Evaluation and Analysis of Electric Cone Penetrometer Test Results

by

Umesh Dayal *

S.K. Jain *

Introduction

The static cone pentrometer test (CPT) has been orginally developed and used in Netherland and Belgium to obtain insitu strength properties of sub-surface soils. The use of static cone pentrometer has increased in recent years because the test (1) is quick, easy and economical, (2) provides information on soil characteristics insitu, and (3) is a particularly good investigative tool for sands, where undisturbed sampling is difficult. The major disadvantage of CPT is that it does not provide samples for visual observations of soil type and laboratory tests. There are various types of CPT equipments and methods, which have been summarized by Sanglerat (1972). Of these, Dutch CPT sounding systems and methods have became popular all over the world. The invention of the friction sleeve for measuring the local side friction has greatly enhanced the value of the information gained from CPT and its use along with the cone tip has become a routine.

Literature review has shown that CPT results can be used, (1) to derive information on soil types (Begemann 1965, 1969 and Dayal and Allen 1975); (2) to determine pile supporting capacity (Vander Veen 1957, Krisel 1961, Menzenbach 1961, De Beer 1963, Heizenen 1974, and Huiter and Beringen 1979), (3) to determine ϕ in sands (De Beer 1948, Meyerhof 1956, 1961, Sanglerat 1972, Janbu and Senneset 1974, Durgunoglu and Mitchell 1975, and Schmertmann 1975), (4) to find shear strength in clays, (Sanglerat 1972, Durgunoglu and Mitchell 1975, Schmertmann 1975, and Lunne and Ruiter 1976), (5) to determine compressibility and in-situ relative density of cohesionless soils (Meyerhof 1956, Rodin 1961; Schultze and Malzer 1965 and Schmertmann 1975); (6) to estimate the settlement of footings on sands, according to the methods proposed by Buisman (1974), De Beer and Martens (1957) and Schmertmann (1970); and (7) to characterise vehicle trafficabiliiy over unpaved soils (Murphy 1965, Freitag et al. 1970 and Wiendieck 1970).

This paper describes the development of an electrical penetrometer which has been designed and fabricated at I.I.T. Kanpur. The performance of the equipment has been evaluated in the field and the test results thus obtained are compared with mechanical cone penetrometer test results. At the outset of this study the state-of-art of estimating shaer strength

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* Department of Civil Engineering, Indian Institute of Technology, Kanpur, India

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parameters from CPT results has been critically reviewed and compared with the field test results of this investigation.

Current Methods for Estimating Shear Strength

The static cone penetration test does not measure shear strength directly but measures the cone bearing capacity (q_c) and soil steel friction (local side friction) along the friction sleeve f_s both of which depend on shear strength of soils. During the last decades several theories for calculating shear strength parameters have been presented. In-addition, purely empirical correlations based on laboratory tests conducted under controlled conditions have been suggested for estimating shearstrength parameters and identifications of soil type. *Several of these theories are summarized* below.

Angle of Internal Friction (ϕ') in sands

De Beer Method (1945)

De Beer's 'old' theory is based on the assumption of an incompressible material and that cohesion can be neglected. This method originates from bearing capacity theory. This theory has been used widely, although it is recognised by the author (De Beer, 1974) that it gives very conservative value of ϕ'.

Meyerhof Methods (1961, 1974)

Meyerhof (1961) has presented bearing capacity factors for various angles of internal friction for rough and smooth cones and wedges of different apex angles as shown in Figure 1*a* correlation between limiting static cone resistance (q_c is the maximum value of q_c obtained at a, critical depth, below which the penetration resistance shows little or no increase with continued penetration). Mitchell and Lunne (1978), found that this method gives ϕ'-values reasonably well where q_c does not increase significantly benefit a certain depth i.e. in case where critical depth is found.

Muhs and Weiss Method (1971)

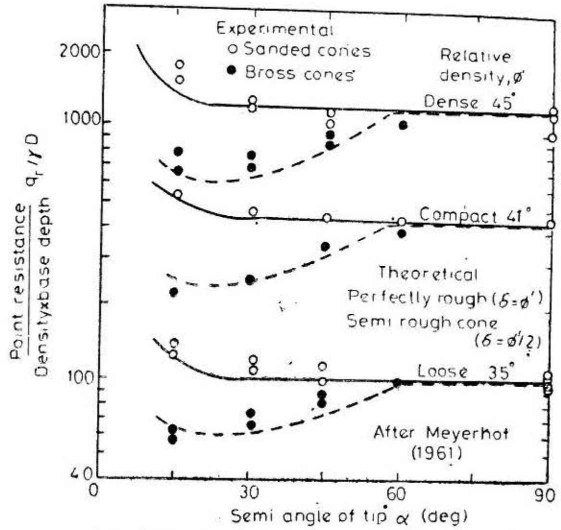
Based on large scale model footings on sand Mush and Weiss (1971) have reported the following relationship,

$$q_c = 0.8\ N_r \qquad \text{... (1)}$$

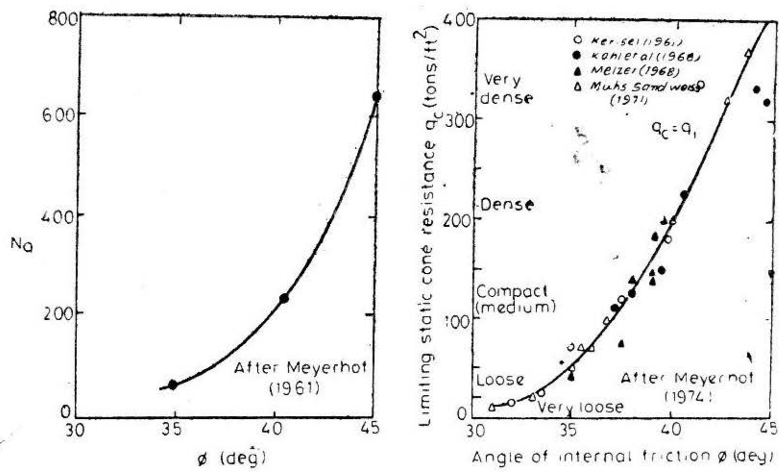
where q_c is unit cone resistance in kg/cm^2 and N_r is ordinary bearing capacity factor. Because, N_r depends only on ϕ', one can compute ϕ' indirectly. The result generally applies to the bearing capacity computation of shallow footings.

Janbu and Senneset Method (1973, 1974)

Janbu and Senneset have proposed a method for determining ϕ' (effective) and, a, (attraction) in sand and cohesive soils, which is based on bearing capacity theory modified by empirical observations. This theory applies to situations where q_c profile increases linearly with depth in deposits



(a) Effect of roughness and apex angle



(b) Values for $\delta/\Phi' = 0.5$ and 30° semi-apex angle

(c) Limiting cone resistance as function of friction angle

FIGURE 1 Meyerhof's bearing capacity factors

assumed to have an approximately constant ϕ, and C over the depth interval. The theory can best be described by a straight line given by

$$\tau_f = (a + \sigma') \tan \phi' \qquad \dots (2a)$$

in which,

τ_f = shear strength

a = 'attraction' $\left(a = \dfrac{C}{\tan \phi'}\right)$, as defined in Figure 2*a*.

σ' = the effective normal stress and

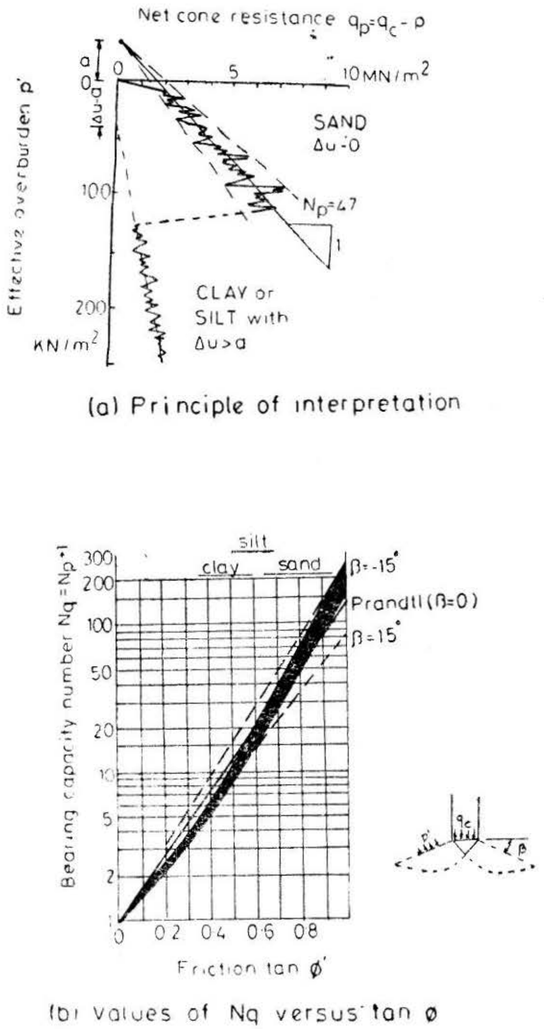


FIGURE 2 Janbu and Senneset method for determining ϕ' and a from CPT sounding log

ϕ' = effective angle of internal friction.

The following formula is given for cone resistance.

$$q_c + a = N_q\ (p' + a) \quad \text{... (2b)}$$

or

$$q_p = N_p\ (p' + a) \quad \text{... (2c)}$$

in which

$$N_p = N_q - 1 \quad \text{... (2d)}$$

and

$$q_p = q_c - p' = \text{net cone resistance} \quad \text{... (2e)}$$

N_q = bearing capacity factor, depending on tan ϕ'.

Hence, by plotting q_p versus effective overburdan p', one arrives at the following procedure of interpretation :

(*i*) Draw an average line through the variation of q_p with p'. The (negative) intercept on the p'-axis equals attraction a, because $q_p = 0$ when $p' = -a$ from Equation (2*c*).

(*ii*) The slope of this line equals $N_p = N_q - 1$. Hence $N_q = N_p + 1$ yields tan ϕ' from Figure 2*b*. Thus the value of ϕ' can be calculated from tan ϕ' value.

Janbu and Senneset have given values of tan ϕ' and, a, in Table 1 for the purpose of illustrating the order of magnitude obtained in the region of Norway.

TABLE 1

Typical Values of Strength Parameters by Janbu and Senneset (1974)

Soil Condition	Friction tan ϕ'	Attraction a, KN/m^2
Sand	0.65-0.95	0.900
Silt	0.50-0.70	0.300
Clay	0.35-0.60	0.120

Mitchell and Lunne (1978), applied this theory for determining ϕ' for number of test sites and found that this method compare reasonably with actual measurements. They confirmed that if a straight line portion of the q_c versus depth cannot be defined, Janbu and Senneset theory can not be used.

Trofimenkov Method (1974)

A less conservative procedure than that of De Beer, with a semi-empirical basis, is being used in USSR. A chart for determining ϕ from overburden pressure and cone resistance presents this correlation, and is valid up to an effective overburden pressure of 1 kg/cm^2 (Figure 3).

Durgunoglu and Mitchell Method (1973, 1975)

This theory is based upon the results of laboratory tests. A rigid plastic, wedge-displacement bearing capacity theory was used, with empirical modifications to take into account the circular shape of the cone. The ultimate static cone penetration resistance is given as :

$$\frac{q_f}{\gamma_s B} = \left(\frac{C}{\gamma_s B}\right) N_c \, \xi_c + N_{rq} \, \xi_{rq} \qquad \ldots (3a)$$

For static penetration tests performed with a given cone there are many combinations of C and ϕ' which satisfy Equation 3*a* for a given value of q_f/γ_s. If penetration data are available for two sizes of cone, or if the soil deposit is homogeneous and the penetration resistance is known at two depths, then specific values of C and ϕ' may be determined by simultaneous solution of two equations of the form of Equation 3*a*, one for each combi-

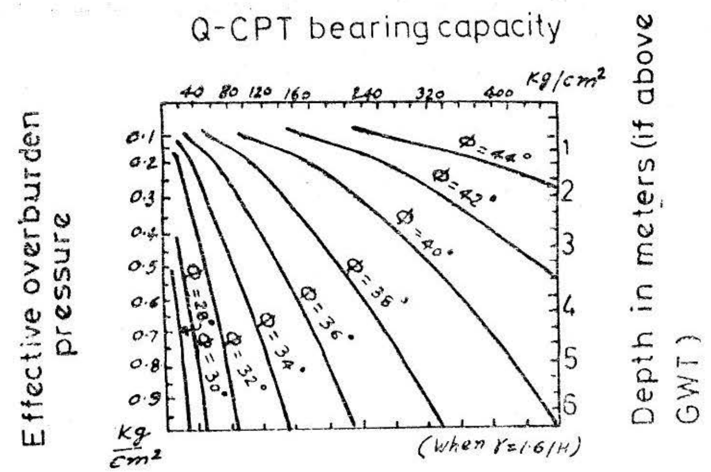


FIGURE 3 Method for estimating ϕ' from q_c reported in use in USSR (Trofimenkov method 1974)

nation of q_f and D/B values. Generally it is easier to eliminate C from the two equations and determine ϕ by trial. For cohesionless soil the theory leads to

$$q_c = \gamma_s \, B \, N_{rq} \, \xi_{rq} \qquad \ldots (3b)$$

in which q_c = static cone point resistance

γ_s = soil unit weight

B = cone diameter

N_{rq} = bearing capacity factor for wedge penetration (plain strain); and

ξ_{rq} = shape factor to convert wedge factors to cone factors.

Therefore, the value of $N_{rq}\,\xi_{rq}$ can be calculated from Equation 3b, which is dependent on soil friction angle ϕ', base roughness δ/ϕ', relative depth of penetration D/B, lateral earth pressure coefficient K_0' and cone apex angle 2α. Durgunoglu and Mitchell (1975) have presented charts to calculate N_c, N_{rq}, ξ_c and ξ_{rq} and thereby ϕ' and c can be estimated indirectly.

Schmertmann Method (1975)

This is an indirect method for estimating ϕ' through the relative density (D_r) parameter. From the results of chamber tests on normal consolidated, medium to fine dry and nearly saturated sands carried out at the University of Florida, Schmertmann constructed the curves shown in Figure 4 which are also based in part on the results from the relative density studies of Mississippi River sands below water table. After making estimate of D_r from Figure 4a, tha value of ϕ' can be estimated using the correlation given in Figure 4b.

Overconsolidated sands must be converted to their equivalent normally consolidated q_c before entering Figure 4. Following two equations can be used for this purpose if an independent estimate of the overconsolidation ratio (OCR) or the insitu K_0' coefficient of the sand is known

$$\frac{K_0'}{K'_{0NC}} = (\text{OCR})^{0 \cdot 42} \qquad \ldots (4a)$$

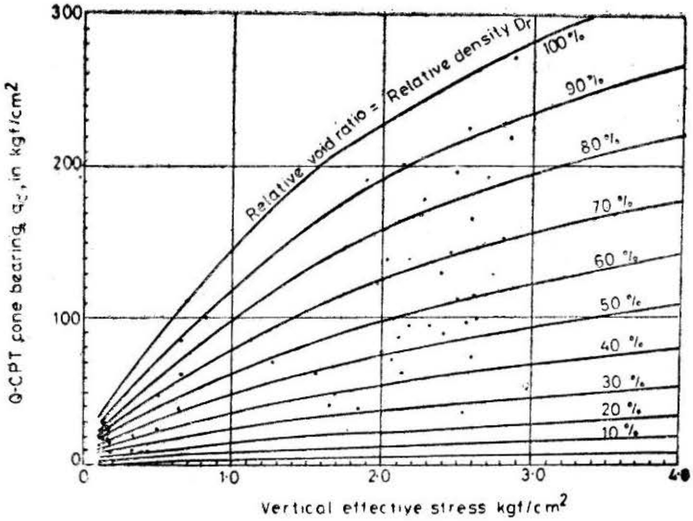


FIGURE 4*a* *Q*—CPT Bearing capacity to estimate relative density in normally consolidated silty fine to uniform medium sands (Schmertmann 1975)

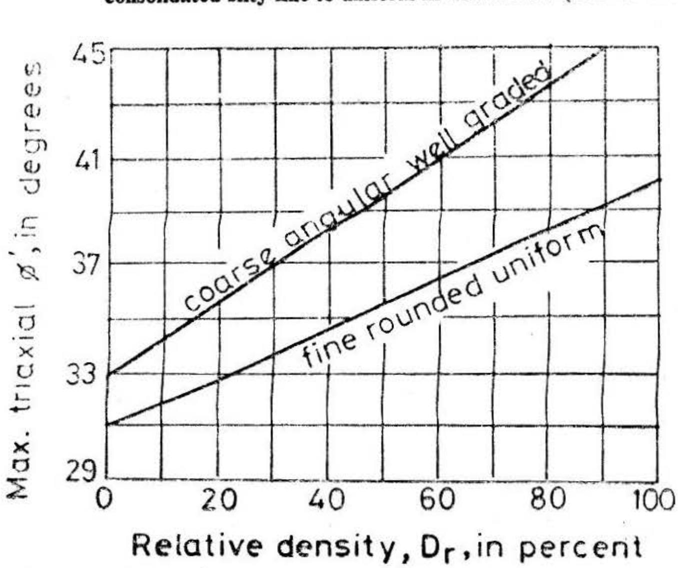


FIGURE 4*b* Approximate correlation between ϕ' and D_r in quartz sands (Schmertmann 1975)

$$\frac{q_c}{q_{cNC}} = \left(1+3/4\left(\frac{K_0'}{K_{0NC}'} - 1\right)\right) \qquad \text{... (4b)}$$

where, OCR = overconsolidation ratio;

K_0' = lateral pressure coefficient;

Mitchell and Lunne (1978) applied this method to various CPT results and found that the value of ϕ' estimated compare well with other methods.

Shear Strength of Cohesive Soils

Sanglerat (1972) observed that for the soft clays of the Annecy area (France), the undrained cohesion was always within the range of $q_c/20$ to $q_c/10$ and often very close to the value of $q_c/15$, thus :

$$C_u = \frac{q_c}{15} \qquad \text{... (5)}$$

Thomas (1965) and Sanglerat (1972) have shown that for stiff fissured clays, the q_c/c_u ratio should be in the range of 25 to 30.

Schmertmann (1975)

Following equation, relating undrained shear strength C_u, with N_c the bearing capacity factor for clay, is popular among engineers to evaluate shear strength of cohesive soils :

$$C_u = \frac{q_u - \gamma z}{N_c} \qquad \text{...(6)}$$

where, N_c is bearing capacity factor for clay, appropriate for a deep, circular foundation and γz = total overburden pressure at depth of q_c.

However, N_c depends on various factors and varies from 5 to 70. The main factors are clay stiffness ratio, effective friction (tan ϕ'), K_0' or OCR shape of penetrometer tip, rate of penetration and method of penetration. Thus, to use a single N_c value for all soils, all penetrometer tips represent a gross simplification which can lead to serious error.

For ordinary clays $N_c \simeq 10$ with electrical penetrometer tip with cylindrical shafts and $N_c \simeq 16$ for the Begemann mechanical tip, are used both at rates of penetration of 1 to 2 centimeters per second. Meight and Corbett (1969) have suggested $N_c = 16$ for soft clays. It is useful when using friction cone tips to compute the adhesion on the local friction sleeve, f_s, and use this as a lower limit for C_u.

C-ϕ Soils

Both Janbu and Senneset (1974) and Durgunoglu and Mitchell (1975) methods can be used for determining C and ϕ' of the soils. These methods have been discussed in connection with the determination of internal friction in sand and shall not be repeated again.

Advantages And Disadvantages of Various Methods

Each method has its own merits and demerits and is applicable to a certain type of soil. De Beer's theory has been used widely, although it is now recognised that it gives very conservative value of ϕ'. Meyerhof's method gives ϕ' values reasonably well where q_c does not increase significantly beneath a certain depth i.e. in case where critical depth is found. Further disadvantage of Meyerhof theory (1961) is that it can be used only for granular soils having $\phi' > 34°$. Jambu and Senneset method compares reasonably with actual field measurements. However, the application of this theory requires a straight line polt of cone pressure vs depth profile. Quite often it is difficult to interpret straight line profile from actual field results. Trofimenkov method is simple to apply provided the overburden pressure does not exceeds 1 kg/cm^2. This is a *serious* limitation of this method and restrict its use to shallow depth only. Durgunoglu and Mitchell method is based on laboratory penetration test results and gives considerable high value of ϕ. Further, the interpretation in this method require lote of mathematical calculations to arrive c and ϕ values. Schmertmann method is easy to apply for interpretation of the penetration test results of normally consolidated, medium to fine dry sands. In case of overconsolidated sands this method requires additional informations such as overconsolidation ratio and in-situ lateral pressure coefficient (K_o) for converting q_c into q_c NC.

Determination of Soil Type

Begemann (1965, 1969) has shown that there is a definite relationship between the ratio of unit frictional resistance (f_s) to unit cone resistance (q_c) and the soil type as shown Figure 5. Schmertmann (1969) has proposed the ranges of friction ratios values for various types of soils and are given in Table 2. These values are in general agreement with similar information reported by Dayal and Allen (1973).

Development of an Electric Penetrometer

Generally two types of static penetrometers are used now-a-days viz., the mechanical penetrometers and the electrical penetrometers. The

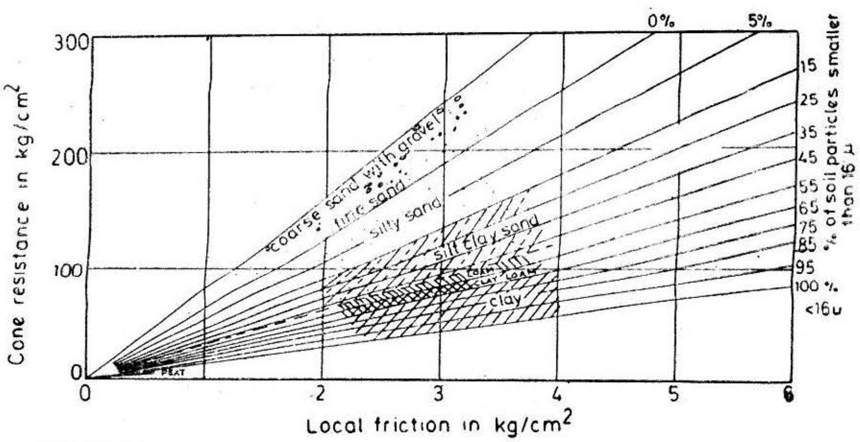


FIGURE 5 Relationship between cone resistance, local friction and soil type (after Begemann 1969)

TABLE 2

Friction Ratio (Schmertmann, 1969)

Soil Type	Friction Ratio per cent*
Very shelly deposits lime rock (soft, shelly, partially indurated limistone)	0.0-0.5
Clear sand, no plastlc fines (independent of relative density)	0.5-2.0
Silty sand	1.75-2.5
Clayey sand, silts, marls, moderately sensitive clays	2.33-3.5
Sandy clay	3.00-4.5
Relatively insensitive clay	over 4.0

* *Note.* Friction ratio is defined as the ratio unit sleeve friction to unit cone resistance.

mechanical type penetrometer has several disadvantages which are listed by Dayal and Suppiah (1979). An electric measuring system offers outstanding advantages, as it allows continuous registration and direct recording of the desired values with a great degree of accuracy. Therefore, various types of electric penetrometers have been developed of which the German Maihak cone equipped with a vibrating wire measuring system is the oldest (Zweak 1969). In recent years, several kinds of strain gauge penetrometers have been developed (DeRuiter, 1971) which provide many advantages over the vibrating wire measuring system.

Traditionally, in India and most other developing countries, mechanical penetrometers are widely used because of non-availability of electrical penetrometers. Recently, a strain gauge type electrical penetrometer was developed in India at the authors' institute. The details of this pentrometer is discussed below.

Mechanical Design

The basic dimensions of the electric penetrometer developed are the same as those generally adopted for the Dutch cone penetrometer. The penetrometer has the following nominal dimensions and characteristics.

Diameter (outer)	= 35.6 mm
Cone Angle	= 60°
Area of the cone base	= 10 sq. cm.
Diameter of the friction sleeve	= 35.6 mm
Area of the friction sleeve	= 150.0 sq. cm.
Internal dia. of tube:	
(*a*) Cone strain tube	= 1.778 cm
(*b*) Sleeve strain tube	= 2.540 cm
Outer dia. of tube:	
(*a*) Cone strain tube	= 2.032 cm.
(*b*) Sleeve strain tub	= 2.667 cm.

Wall thickness

(*a*) Cone strain tube = 0.127 cm.

(*b*) Sleeve strain tube = 0.0635 cm.

The cone and the friction sleeve have been designed for a rate capacity of 900 Kg. and 500 Kg. respectively. The design is such that there is no contact between the cone and connecting rods other than through the load cell. The sleeve friction tube forms a cylinderical shaft above the cone.

The details of the various elements of the penetrometre are shown in Figure 6. The major components are

(1) Main block housing the cone strain gauges

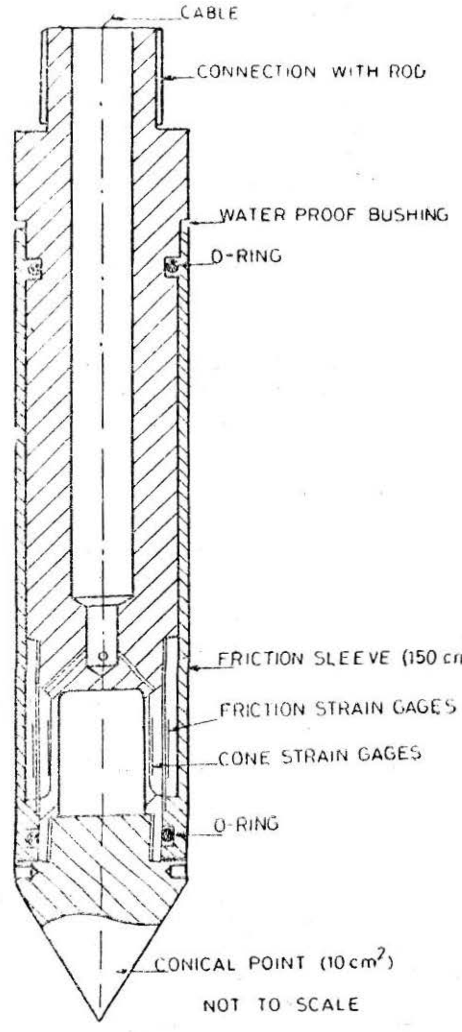


FIGURE 6 Details of load cell arrangement

(2) Detachable cone base

(3) Sleeve strain tube

(4) Sleeve tube

Electrical Transducers

To measure the cone load and the sleeve friction, strain gauge type load cells have been developed which are similar in principle to these described for 'Fugro' penetrometer by DeRuiter (1971). Each load cell contains four pairs of gauge arranged in such a manner that automatic compensation is made for pending stress and temperature and only axial stress is measured. Four strain gauges are gauged in the axial direction and the remaining four in the circumferential direction, at equal distances, in the periphery in the tube. A cross-sectional view of the cone and the sleeve measuring arrangements is shown in Figure 6.

Recording System

The output signals of the cone load cell and the friction load cell are recorded on multichannel 'Honeywell' visicorder. Before feeding to the recorder, the signals are amplified by selecting a suitable gain on the differential amplifiers. The signals are recorded on photosenstive paper by the galvnometer deflactions. The paper speed can be adjusted according to the requirements. Alternatively, the data can be recorded on a dual trace storage-scope.

Calibration

The cone and the friction load cells are calibrated on a loading frame. For the calibration of the load cells, special jigs were designed and fabricated so that only axial load was applied during the compression of the tube. The cells were loaded up to the design load in 50 kg. increments and their response was recorded. The plot of the galvanometer deflection vs the applied stress obtained from the proving ring provided the calibration curve. The relationship was linear except for a littie scatter. To taken into account the drifting of the load cells and the recording system, the following calibration checks are usually performed before conducting the test :

(1) the actual gain of all amplifiers

(2) the balancing of the amplifiers

(3) the input voltage of each load cell (the load cells were calibrated at 10 V DC excitation)

(4) balancing the bridges of the cone and the friction load cells.

In-Situ Test Programme

Experimental Set-up

The penetrometer is advanced into the soil at penetration rate of approximately $1\frac{1}{2}$ cm/sec. by hand operated ring having rated capacity of 3T. The pentrometer rods are added at regular interval of 1 meter penetration. The signals from the cone and the friction sleeve load cells are transmitted via cables and through the hollow sounding rods to the surface. The signals are amplified before feeding to visicorder which continuously record resristances. The system is calibrated to provide cone resistance and local friction directly in kg/cm². The field experimental set-up in Figure 7.

The tests were performed in conjunction with electric cone penetrometer with mechanical cone penetrometer to compare and evaluate the suitability of electric cone penetrometer.

Test Beds

Tests were carried out on following types of test beds :

(1) Cohesive soil

(2) Cohesionless soils

(3) Layered soils.

Cohesive Soils—The tests on cohesive soil was conducted on local soil of I.I.T. Campus. The soil at the campus is a typical alluvail soil of Indo-Gangetic plain which predominently consists of silts. The average properties of the test site are given below :

Liquid Limit	= 31 per cent
Plastic Limit	= 13 per cent
Grain Size, sand	= 10 to 15 per cent
silt	= 70 to 80 per cent
clay	= 10 to 15 per cent
Dry density γ_d	= 1.66 gm/cc
Undrained angle of internal friction (ϕ)	= 19^0
Undrained cohesion (c)	= 0.3 kg/cm²
Effective angle of internal friction (ϕ')	= 32^0

The above soil has been classified as CL from IS: 1498-1970.

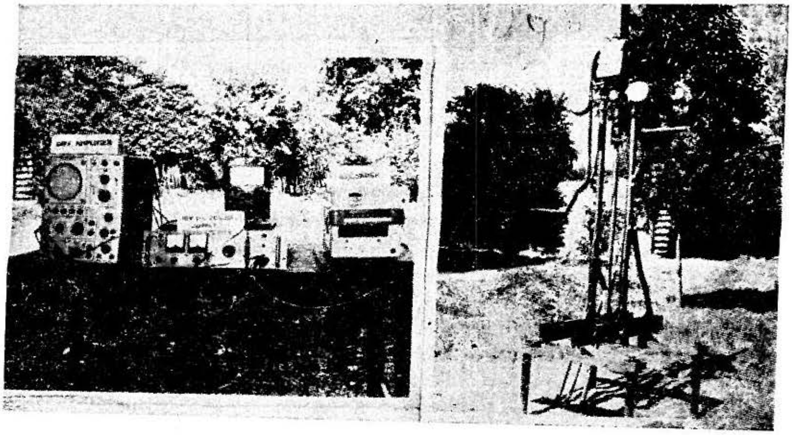


FIGURE 7 Photographic views of field test

Cohesionless Soils-Due to non-availability of cohesionless soil locally, artificial beds of Kalpi sand were prepared. For this, holes of 15 cm dia. were augured to a depth of 2.8 meter and then weighted amount of Kalpi sand was filled in the hole. The tests were conducted in the loose sand (1.48 gm/cc) and medium dense sand (1.63 gm/cc). The soil properties of Kalpi sand are summarised below.

Effective size (D_{10})		$=0.3$
Coefficient of uniformity (C_u)		$=6.7$
Coefficient of curvature (C_c)		$=1.67$
Specific gravity of grains (G)		$=2.67$
Maximum void ratio ($e_{max.}$)		$=0.82$
Minimum void ratio ($e_{min.}$)		$=0.46$
Test Bed Very loose		Medium dense
Dry density (γd)	1.48	1.63
Void Ratio (e)	0.804	0.638
Relative density (D_r)	4	50
Angle of Internal Friction (ϕ')	33	37

Kalpi sand has been classified as per IS:1498-1970 as SW.

Layered Soils

To evaluate the sensitivity of the equipment for layered system, few tests were conducted in a artificially prepared layered system in which top layer was Kalpi sand to a depth of 2.5 meter followed by local silty soils. The procedures for preparation of layered test beds are same that discussed for cohesionless sols.

Test Results

The CPT results on insitu silty soil (F.C.T.1) have been plotted in Figure 8. The depth versus cone resistance and sleeve friction obtained by electric cone penetrometer are plotted to a depth of 3.5 meter. For comparison, the results of mechnical cone penetrometer have also been plotted in the same Figure.

Friction ratio (F.R.) calculated from electrical and mechanical cone pentrometers are also plotted in Figure 8.

The results of electric cone penetration tests on dry loose sand (F.S.T.1) and dry medium dens (F.S.T.2) Kalpi sand are given in Figure 9. The cone resistance and sleeve friction profiles are plotted to a maximum penetration depth of 3.5 m. It should be noted that sand was filled in the augured hole of a depth of approximately 2.8 m and beyond this depth the penetration was in natural silty soil. Friction ratio (per cent) versus penetrations depth is also shown in the same Figure 9. The layering effect at 2.8 m depth can be clearly observed in this profile. Below 2.8 m depth, a sudden increase in sleeve friction is observed which increases continuously to penetrated depth. The cone resistance at interface of

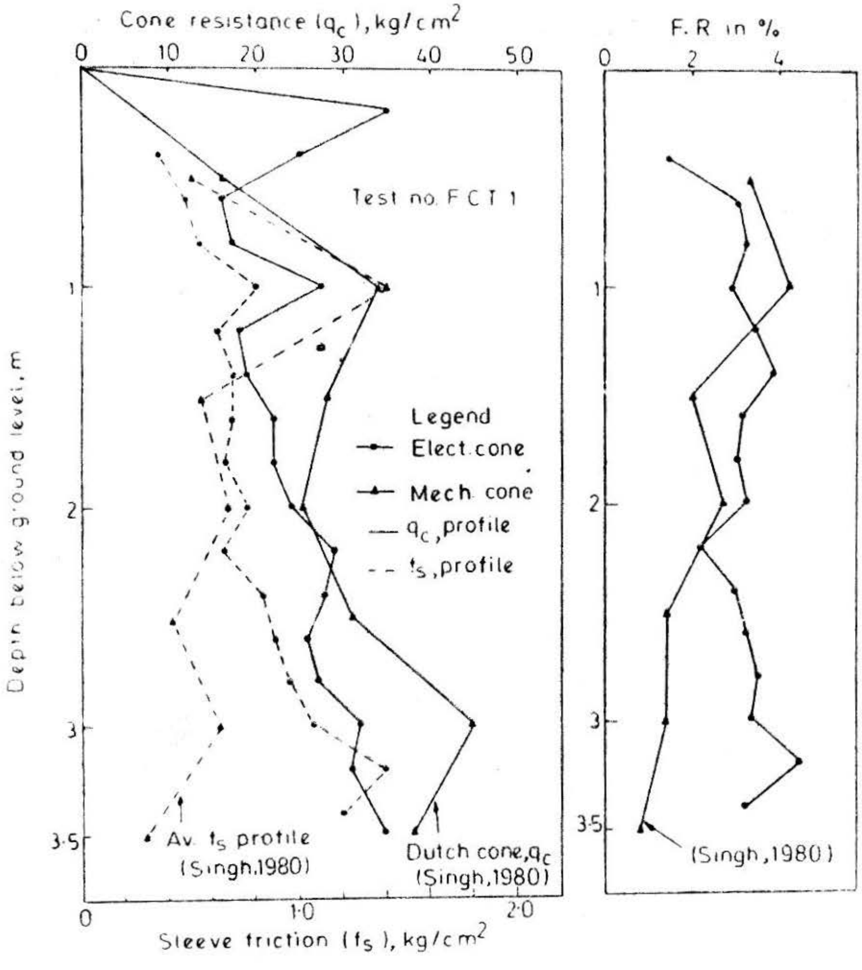


FIGURE 8 In-situ test results in IIT., Kanpur silty soil by electric and mechanical cone penetrometer

two layers decreases suddenly and then start increasing again up to a penetration depth of 3.5 m. Frictin also shows a sudden increase at 2.8 m depth, where penetrometer just enters into the silty soil.

Interpretation of Test Results

The different theories and methods for computing shear strength of soils, described earlier, are used herein to interpret the results of field test data. Following test results are analysed.

(*i*) field CPT results on loose sand (F.S.T.1),

(*ii*) field CPT results on medium dense sand (F.S.T.2),

(*iii*) field CPT results on silty soil (F.C.T.1).

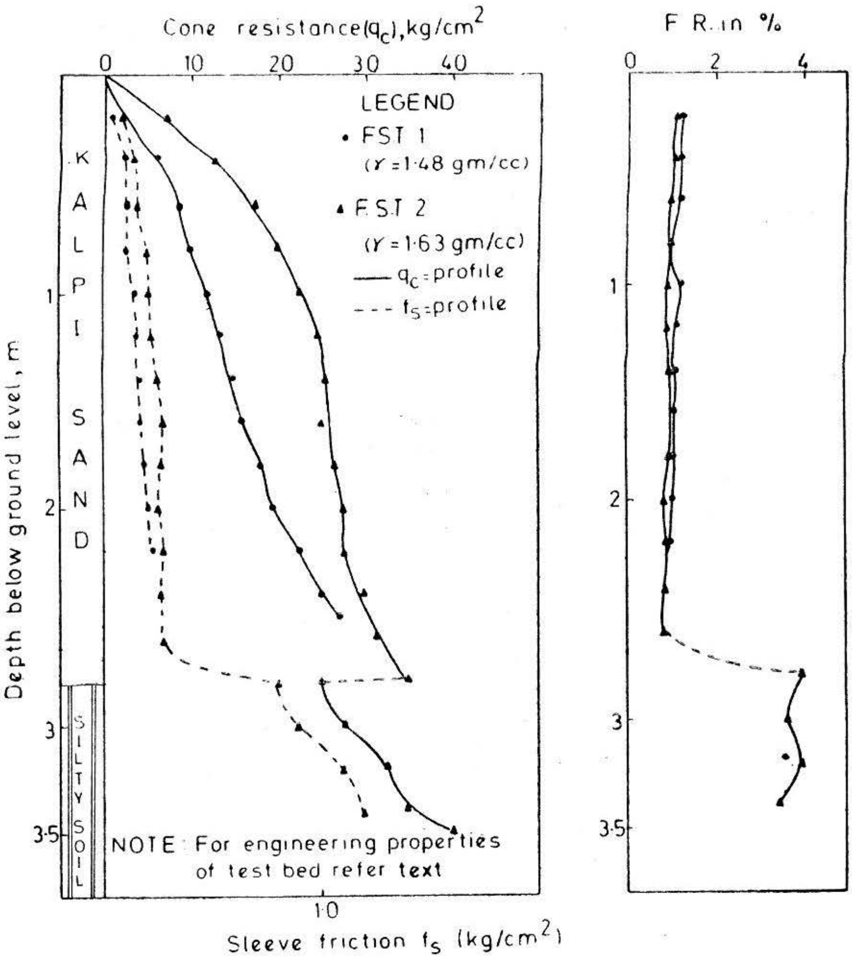


FIGURE 9 Field test results in Kalpi sand by electric cone penetrometer

Meyerhof Method (1961)

The data required to use this method is cone resistance divided by overburden pressure (or N_q). A relation between $q_c/\gamma b$, N_q, and ϕ is given in Figure 1 from which the ϕ can be computed. The values of ϕ obtained by this method for loose and medium dense sand for depth intervals of 0 5 m are given in Table 3.

Janbu and Senneset Method (1973, 1974)

In order to determine the ϕ and a (attraction) values, the results of penetration test are plotted in the form of net cone resistanee q_p versus effective overburden pressure as shown in Figure 10. A straight line portion for q_p versus effective overburden pressure is selected and ϕ and a values are calculated (Table 4) by the interpretation method as discussed previously.

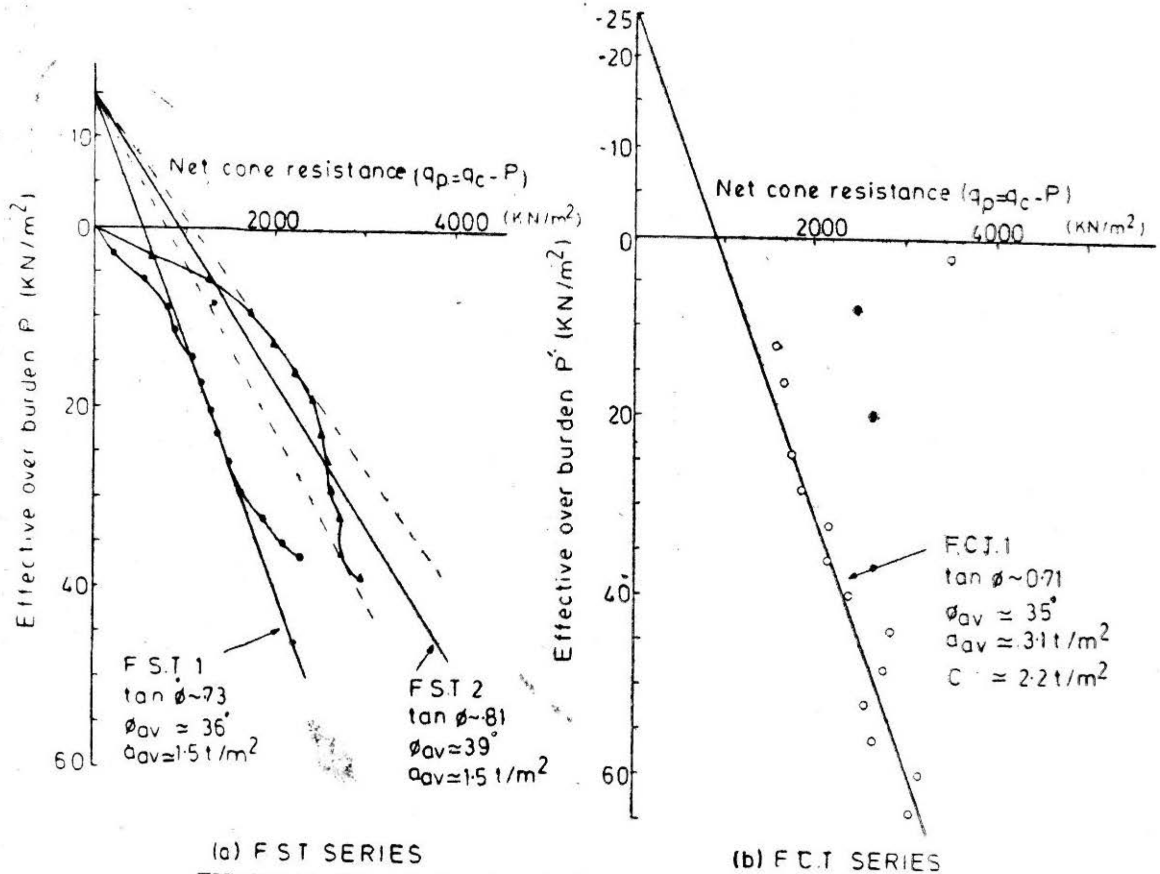


FIGURE 10 Interpretation of penetration results by Janbu and Senneset theory

TABLE 3

Determination of ϕ from Meyerhof Method

Depth m	M mean Depth	F.S.T.1 (Loose sand)		F.S.T.2 (Medium dense sand)	
		N_q	ϕ' in degree	N_q	ϕ' in degrees
0.0–0.5	0.25	95	36.0	207	40.0
0.5–1.0	0.75	86	35.5	160	39.0
1.0–1.5	1.25	76	35.0	123	37.5
1.5–2.0	1.75	68	34·5	093	36.0
2.0–2.5	2.25	71	35.0	076	35.0
Average value			$\phi_{av} = 35°$		$\phi_{av} = 37.5°$

TABLE 4

Determination of ϕ and a from Janbu and Senneset Method

Test No	Electric CPT		
	ϕ_{av} degrees	a_{av} t/m²	c_{av}. t/m²
Loose sand (F.S.T.1)	36	1.5	1.1
Medium Dense Sand (F.S.T.2)	39	1.5	1.2
Silty Soils (F.C.T.1)	35	3.1	2.2

Trofimenkov Method (1974)

The input required is cone resistance and effective overburden pressure. Chart given in Figure 3 is used for computing the values of ϕ. The computed values of ϕ are tabulated in Table 5

Durgunoglu and Mitchell Method (1973, 1975)

For interpreting the results of CPT, a plot of cone factor (N_{rq}, ξ_{rq}) versus ϕ is prepared for the various values of ϕ rangin from 30° to 45° at the interval of 5° and D/B values of 1 to 30 (for the specific value of semi apex angle $\alpha = 30°$). These curves have been prepared from the curves given by Durgunoglu and Mitchell (1975). Knowing N_{rq}. ξ_{rq} $(=\frac{q_f}{\gamma_s . B})$ from experimental results the values of ϕ is estimated.

TABLE 5

Determination of ϕ from Trofimenkov Method

Depth m	Mean depth m	F.S.T.1 (Loose sand)			F.S.T.2 (Modium dense sand)		
		q_c kg/cm²	γZ kg/cm²	ϕ in degrees	q_c kg/cm²	γZ, kg/cm²	ϕ in degrees
0.0—0.5	0.25	03.5	0.04	—	08.5	0.04	—
0.5—1.0	0.75	09.5	0.11	33.0	19.5	0.12	34.0
1.0—1.5	1.25	14.0	0.19	32.5	25.0	0.20	33.0
1.5—2.0	1.75	17.5	0.26	31.5	26.5	0.29	32 0
2.0—2.5	2.25	23.5	0.33	31.5	28.0	0.37	31.5
Average value		$\phi_{av} = 32$			$\phi_{av} = 32.5$		

For $D/B \geqslant 30$ the values were calculated by trial and error method. Table 6 gives the calculated values of cone factor and ϕ for two depth of 0 to 1.5 m and 1.5 to 2.5 m.

TABLE 6

Durgunoglu and Mitchell Method, (Electric CPT)

Depth, m	D/B	F.S.T.1 (Loose sand)		L.S.T.2 (Medium dense sand)	
		N_{rq} ξ_{rq} (t/m²)	ϕ', degrees	N_{rq}. ξ_{rq} (t/m²)	ϕ', degrees
0.0—1.5	21.0	1783	38.5	3323	41.5
1.5—2.5	55.6	3660	45.0	4686	46.0
Average value			41.5		43.5

This theory could not be used, for interpreting the results of CPT on silty soil of I.I.T. Kanpur because

(*i*) two sizes of cone tips are not available ;

(*ii*) the plot of N_c vs ϕ and N_{rq} ϕ are not available for the range of ϕ generally encountered (i.e. 15° to 25°).

Schmertmann Metheod (1975)

To use this method, values of K_o, cone resistance, and effective overburden pressure are required. From the chart given in Figure 4a, the relative density, D_r, can be found and the relation between D_r, and ϕ shown in Figure 4b is then used to estimate ϕ. The values thus calculated are tabulated in the Table 7.

TABLE 7

Determination of D_r and ϕ from Schmertmann Method

Depth, m	F.S.T.1 (Loose sand)			
	q_c kg/cm²	Z kg/cm²	D_r Per cent	ϕ' Degrees
0.0-0.5	03.5	0.04	—	—
0.5-1.0	09.5	0.11	30.0	35.0
1.0-1.5	14.0	0.19	41.0	36.5
1.5-2.0	17.5	0.26	45.0	37.0
2.0-2.5	23.5	0.33	50.0	37.5
Average Values			41.5	36.5
0.0-0.5	08.5	0.04	—	—
0.5-1.0	19.5	0.12	60	38.5
1.0-1.5	25.0	0.20	65	39.0
1.5-2.0	26.5	0.29	60	38.5
2.0-2.5	28.0	0.37	55	38.0
Average Values			60	38.5

Determination of Soil Type

(*a*) Begemann Graph (1969)

Figure 5 has been used to predict the type of soil for a particular value of unit frictional resistance and unit cone resistance. The relationship in Figure 5 reveals, for cohesionless soils, that percentage of soil particles smaller than 16 μ is zero and for silty soils, this relationship concludes that the tested soil is silt-clay-sand containing 25 per cent to 45 per cent of soil particles smaller than 16 micron.

(*b*) Schmertmann (1969)

To identify the soil type, the relationship between friction ratio and soil type has been used. Table 8 gives the soil type predicted from this method.

TABLE 8

Prediction of Soil Type, Schmertmann (1969)

Test No.	Range of F.R. Per cent	Mean value of F.R. Per cent	Type of soil
Loose sand (F.S.T.1)	1.0-1.3	1.15	Clean sand with no plastic fines
Dense Sand (F.S.T.2)	0.9-1.1	1.00	Clean sand with no plastic fines
Silty Soil (F.C.T. 3)	1.4.4.4	2.90	Silty sand, clayey sand and silts

Evaluation of Existing Theories

In order to compare the predicted and measured CPT values, a summary of results is presented in Table 9. The computed values of ϕ, attraction, a (Janbu and Senneset method) and relative density D_r (Schmertmann method), if applicable are shown in this table. The values such as ϕ' and D_r calculated/best predicted from laboratory tests are included in this table for comparison of the results.

The comparison of theroetical and experimental results that the Durgunoglu and Mitchell method gives very high values compared to actual values of ϕ' obtained by other methods. The range of variation is from 5° to 9°. Trofimenkov method gives the lowest values of ϕ' in comparsion to others.

The values calculated by Meyerhof theory is in between the above two extremes. The Meyerhof values are generally higher than the actually measured values and the difference ranges from 1° to 3°.

Schmertmann's method has been applied assuming the value K_0 equal to (1-Sinϕ). The estimated values of angle of internal friction are within the accuracy of 3°.

Only Janbu and Senneset's theory provides higher values than the measured values. The soil type as determined from the method proposed by Begemann is not very dependable. Schertmann method (1969) predicts the soil type reasonably well.

Summary and Conclusions

The main objectives of the present study were, (1) to design and develop an electric cone penetrometer, (2) to test the capability of the equipment in laboratory and field; and (3) its usage to cualuate the existing theories for interpretation of GPT results in terms of soil parameters (C and ϕ) and relative density (D_r), which may be considered reliable and accurate by the practising engineer,

TABLE 9

Summary of Results

Type of Test Bed	Meyerhof		Trofimenkov		Janbu and Senneset		Sohmert mann		Durgunog -lu and Mitchell			Observed Value	
	$<\phi'>$	ϕ'_{av}	ϕ'_{av}	ϕ'_{av}	a_{av} t/m²	ϕ'_{av}	$D_{r_{av}}$	$<\phi'>$		ϕ'_{av}	ϕ'	C t/m²	Dr (per cent)
Loose Sand F.S.T.1	34.5-36	35.0	32.0	36	1.5	36.5	41.5	38.5 45	—	41.5	33	—	4.0
Medium Dense Sand F.S.T. 2	35.0-40	37.5	32.5	39	1.5	38.5	60.0	41.5 46	—	43.5	37	—	50
Silty Soil F.C.T. 1	—	—	—	35	2.2	—	—	—	—	—	32	—	—

In view of above objectives, an electric cone penetrometer has been developed and tested successfully in laboratory and field. The developed penetrometer contains a load cells of strain gauge type to measure cone resistance and sleeve friction simultaneously and continuously up to penetrated depth. The theories available in literature relating to interpretation of CPT results have been reviewed.

The comparision of existing theories with the field test results indicate that Trofimenkov method provides over conservative estimates of ϕ'. Janbu and Senneset methods and Durgunoglu and Mitchell method gives high ϕ' values than the actual measured values. Both Schmertmann and Meyerhof methods provide a reasonable agreement with directly measured values and may be used by practising engineer for estimating the ϕ' values of conhesionless soil. In ordinary situation the ϕ' values estimated from these methods should be within the range of $\pm 3^\circ$. The friction ratio concept can be used for a crude estimation of subsurface materials.

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