Using Miniature Cone Penetration Test (Mini-CPT) to determine engineering properties of sandy soils

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Abstract

In-situ testing techniques have proven to be successful in improving the speed and reliability of geotechnical investigations. One of the most common in-situ methods in engineering geology and site investigation is Cone Penetration Test (CPT), which is mainly used for characterization of soils, as it is a robust, simple, fast, reliable and economic test that can provide continuous soundings of subsurface soil. Miniature Cone Penetration (Mini-CPT) Test is a new type of CPT but in diameter less than conventional CPT to determine the bearing capacity and strength parameters of loose to semi-dense soils at shallow depth. Mini-CPT needs lower force to penetrate into the soil, and its ability to identify very thin underneath layers is higher than CPT. In this research, a Mini-CPT apparatus was used in laboratory conditions to define the relationships between tip resistance ($q_{c(MCPT)}$), friction resistance ($f_{s(MCPT)}$) and some engineering properties of poorly graded sandy soils such as Relative Density (D_r), Friction Angle (φ), Elastic Modulus (E), Shear Modulus (G), and the Modulus of Subgrade Reaction (K_s) with different densities. Based on the results of the experiments, the relationships between both the $q_{c(MCPT)}$ and $f_{s(MCPT)}$ with engineering properties were obtained with a high determination coefficient (R^2>0.85).

Keywords: *Engineering Properties, Miniature Cone Penetration Test (Mini-CPT), Poorly Graded Sandy Soils*

Introduction

In soil exploration, a modern and expedient approach is offered by cone penetration testing (CPT), which involves pushing an electronic penetrometer instrument into the soil and recording multiple measurements continuously with depth (Schmertmann, 1978; Briaud & Miran, 1992). By using ASTM-D5778-95 (2003), three separate measurements of tip resistance (q_c), sleeve friction (f_s), and pore-water pressure (u) are obtained with depth. In its simplest application, the cone penetrometer offers a quick, expedient and economical way to profile the subsurface soil layering at a particular site. No drilling, soil samples or spoils are generated; therefore, CPT is less disruptive from environmental standpoint. The continuous nature of CPT readings permits clear explanations of various soil strata as well as their depths, thicknesses and extent perhaps better than conventional rotary drilling operations. A variety of cone penetrometer systems is available, ranging from small mini-pushing units to very large trucks. The electronic penetrometers range in size from small to large probes, from one to five separate channels of measurements (TRBNA, 2007). Miniature cone penetrometers are available with the reduced cross-sectional sizes of 5 cm^2 and 1 cm^2 (Tumay *et al.,* 1998).

The most important advantages of miniature penetrometer are:

- Smaller downward thrust needed to advance the penetrometer into the soil (Tumay *et al.,* 1998).
- Ability to identify very thin lenses (Meigh, 1987).
- Installation in a smaller vehicle that provides greater mobility and site accessibility (Tufenkjian & Thompson, 2005).

The Mini-CPT has been used for identification of geotechnical properties of near surface seafloor soils during the installation of military seafloor cable systems in sands (Tufenkjian & Thompson, 2005). Earliest versions of sounding were developed in 1917 by the Swedish State Railways, then by the Danish railways in 1927 (Meigh, 1987). Initial cone systems were the mechanical-type design with two sets of rods. An outer set of steel rods was employed to minimize soil friction and protect an inner stack of rods that transferred tip forces inside the hole into a pressure gauge read-out at the ground surface (TRBNA, 2007). A friction sleeve to measure local skin friction over a short length above the cone was then introduced in Indonesia (Begemann, 1965). The electric penetrometer was first introduced in 1948 (Meigh, 1987). As early as 1962, a research piezocone was designed for tip and pore-water readings by the Delft Soil Mechanics Laboratory (Vlasblom, 1985).

The combination of the electric cone with the electric piezoprob was an inevitable design; as the hybrid piezocone penetrometer could be used to obtain three independent readings during the same sounding: tip stress, sleeve friction and pore-water pressures (Baligh *et al.,* 1981). Mini-CPT is a new type of CPT with a projected cone area of 2 cm^2, which gives finer details than the standard 10 cm^2 cross-section area reference cone penetrometer. Continuous Intrusion Miniature Cone Penetration Test (CIMCPT) may be used for rapid, accurate and economical characterization of sites and to determine the engineering parameters of soil, which are needed in the design of pavements, embankments and earth structures. The maximum depth of penetration that can be achieved by the CIMCPT system is 12 m (Tumay *et al.,* 1998).

The main objective of this paper is to describe the capability of the Mini-CPT to study the engineering properties of sandy soils. We used a penetrometer with a diameter of 1.6 cm (projected area of 2 cm^2) in laboratory conditions. In addition, several testes such as Plate Load Test (PLT) and direct shear test were conducted to determine the deformability and strength characterizations of soil sample.

Geotechnical characteristics of the tested soils

In order to achieve the appropriate correlation between the Mini-CPT results and the engineering properties of sandy soils, it was necessary to select suitable samples. The appropriate sampling area was selected based on our previous experiences, and the sampling was performed according to the standard methods. The sampling area contains four types of lithology belonging to late Eocene and Quaternary deposits (Fig. 1).

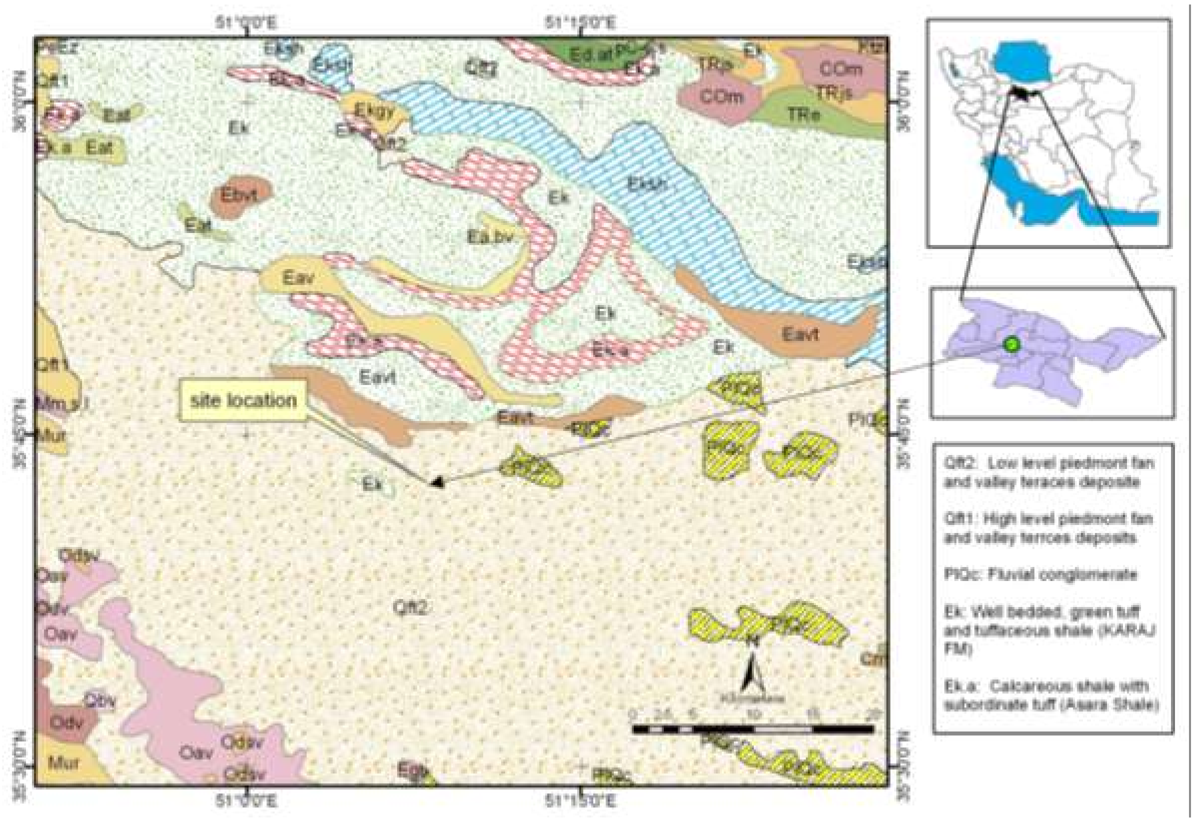

Figure 1: Geological map of the sampling location

The late Eocene rock covers almost 5% of the land surface, and comprises one type of lithology, which is categorized as Trachyte to Trachyandesite. The Quaternary deposits and Sedimentary rocks cover almost 95% of the sampling area and comprise three types of lithology, which are categorized as conglomerate, old terrace deposits and young terrace deposits. In the present research, the sampling was performed on the young terrace deposits. Geologically, the young deposits comprise sub-rounded sand grains containing 5% gravel. The X-ray analysis has showed that the sandy samples are made of quartz, feldspar, pyroxene and calcite.

To prepare the testing samples, alluvial deposits were oven-dried and passed through sieve No. 4. Fig. 2 shows the gradation curve of the sample after passing sieve No. 4, which is classified as poorly graded sand (SP) according to the Unified Soil Classification System (USCS). The index properties of the soil are shown in Table 1. To obtain a uniform compaction, the sample in the testing mould was compacted in several 100 mm thick layers. The compaction effort for dry soil was applied using a 300 mm vibrating plate in a way that the required density was achieved. The in-place density for each soil layer was controlled using the sand cone method. For the sand cone method, the hole from which the soil sample was removed was filled with dry sand from a graduated bottle. The sand has a uniform known density, so its dry weight volume is then known (see Fig. 3). Details of the tests on the samples with different densities are shown in Table 2.

Table 1: The index properties of used soil

Parameter	Value
$e_{max(-)}$	0.97
$e_{min(-)}$	0.46
$G_{s(-)}$	2.66
γ_d(max)(KN/m^3)	17.85
γ_d(min)(KN/m^3)	13.24
Cu(-)	1.16
Cc(-)	1
Value of clay (%)	0
Value of silt (%)	2
USCS soil classification	sp

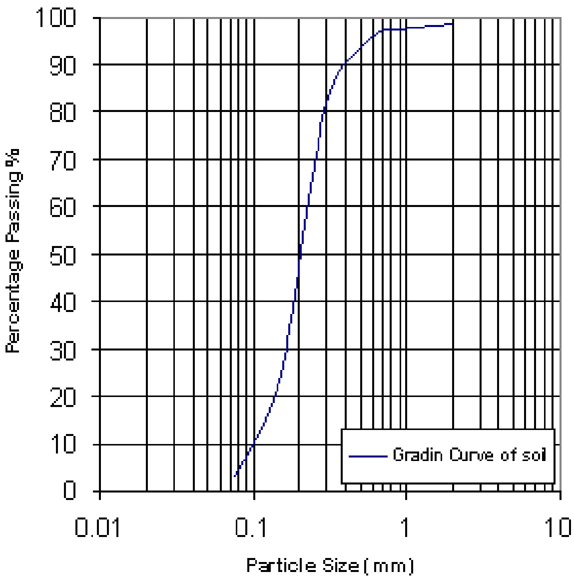

Figure 2: Gradation curve of the used soil

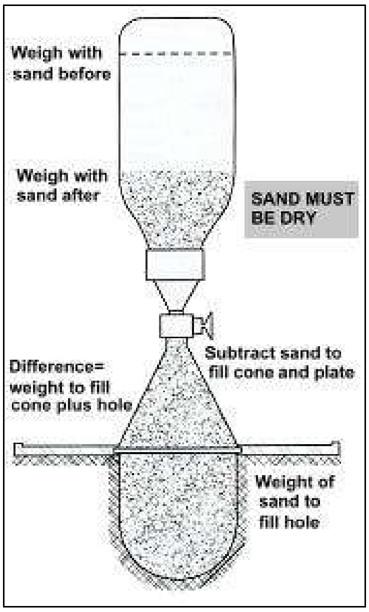

Figure 3: Schematic diagram of sand cone method

Testing procedure

Several tests including Mini-CPT, plate load and direct shear were carried out to determine the engineering properties of sandy soils. Details of the tests on the samples with different densities are indicated in Table 2.

Mini-CPT test

In this research, Mini-CPT$_s$ were carried out to the depth of approximately 1m in an especial designed circular mould. Two miniature cone penetrometers with a projected cone area of 2 cm^2, friction sleeve area of 43 cm^2, and a cone apex angle of 60° were used (Fig. 4). They both were of the subtraction type that measures either the cone resistance or combined cone resistance plus local sleeve friction resistance. In the latter state, combined cone resistance plus the local sleeve friction resistance must be subtracted from the local sleeve friction resistance. The ASTM-D3441 (2004) standard method was followed to perform the tests. Mini-CPT$_s$ were done on the samples with different relative densities (25%, 35%, 50%, 60% and 75%) and repeated three times. The results for mean $q_{c(MCPT)}$ and $f_{s(MCPT)}$ are shown in Fig. 5a, b, respectively. Tip resistance ($q_{c(MCPT)}$) and friction resistance ($f_{s(MCPT)}$) are shown in MPa and kPa, respectively. It is to be noted that the presented results for $q_{c(MCPT)}$ and $f_{s(MCPT)}$ are the average of resistances in different depths. The results of these determinations are given in Table 3.

Table 2: Testing program for the laboratory investigations, and different densities for the tested soil

D_r(%)	Mean of water content (%)	Dry unit weight (gr/cm^3)	Mini-CPT (number of tests for f_s and q_c)	PLT (number of tests)	Direct shear (number of tests)
25	**0.4**	**1.44**	**6**	**3**	**3**
35	**0.4**	**1.48**	**6**	**3**	**3**
50	**0.4**	**1.55**	**6**	**3**	**3**
60	**0.4**	**1.60**	**6**	**3**	**3**
75	**0.4**	**1.67**	**6**	**3**	**3**

Table 3: Summary of all the results

Dr%		φ (Deg)	$q_{c(MCPT)}$ (MPa)	$f_{s(MCPT)}$ (kPa)	$E_{PLT(i)}$ (MPa)	$E_{PLT(R2)}$ (MPa)	$G_{PLT(i)}$ (MPa)	k_s (MN/m^3)
25	**1**	**29.5**	**1.023**	**31.96**	**6**	**2.5**	**1.5**	**26**
	2	**32**	**1.052**	**36.27**	**7.5**	**2.9**	**1.9**	**27**
	3	**33**	**1.062**	**40.73**	**8**	**3.8**	**2.2**	**28**
35	**1**	**32**	**2.740**	**42.62**	**9.3**	**4.1**	**2.5**	**46**
	2	**34.1**	**2.905**	**63.13**	**9.5**	**5**	**2.6**	**48**
	3	**34.2**	**3.308**	**71.90**	**10**	**5.5**	**3**	**49**
50	**1**	**35.5**	**3.712**	**63.25**	**13**	**6**	**4.3**	**100**
	2	**37**	**4.325**	**66.50**	**15**	**8.5**	**5**	**100**
	3	**39**	**4.295**	**111.03**	**15**	**9**	**5.2**	**125**
60	**1**	**39**	**6.512**	**157.58**	**15.5**	**10.5**	**9.5**	**120**
	2	**39**	**8.058**	**185.92**	**15.5**	**11**	**10**	**120**
	3	**39.5**	**8.145**	**214.93**	**16**	**11.5**	**11**	**130**
75	**1**	**41**	**10.187**	**427.46**	**20**	**14.2**	**14.5**	**162**
	2	**42**	**10.537**	**492.90**	**22.5**	**15**	**18**	**178**
	3	**43.5**	**12.200**	**499.65**	**24.9**	**15.8**	**19**	**200**

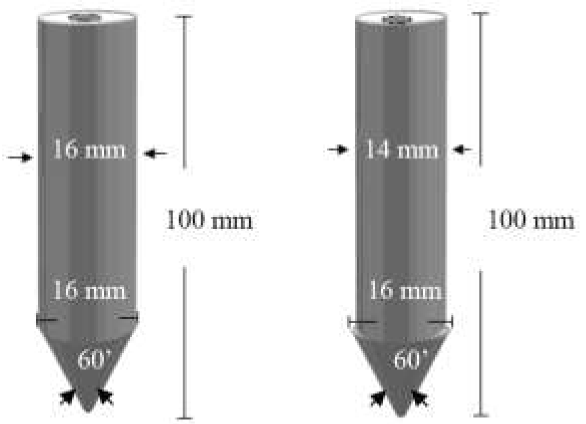

Figure 4: Miniature cone pentrometers

Plate Load Test (PLT)

Plate Load Test (PLT) is a common site investigation tool, which has been used for proof testing of pavement layers in the European countries for many years. Currently, it is also for evaluation of both rigid and flexible pavements (Abu-Farsakh *et al.,* 2004). The PLT in full or small scale is sometimes considered as the best means of determining the deformation characteristics of the soils; however, it is only used in exceptional cases due to the costs involved (Bowles, 1997). In the present research, a round plate with 230 mm diameter was used. PLT was used as a reference test to obtain the strength parameters of the soil under investigation. A loading frame was designed to fit the mould and its support. To perform the test, the bearing plate and the hydraulic jack were carefully placed at the center of the sample under the loading frame (Fig. 6). The hydraulic jack and the supporting frame were able to apply a 60 ton load. For measurement of deformations, dial gauges that are capable of recording a maximum deformation of 25.4 mm (1 in) with an accuracy of 0.001 in, were used. The ASTM-D 1195-93 (1998) standard method was followed to perform the test.

Elasticity modulus is always considered as an important deformability parameter for geomaterials. Based on stress-strain curve, different elasticity modulus can be defined as: 1) the initial tangent modulus 2) the tangent modulus at a given stress level 3) reloading and unloading modulus and 4) the secant modulus at a given stress level. In this study, since the stress–strain curves had a clear peak, the initial tangent modulus was determined for all PLT results. To determine the initial modulus ($E_{PLT(i)}$), a line was drawn tangent to the initial segment of the stress–strain curve. Then an arbitrary point was chosen on the line, and the stress and deflection corresponding to this point were determined for calculating of the initial modulus. Fig. 7 describes the settlement and stress used for determining $E_{PLT(i)}$ (Abu-farsakh *et al.,* 2004). A reloading stiffness modulus called $E_{PLT(R2)}$, was also determined for each stress–strain curve.

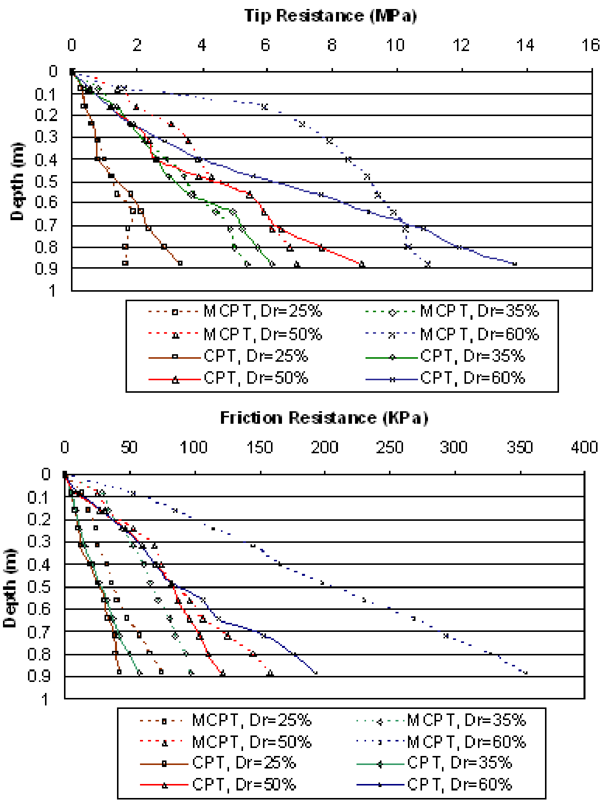

Figure 5: a) Miniature cone tip resistance ($q_{c(MCPT)}$) b) Miniature cone friction resistance ($f_{s(MCPT)}$) profiles in the sands with different relative densities

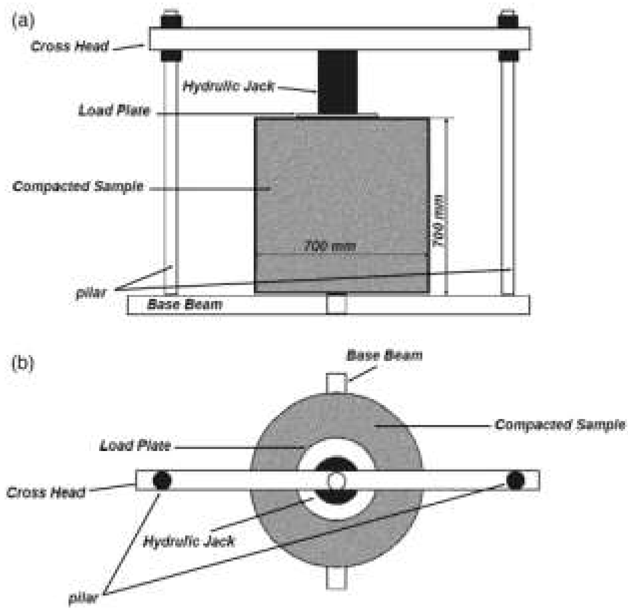

Figure 6: A schematic diagram of plate load test (PLT) set up: (a) side view, and (b) plan view

The second parameter, which can be calculated from the PLT results, is shear modulus (G). Shear modulus is defined as the ratio of shear stress to shear strain (Bowles, 1997) and is calculated from equation (1) (Timoshenko & Goodier, 1970):

$$G_{PLT} = \frac{qD\pi}{\rho 8}(1-\upsilon) \qquad (1)$$

Where, q is the bearing pressure, D is the diameter of the loading plate, ρ is the settlement and υ is the Poisson ratio.

From the theory of elasticity, the relationship between the modulus of elasticity and the shear modulus can be given as below:

$$E=2(1+\upsilon)/G \qquad (2)$$

Since the non-rigid methods consider the effect of local mat deformations on the distribution of bearing pressure, it is needed to define the relationship between settlement and bearing pressure. This is usually done using the coefficient of subgrade reaction (K_s). Equation (3) is used to determine K_s from the PLT results (Coduto, 2001):

$$k_s= \Delta P/\Delta S \qquad (3)$$

Where, K_s is the modulus of subgrade reaction, ΔP is the applied pressure and ΔS is the measured settlement. The results of these determinations are given in Table 3.

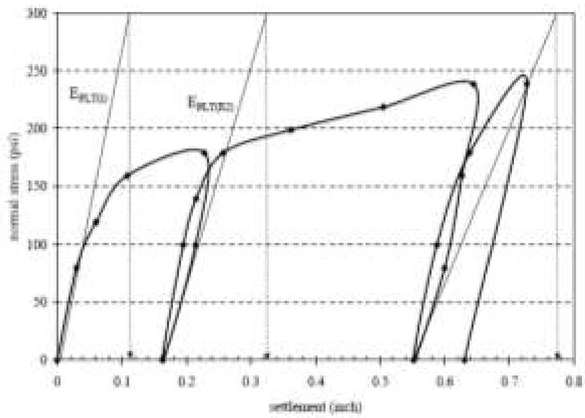

Figure 7: Definition of modulus from PLT (Abu-farsakh *et al.,* 2004)

Direct shear test

In order to determine the soil friction angle (φ), 15 direct shear tests were carried out in a circular shear mould according to ASTM-D 308-90 (2000) (see Table 2). To prepare the soil samples for direct shear test, a circular shear box, having 60 mm internal diameter and 25 mm height, was used. To achieve a uniform compaction in the circular shear mould of the direct shear machine, tamping by a small circular steel plate with 60 mm diameter was used.

Due to the nature of the soil samples (non-cohesive), cohesion parameter (C) was equal to zero, and thus friction angles were calculated. To eliminate the effect of pore pressure, all direct shear tests were carried out in dry conditions. The results of direct shear tests are given in Table 3.

Results and discussion

One of the most commonly accepted methods of investigating empirical relationships between soil properties is simple regression analysis. In this study, the results of all tests were assessed to find the best correlation between Mini-CPT strength parameters ($q_{c(MCPT)}$ and $f_{s(MCPT)}$) and the index properties of the soil samples, i.e. Relative Density (D_r), Friction Angle (φ) Elastic Modulus (E), Shear Modulus (G) and Subgrade Reaction Modulus (K_s). The linear and non-linear simple regressions were undertaken with 95% confidence level and the determination coefficient (R^2) was obtained for the relationships. The authors attempted to develop the best correlation between different variables in order to attain the most reliable empirical equation.

Mini-CPT parameters versus relative density (D_r)

Relative density is a useful parameter to describe the consistency of sands. CPT can be used to estimate both the relative density of cohesionless soil and the undrained strength of cohesive soils through empirical correlations (USACE, 1992). Several investigators including Schmertmann (1978), Villet & Mitchell (1981), Baldi *et al.,* (1982), Robertson & Campanella (1983), Jamiolkowski *et al.,* (1988), Puppala *et al.* (1995) and Juang *et al.,* (1996) have developed correlations for the relative density (D_r) as a function of q_c for sandy soils. These relationships are also functions of vertical effective stress (Amini, 2003).

In this study, the correlation of D_r with $q_{c(MCPT)}$ and $f_{s(MCPT)}$ was investigated. (Fig. 8) show the relationships of D_r with $q_{c(MCPT)}$ and $f_{s(MCPT)}$ for the tested samples, respectively. A very high correlation with the determination coefficient 0.96, was found between D_r and $q_{c(MCPT)}$:

$D_r(\%)=23.733\ (q_{c(MCPT)})^{0.469}$ $R^2=0.96$ (4)

Similarly, a power relationship was observed between D_r and $f_{s(MCPT)}$ with a high determination coefficient 0.84:

$D_r(\%)=7.408\ (f_{s(MCPT)})^{0.389}$ $R^2=0.85$ (5)

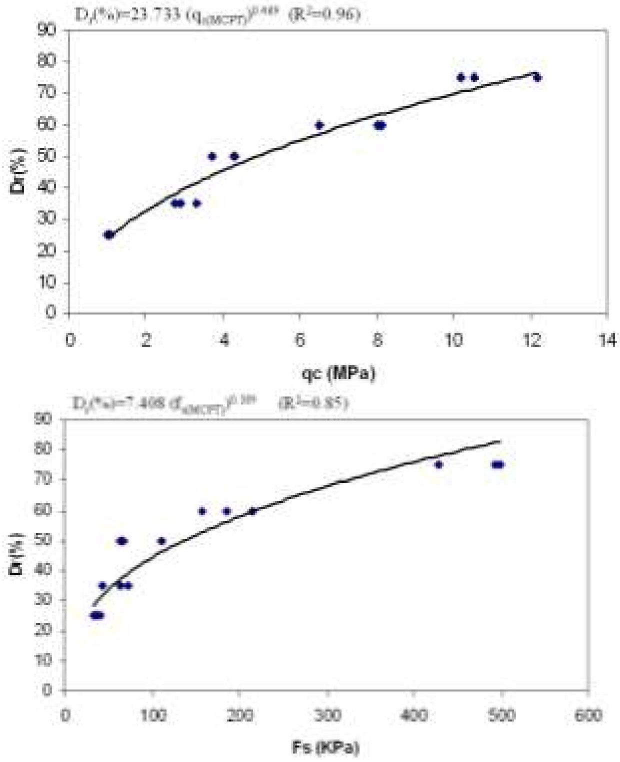

Figure 8: Correlation between a) $q_{c(MCPT)}$ and b) $f_{s(MCPT)}$ with D_r

According to these results, for estimation of D_r, using $q_{c(MCPT)}$ is more reliable than $f_{s(MCPT)}$ because of higher determination coefficient. The application range of the equations obtained in this study is 1.023-12.2 MPa for $q_{c(MCPT)}$, and 31.96-499.65 kPa for $f_{s(MCPT}$.

Mini-CPT parameters versus friction angle (φ)

Friction angle is one of the most important index parameters needed for calculation of shear strength in any foundation design.

(Fig. 9) depict that good correlations were found between friction angle (φ), and $q_{c(MCPT)}$ and $f_{s(MCPT)}$ with determination correlations of 0.87 and 0.89, respectively:

$\varphi=30.5(q_{c(MCPT)})^{0.127}$ $R^2=0.87$ (6)

$\varphi=21.5(f_{s(MCPT)})^{0.113}$ $R^2=0.89$ (7)

As can be seen from these Figs., also equations (6) and (7), in each case, the best fit relation is represented by power regression curves. An increase in friction angle was recorded following an increase in both $q_{c(MCPT)}$ and $f_{s(MCPT)}$.

The correlation between friction angle (φ) and relative density (D_r) for the results obtained in this research is presented in Fig. 10 and equation (8):

$$\varphi = 0.21(D_r)+26 \qquad R^2=0.92 \qquad (8)$$

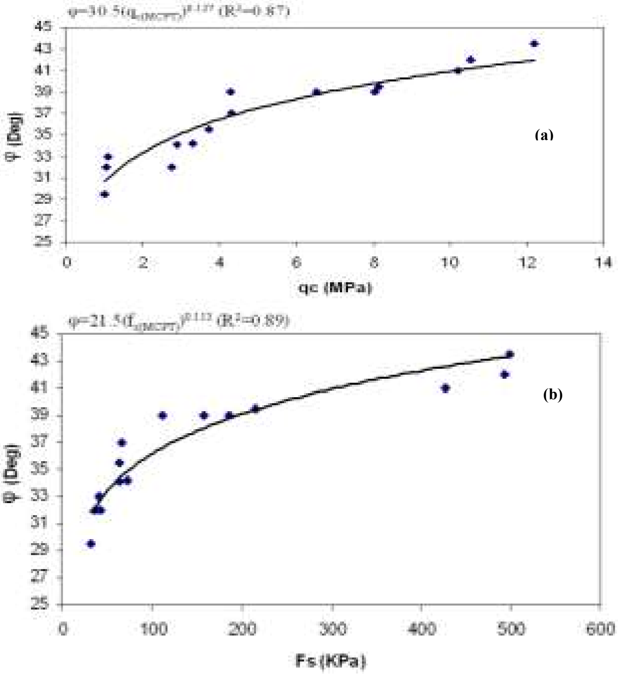

Figure 9: Correlation between a) $q_{c(MCPT)}$ and b) $f_{s(MCPT)}$ with φ

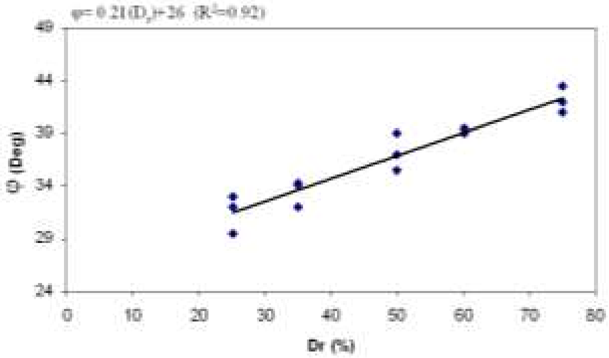

Figure 10: Correlation between D_r and φ

There is a linear relationship between friction angle (φ) and relative density (D_r) with a high determination coefficient (0.92).

Similar relationships have been obtained between friction angle (φ) and relative density (D_r) by different authors such as Meyerhof (1959). He has suggested equation (9) for normally consolidated sands:

$$\varphi=0.15(D_r) \qquad (9)$$

Mini-CPT parameters versus elastic modulus (E)

The cone penetration resistance has been correlated with the equivalent elastic modulus of soils by various investigators (Trofimenkov, 1974; Schmertmann *et al.,* 1986).

Table 4: Summary of the equations developed in this paper

Parameters	Equations	Type correlation	Determination coefficient (R^2)
D_r-$q_{c(MCPT)}$	$D_r(\%)=23.733\ (q_{c(MCPT)})^{0.469}$	power	(R^2=0.96)
$D_r(\%)$-$f_{s(MCPT)}$	$D_r(\%)=7.408\ (f_{s(MCPT)})^{0.389}$	power	(R^2=0.85)
$E_{PLT(i)}$-$q_{c(MCPT)}$	$E_{PLT(i)}=1.457(q_{c(MCPT)})+6.07$	linear	(R^2=0.92)
$E_{PLT(R2)}$-$q_{c(MCPT)}$	$E_{PLT(R2)}=1.2(q_{c(MCPT)})+2$	linear	(R^2=0.96)
$E_{PLT(R2)}$-$f_{s(MCPT)}$	$E_{PLT(R2)}=4.6\ Ln(f_{s(MCPT)})-13$	logarithmic	(R^2=0.97)
$E_{PLT(i)}$-$f_{s(MCPT)}$	$E_{PLT(i)}=5.56\ Ln(f_{s(MCPT)})-12$	logarithmic	(R^2=0.91)
$G_{PLT(i)}$-$q_{c(MCPT)}$	$G_{PLT(i)}=1.37(q_{c(MCPT)})^{0.97}$	power	(R^2=0.92)
$G_{PLT(i)}$-$f_{s(MCPT)}$	$G_{PLT(i)}=0.1(f_{s(MCPT)})^{0.869}$	power	(R^2=0.94)
k_s-$q_{c(MCPT)}$	$k_s=25(q_{c(MCPT)})^{0.817}$	power	(R^2=0.92)
k_s-$f_{s(MCPT)}$	$k_s=57Ln(f_{s(MCPT)})-169$	logarithmic	(R^2=0.90)
φ-$q_{c(MCPT)}$	$\varphi=30.5(q_{c(MCPT)})^{0.127}$	power	(R^2=0.87)
φ-$f_{s(MCPT)}$	$\varphi=21.5(f_{s(MCPT)})^{0.113}$	power	(R^2=0.89)
φ-D_r	$\varphi= 0.21(D_r)+26$	linear	(R^2=0.92)

In Fig. 11, the correlations of $q_{c(MCPT)}$ with the loading and reloading elastic modulus ($E_{PLT(i)}$ and $E_{PLT(R2)}$) are presented for the data obtained in this study (also see equations 10 and 11). It can be seen from the figures that the best-fitted relations are represented by linear regression curves. The results of regression equations and the determination coefficients are summarized in Table 4.

$$E_{PLT(i)}=1.457(q_{c(MCPT)})+6.07 \qquad R^2=0.92 \qquad (10)$$

$$E_{PLT(R2)}=1.2(q_{c(MCPT)})+2 \qquad R^2=0.96 \qquad (11)$$

A strong logarithmic correlation was also found between $f_{s(MCPT)}$, and loading and reloading elastic modulus ($E_{PLT(i)}$ and $E_{PLT(R2)}$) with the determination coefficients of 0.91 and 0.97, respectively (Fig. 12):

$$E_{PLT(i)}=5.56\ Ln\ (f_{s(MCPT)})-12 \qquad R^2=0.91 \qquad (12)$$

$$E_{PLT(R2)}=4.6\ Ln(f_{s(MCPT)})-13 \qquad R^2=0.97 \qquad (13)$$

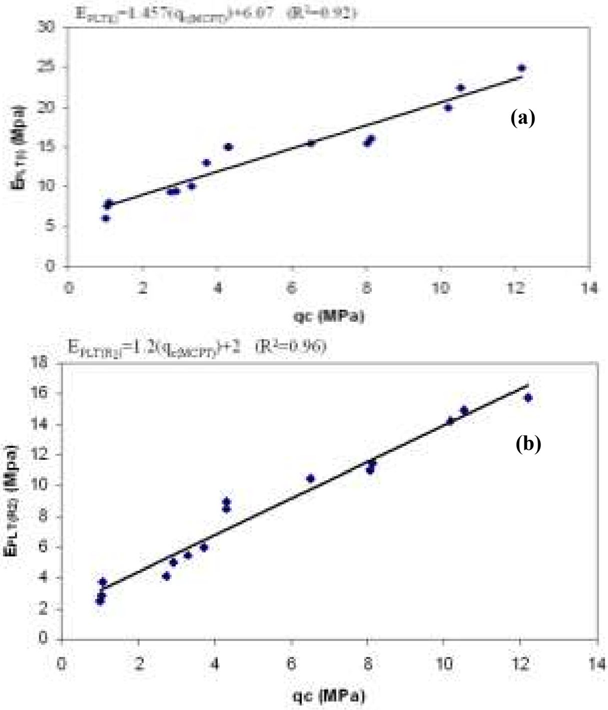

Figure 11: Correlation between a) $E_{PLT(i)}$ and b) $E_{PLT(R2)}$ with $q_{c(MCPT)}$

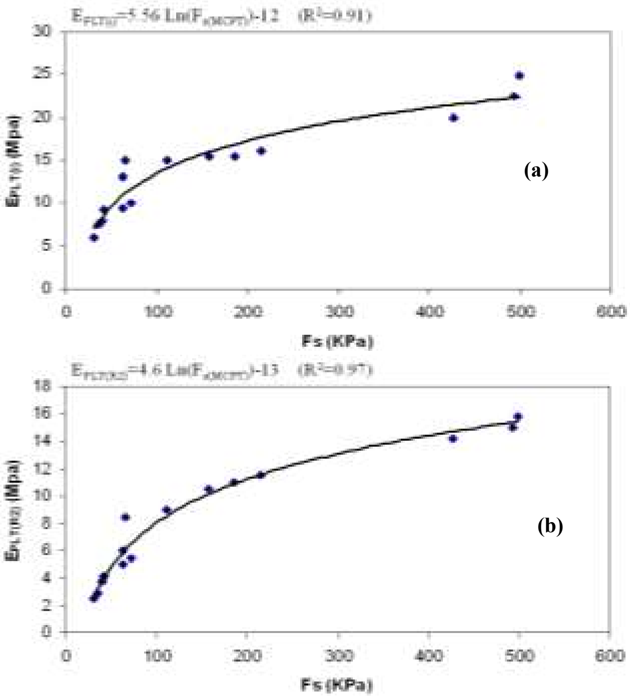

Figure 12: Correlation between a) $E_{PLT(i)}$ and b) $E_{PLT(R2)}$ with f_s

The correlations established in this study were compared with the equations obtained by different researchers. Schmertmann *et al.,* (1986) gave a simple linear correlation for silty/clayey sands as follows:

$E=7q_c$ (14)

Trofimenkov (1974) also suggested the following correlations for the elasticity modulus in sand and clay:

$E=3q_c$ (for sand) (15)

$E=7q_c$ (for clay) (16)

These researchers established linear equations between E and q_c, while in this study logarithmic equations between them were obtained.

Mini-CPT parameters versus shear modulus (G)

Several methods are available to evaluate the shear modulus of coarse-grained and fine grained soils; they include geophysical methods, Plate Load Test (PLT), etc., which are all costly.

In the present research, correlations between the shear modulus ($G_{PLT(i)}$) and miniature cone and resistance parameters ($q_{c(MCPT)}$ and $f_{s(MCPT)}$) were investigated. The best correlation between the shear modulus (G) with $q_{c(MCPT)}$ and $f_{s(MCPT)}$) is presented in Fig. 13, and also equations (17) and (18), respectively:

$G_{PLT(i)}=1.37(q_{c(MCPT)})^{0.97}$ $R^2=0.92$ (17)

$G_{PLT(i)}=0.1(f_{s(MCPT)})^{0.869}$ $R^2=0.94$ (18)

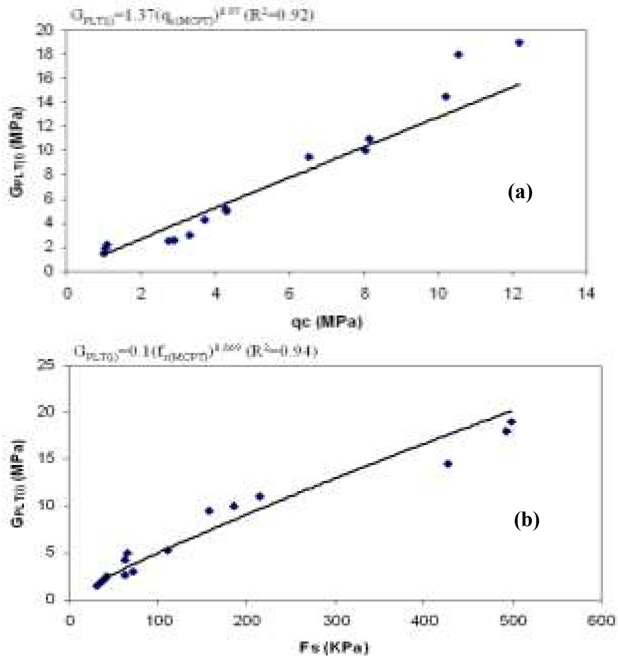

Figure 13: Correlation between a) $q_{c(MCPT)}$ and b) $f_{s(MCPT)}$ with G

It can be seen from equations (17) and (18) that the best-fitted relations are represented by power regression curves. The determination coefficient was obtained between $G_{PLT(i)}$ and $q_{c(MCPT)}$ and $f_{s(MCPT)}$ is 0.92 and 0.94, respectively.

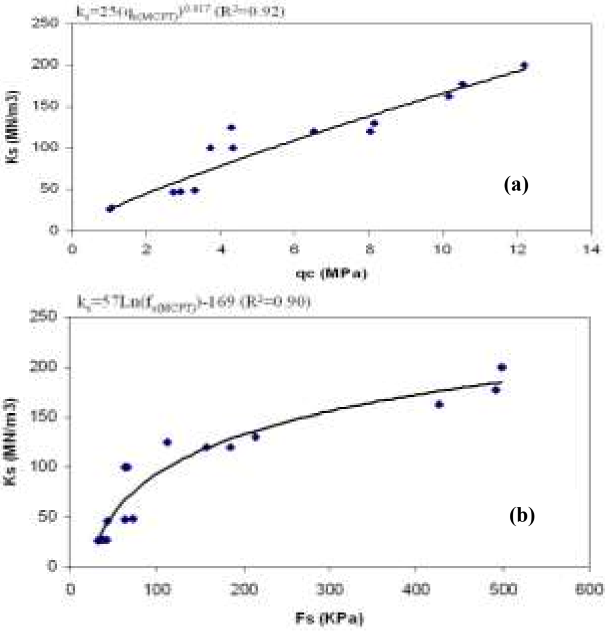

Figure 14: Correlation between k_s with a) $q_{c(MCPT)}$ and b) $f_{s(MCPT)}$

Mini-CPT parameters versus subgrade reaction modulus (K_s)

The best relationships between subgrade reaction modulus and Mini-CPT parameters are presented in Fig. (14), and also equations (19) and (20):

$$K_s=25(q_{c(MCPT)})^{0.817} \qquad R^2=0.92 \qquad (19)$$

Where, K_s is the subgrade reaction modulus and $q_{c(MCPT)}$ is the tip resistance.

As can be seen from equation (20), there is a power relationship between K_s and $q_{c(MCPT)}$ with a determination coefficient of 0.92.

A logarithmic relationship was observed between K_s and friction resistance ($f_{s(MCPT)}$) with a lower determination coefficient (R^2=0.90) than that of K_s and $q_{c(MCPT)}$:

$$K_s=57Ln(f_{s(MCPT)})-169 \qquad R^2=0.90 \qquad (20)$$

Comparison of the determination correlations in equations (19) and (20) shows that $q_{c(MCPT)}$ is better than $f_{s(MCPT)}$ for estimating K_s (because of higher determination coefficient).

Conclusions

In soil exploration, an approach is offered by cone penetration testing (CPT). Mini-CPT is one of the newest types of CPT, which is less in diameter than the conventional test equipments. The most important advantages of miniature penetrometer are: 1- smaller downward thrust needed to advance the penetrometer into the soil, 2- ability to identify very thin layers, and 3- installation in a smaller vehicle that provides greater mobility and site accessibility. In this research, the ability of Mini-CPT to determine some engineering properties of sandy soil was investigated. Due to the lack of convenient field conditions, all tests were done in laboratory conditions and also a Mini-CPT apparatus was developed. By having the tip and friction resistance and carrying out some tests such as PLT and direct shear test to determine soil deformability parameters and friction angle, the best correlations between them were obtained. Final results are shown in the form of empirical correlations with high value of determination coefficient (Table 4). The use of $q_{c(MCPT)}$ and $f_{s(MCPT)}$ to the determine properties of sandy soils, in most instances, gives the same value of R^2 except for the case of D_r. Therefore, it is highly recommended to use $q_{c(MCPT)}$ than $f_{s(MCPT)}$ to determine relative density (for its higher value of R^2).

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Appendix 1

The static load plate test is evaluated by assuming that the assessed subsoil can be characterized by a linear elastic, homogeneous, isotropic half-space. From the settlements of the rigid circular plate, which is loaded by a concentrated force P the Young's modulus E can be determined according to the theory of elasticity :

$E=(1-\upsilon^2)P/2rs$

Where υ is Poisson's ratio of the subsoil, and r denotes the radius of the plate. Assuming that the pressure p below the load plate is uniformly distributed, we have:

$p=P/r^2\pi$

Assuming further that the Poisson's ratio is constant for all soils with $\upsilon = 0.212$, the so-called deformation modulus E can be readily defined as:

$E_v=1.5r\Delta p/\Delta s$

In the above expression pressure p and settlement s are replaced in this equation by their increments Δp and Δs since the soil behavior is nonlinear.