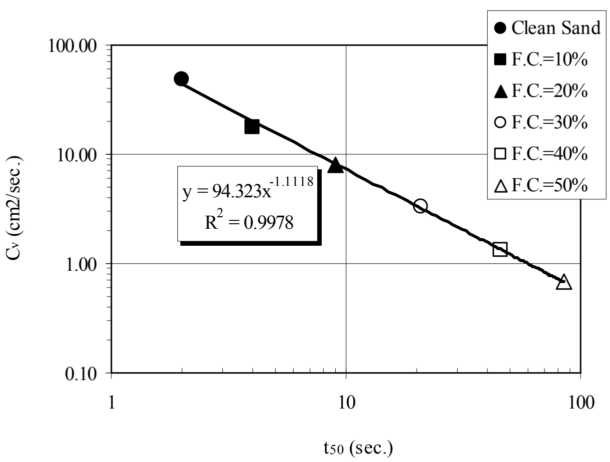


Figure. 8. Relationship between C_v and t_{50}

B. *Coefficient of Permeability (hydraulic conductivity)*

Soil permeability can be estimated as a function of t_{50}. Parez and Fauriel (1988) presented a summary of available data from dissipation tests (t_{50}) and laboratory determined K (Figure 10).

They proposed an average relationship approximately expressed by:

$$K\ (cm/sec) = [1/ (251.t_{50})]^{1.25}$$

Where t_{50} is given in seconds.

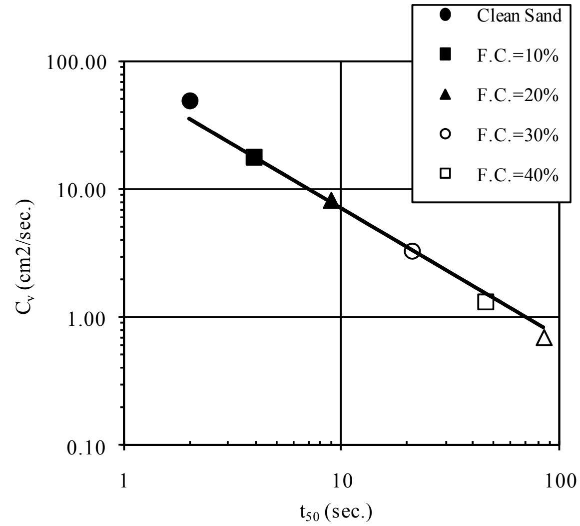


Figure. 9. Comparison between experimental and theoretical C_v (Houlsby & Teh method)

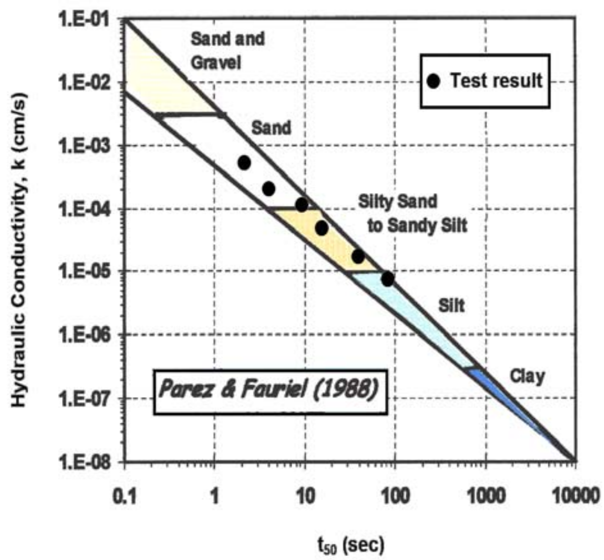


Figure. 10. Comparison between obtained permeability results and Parez & Fauriel (1988) data

At present research, the coefficients of permeability of tested samples with different silt contents evaluated from falling head permeability tests. The t_{50} values are also determined for each sample from pore pressure dissipation curves. Then, the correlation between t_{50} and K are presented in Figure 8. It is shown that the permeability coefficient decreases as the t_{50} increases. Therefore, the following expression may be drawn for determination of K from CPTU in silty sands samples:

$$K = [1 / (720.t_{50})]^{1.05}$$

Where t_{50} is given in second.

The comparison of the obtained results and Parez and Fauriel data are presented in Figure 10. As it is shown, the obtained results are approximately compatible with proposed silty sand to sandy silt region. However, there is some difference in fine content smaller than 20% due to rapid drainage condition.

5. CONCLUSIONS

In this study, six piezocone tests with pore pressure dissipation phase were performed in saturated silty sand samples with several different silt contents ranging from 0 to 50 percent and t_{50} (the time for 50% pore pressure dissipation) values were evaluated. Laboratory tests including consolidation, permeability and triaxial tests were also performed for soil parameter determination. Based on the obtained results, the interrelationships between "K-t_{50}", "C-t_{50}" and "ϕ-q_c" were presented. From the results of this study, the following conclusions can be drawn:

1. In tested silty sand samples with low percent of silt (0 to 40%), as the silt content increases, the peak ϕ' decreases. But in 50% silt content, due to high consolidation behavior of soil, the global void ratio decreases and the peak ϕ' increases. Therefore, it seems that the direct relationship between peak ϕ' and q_c can be evaluated only in low percent of silt.
2. The amount of silt content in sand is an important parameter affecting cone tip resistance. In low percent of silt (0 to 40%) as the silt content increases, the cone tip resistance decreases and also the peak ϕ' decrease.
3. The coefficient of consolidation (C_v) of silty sand samples can be determined from PPD test results as follows:

$$C_v = 94.32 / (t_{50}^{1.11})$$

4. The good agreement between experimental data and theoretical solution is shown in determination of C_v. Only the inaccordance exists in clean sand sample due to rapid drainage condition.
5. Due to obtained results, it can be concluded that, the permeability coefficient decreases as the t_{50} increases. Also, the following expression may be drawn for determination of K from CPTU in silty sands samples:

$$K = [1 / (720.t_{50})]^{1.05}$$

6. The comparison of the obtained permeability results and Parez and Fauriel data shows that, the obtained results are approximately compatible with proposed silty sand to sandy silt regions of this graph. However, there are some differences for samples with fine content smaller than 20% due to rapid drainage condition.

6. ACKNOWLEDGMENTS

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