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CONE PENETRATION TESTING FOR FIELD DENSITY PREDICTION

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ABSTRACT

This paper summarizes an experimental program for developing a pilot procedure to check and predict field densities of compacted soils using static cone penetration tests. Three sandy soils were tested. The testing program included both laboratory and field tests. On the basis of laboratory tests, density prediction curves were developed while field results were used to find a correlation to predict densities on the basis of measured penetration resistances. The results are rather encouraging and promising. It is anticipated that the presented methodology would be very effective for compaction quality control in large areas of sandy soils because it is fast, simple and causes minimum disturbance to the compacted soil.

NOMENCLATURE

a,b	=	regression constants
CC	=	calibration chamber
K_o	=	coefficient of lateral earth pressure at rest
K_q	=	a factor
NC	=	normally consolidated
OCR	=	overconsolidation ratio
q_c	=	cone point resistance
q_{cf}	=	field cone point resistance
Dr	=	relative density
σ_{vo}'	=	vertical effective stress
σ_{ho}'	=	horizontal effective stress
ρ	=	density
ρ_{dm}	=	measured density
ρ_{dp}	=	predicted density

INTRODUCTION

It is generally accepted by geotechnical engineers that static cone penetrometers could be utilized to get good estimation of soil parameters such as density, angle of internal friction and, soil moduli; and in evaluating bearing capacity, settlement and shear strength of foundation soils. Efforts have been made to calibrate cone data against these soil parameters, e.g. Parkin and Lunne (1982), Sanglerat (1972), Mitchell and Lunne (1978), De Ruiter (1982) and Simone (1988).

The use of penetrometers for compaction control dates back to the early studies by Proctor in 1933, who developed a simple spring-loaded penetrometer, referred to as the Proctor needle (Hausmann 1990). The instrument has been primarily devised for moisture control in embankment construction. It is standardized by ASTM Moisture - Penetration Resistance Relations of Fine - Grained Soils, D 1558. Another apparatus that employed penetration for compaction quality control of compacted moist sands, the comprimeter, was introduced by Eggestad (1974). The comprimeter was tried in field works and it showed promising performance.

Cohesionless soils could not be sampled without affecting their state. Therefore, in-situ measurements of density are necessary. The conventional density control and measurement tests such as sand cone, rubber balloon, nuclear density meter have their limitations especially when loose or submerged sandy soils are encountered. In such situations, these tests would be very difficult to perform. In such cases penetration tests such as the standard penetration test (SPT) or the cone penetration test (CPT) may be performed to estimate the in-situ densities of soil formations.

A recent study conducted by the authors (Baghdadi et al, 1988) indicated that static cone penetrometer could be used to predict the density of cohesionless soils at shallow depths. Thus there is a great potential in using the cone penetrometer in compaction control of man made fills, bases and subbases of roads, and densification of natural soils, with the advantages of expedience and ease of testing, economy and minimum disturbance of the finished surface. This paper summarizes a further extension of the work in that direction, where laboratory and field tests results are combined to develop a procedure for checking and predicting field densities of compacted soils on the basis of field penetration resistances.

Background

The idea of pushing rods into the ground to determine the strength of subsurface soil is a very old one. The split spoon sampler has been used as a sounding and sampling device for estimating densities, (Gibbs and Holtz, 1957). Standard penetration tests have been used as an indirect method of evaluating soil properties of subsurface soils with an advantage that the sample is retained for inspection. However, some articles and references which are available in the engineering literature such as Schmertmann (1970) and Sutcliffe and Waterton (1983), concerning static penetrometers underlined the possible superiority of the static cone penetration tests.

Penetration tests for quality control of compaction and density have already been attempted in different parts of the world. A sample of such utilization follows.

1- A 2 ton penetrometer was used to control compaction of runways for the Leopoldville - Kinshassa airport in Congo (Sanglerat, 1972). The values of cone point resistance, qc, in the quartzitic sand ranged from 1500 - 2000 kPa before compaction and increased to a range of 5000 - 7456 kPa after compaction. As a result of these tests, specifications for the control of compaction were set up based on the use of the penetrometer. The control method was found very effective for sands where water content was uniform.

2- The static cone penetrometer has been used on various occasions in France for the control of compaction of fills (Sanglerat, 1972). A quality of compaction of fills in terms of point resistance (q_c) of the static penetrometer for gravelly and sandy fills for the Rhone-Alpes area was summarized and that permitted a quick method of determining the degree of compaction of fill project in the Rhone-Alpes area.

3- Mitchell (1986) discussed utilization of in-situ tests in design and evaluation of a large sand densification project for the Jebba Hydroelectric Development in Nigeria. He reported that correlations between CPT tip resistance, relative density and depth were used successfully to assure that the required ground improvement had been achieved.

4- Several research studies with penetrometers have utilized large diameter calibration chambers (CC). A sample of soil at a known density is prepared in the chamber and then consolidated to the desired stress level. The cone is pushed into the soil and the tip resistance and sleeve friction recorded. Laboratory tests are then conducted to determine engineering properties of the soil in the chamber. The cone resistance is then related to relative density and the engineering properties in turn defined as a function of relative density. Parkin and Lunne (1982) and Parkin (1988) presented results of investigations carried out on calibration chambers to estimate parameters such as friction angle ϕ' and moduli of sandy and clayey soils.

It was also indicated that the cone tip resistance can be used to estimate relative density provided that the OCR (overconsolidation ratio), or lateral stress is known. Correlations were obtained for NC (normally consolidated) or unaged sands, and OC (overconsolidated) sands (Ko > 0.45), relating relative density (D_r), tip resistance (q_c) and vertical effective stress (σ_{vo}') or mean effective stress (σ_m').

Jamiolkowski et al (1985) presented a correlation worked out by Lancellotta from a regression analysis of well documented CC tests for NC sands of varying compressibilities, mainly on the assumption that a linear relationship is postulated between D_r and $\log_{10} q_c/\sqrt{\sigma_{vo}'}$. The obtained D_r values may be corrected for the chamber size effect (Parkin and Lunne, 1982) by dividing the field measured q_c value by a factor K_q:

$$K_q = 1 + \frac{0.2(D_r - 30)}{60}$$

The chamber size effect will lead to an overestimate of D_r, the amount increases as D_r increases. Robertson and Campanella (1989) recommended using Baldi's relationship to predict D_r of NC sands ($K_o \leq 0.45$), and then adjusting for compressibility using Lancellotta's correlation given by Jamiolkowski et al.(1985). For OC sands the lateral stress σ_{ho}' should be estimated first, then use Baldi's relationship.

EXPERIMENTAL PROCEDURE

Three soils were selected for this investigation. These soils were selected to represent the most predominant types in the western region of Saudi Arabia according to Ahmed et al (1986). These three soils belong to the A-1-b (SP), A-2-4 (SM) and, A3 (SP) groups according to the ASTM Classification of Soils and Soil-Aggregate Mixtures For Highway Construction Purposes, D 3282 (and ASTM Classification of Soils For Engineering Purposes, D 2487). The selected soils were named A, B and C, respectively. The soils were sieved to a maximum grain size of 2 mm for the testing program. The physical parameters for the three soils and grain size distribution curves are shown in Table 1 and Fig. 1.

In the laboratory work, samples of the soils were compacted in dry and partially saturated conditions. The dry samples were statically compacted in steel molds (152 mm dia. and 120 mm height) prior to penetration tests; while the partially saturated samples were compacted in the molds in a manner similar to the modified Proctor procedure (25 blows per layer) in accordance with ASTM

Table 1: Classification of the Tested Sands

Soil Data	Soil A	Soil B	Soil C
Source	Makkah Road	KAU Campus	East of Jeddah
Colour	Light Gray	Yellowish Brown	Light Yellow-ish Brown
Shape of particles	Angular to Subangular	Subrounded to Subangular	Rounded to Subrounded
Unified classification ASTM D 2487	SP	SM	SP
AASHTO classification ASTM D 3282	A-1-b	A-2-4	A3
The effective size, D10 mm	0.180	0.068	0.103
The median grain size D 50 mm	0.513	0.167	0.225
Coefficient of uniformity, Cu	3.39	3.72	2.47
Maximum grain size, mm	2.0	2.0	0.85
% passing No. 200	1.38	12.68	3.48
Specific gravity, Gs	2.68	2.73	2.69
Max. dry density, kg/m^3 ASTM D 1557	1878	1854	1710
Min. dry density, [(1)] kg/m^3	1580	1583	1546
Optimum water content, [(1)] %	10.73	11.17	13.10

(1) ASTM minimum index density of soils and calculation of relative density D 4254.

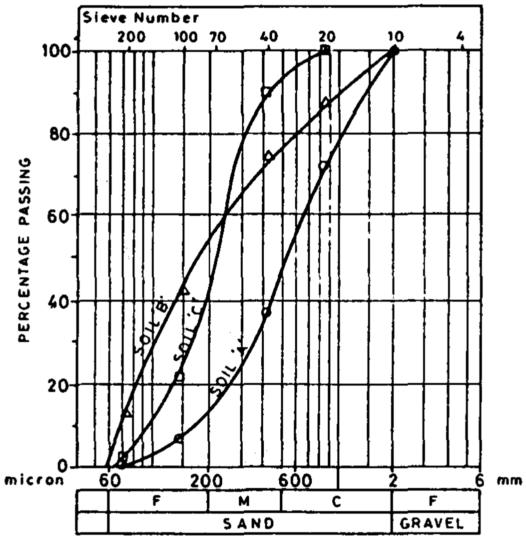

Figure 1: Grain size distribution curves for soils A, B and C

Moisture-density Relations of Soils And Soil-Aggregate Mixtures Using 10-lb (4.540kg) Rammer And 18-in.(457-mm) Drop, D 1557. In both cases surcharge weights were mounted on the samples during penetration. In the dry condition case six surcharge pressures: 0.39, 2.5, 4.7, 6.8 and 11.2 kPa were used and which were assumed to represent soil surcharges corresponding to depths of up to 0.7 m of soil overburden. While for partially saturated samples, 0.39, 4.7 and 9 kPa were employed.

The motorized penetration assembly in the laboratory shown in Fig. 2 consists mainly of a cone 10 cm^2 in projected base area with a 60° apex angle as specified by ASTM Deep Quasi-Static, Cone And Friction-Cone Penetration Tests of Soils, D3441. The cone ic connected to a proving ring by a steel shaft where the cone tip resistance q_c is read. Friction on the shaft is averted by encasing the cone shaft with an external hollow pipe connected to the upper part of the frame. The motor was calibrated to produce a penetration rate of 20 mm/sec (ASTM D 3441). At the end of compaction of each sample of each soil, cone penetration was carried out and the penetration resistance at a depth of 50 mm from the top was recorded.

In the field, the cone penetrometer assembly was mounted on a water tank truck as shown in Fig. 3. Sufficient amounts of soils A, B and C were transported to the site where sections of 4.5 m long, 2.5 m wide and 0.4 m deep each of the soils were prepared (Fig. 4). Each section was formed by compacting the soil in layers while sprinkling water on the soil, using a single drum wheel vibratory

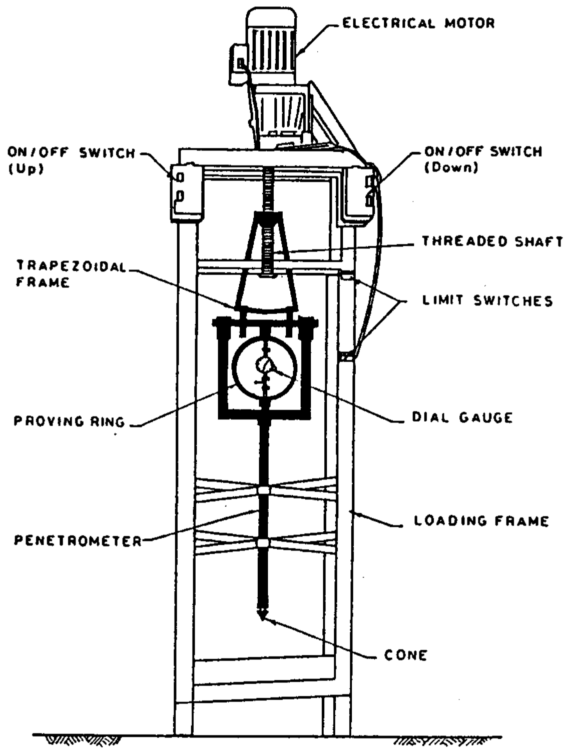

Figure 2: Cone penetrometer assembly in the laboratory

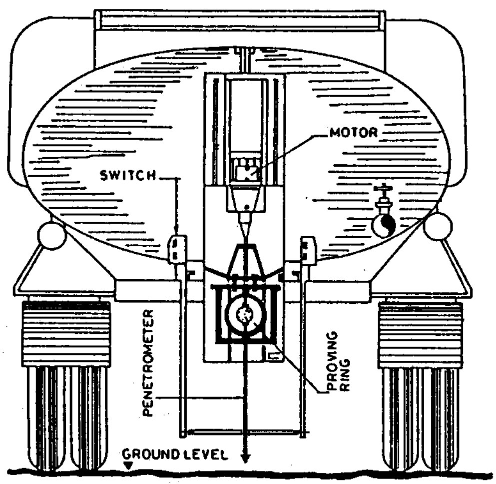

Figure 3: Cone penetrometer assembly in the field

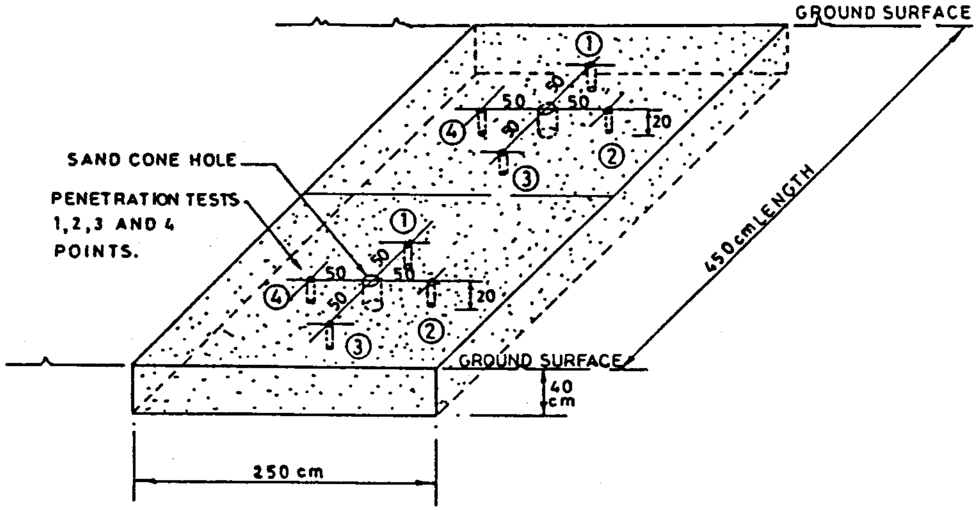

Figure 4: Soil section in the field

roller. Two sand cone density tests were performed in each soil section. These were run according to ASTM Density of Soil In Place By The Sand-Cone Method, D 1556. This test was selected since it resulted in closest density values to "actual" field densities compared with other methods according to a study by Alzaydi and Khalil (1988) on a silty sand soil, the error ranged from +0.4 to -0.68% compared to reference densities estimated by dividing weight of soil in a large container by volume. Four penetration tests were carried out for each hole at a radial distance of 0.5 m from the hole center. This distance was chosen so that the effects of the sand cone hole do not influence penetration readings. Penetration resistance values (q_{cf}) were recorded at a depth of 0.2 m (mid-depth). This represents testing of compacted soil lifts of 0.4-m-thick layers. Details of apparatus and the testing procedures are given in Al-Ahmadi (1989).

RESULTS AND DISCUSSION

Initially, samples of the three soils were compacted according to the modified Proctor procedure using molds of larger dimensions (as given earlier) than the regular standard Proctor mold. This was thought to make the penetration test more convenient to run. However, penetration tests with 10 cm^2 in 152 mm diameter steel molds are highly affected by the lateral rigid boundaries. Because of that, the results are not comparable with the field results in absolute values, although they may be in relative terms. Parkin and Lunne (1982) results mentioned earlier, proposed to correct penetration values for boundary effects. These effects were found to increase with higher relative densities (or densities). Unlike calibration chambers tests, the penetration results obtained here are not corrected for boundary effects, but they are used directly (raw) in the analysis which means that the boundary effect is included in the obtained correlation.

The obtained results are presented in Figs. 5, 6 and 7. These figures show that q_c values followed the pattern taken by compaction curves. As the density increases on the dry side of optimum, the penetration resistance increases accordingly with maximum q_c occurring at or near maximum ρd. On the wet side of optimum, the penetration resistance decreases with decreasing dry density. Such behaviour was also reported by Baghdadi et al (1988) on other granular soils. This observation indicates that the degree of compaction or density may be estimated on the basis of cone penetration resistance.

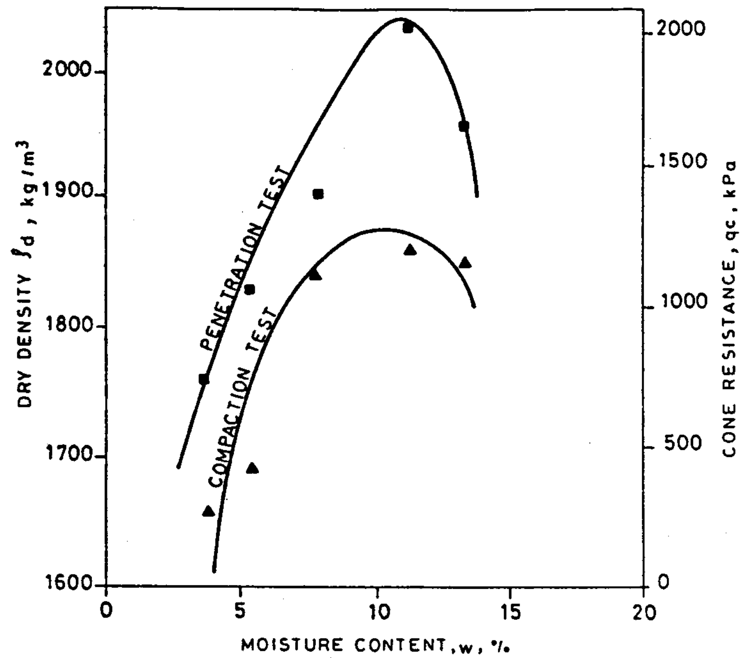

Fig. 5: Compaction and corresponding cone penetration resistance curves, Soil A

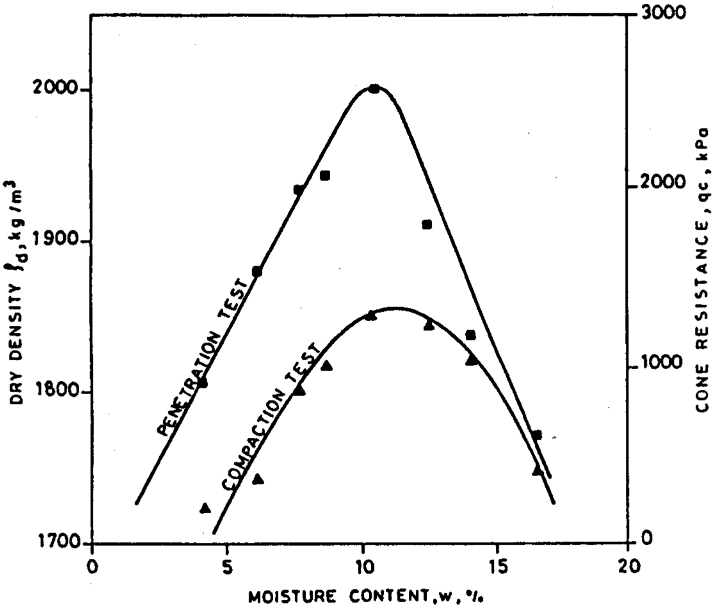

Fig. 6: Compaction and corresponding cone penetration resistance curves, Soil B

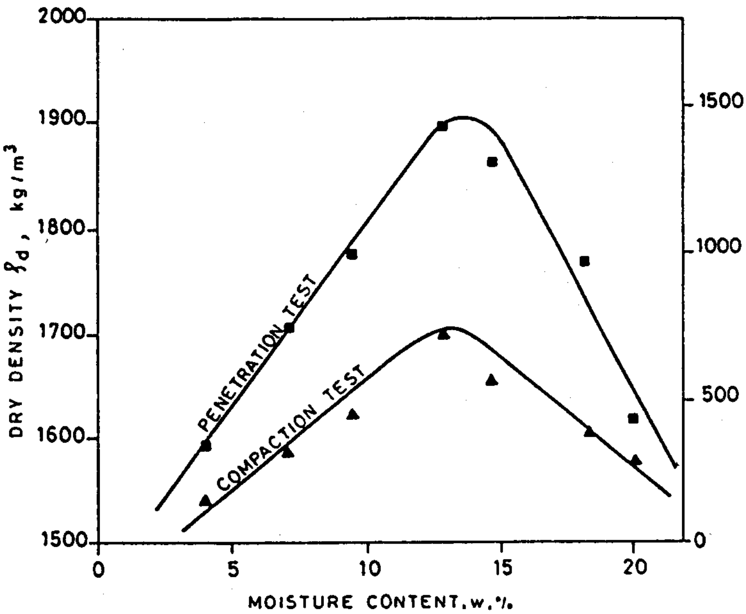

Fig. 7: Compaction and corresponding cone penetration resistance curves, Soil C

In the second stage, soils A, B and C were statically compacted in the molds, in its dry condition. The soils were compacted to densities within their maximum and minimum densities, (Table 1). After compaction, six surcharge weights were used for every soil and for every density. These surcharges resulted in equivalent overburden pressures between 0.39 to 11.2 kPa. The values of surcharge pressures were transformed to equivalent depths of soil surcharges of same densities of samples. This was accomplished by dividing total surcharge stress by the density (unit weight) of soil. The cone penetration resistances obtained were then plotted by linear fitting against depths of equivalent soil surcharges; such a plot is shown in Fig. 8 for soil A as an example. Linear regression was used in obtaining correlations throughout this work because of its simplicity and can be performed easily by a hand calculator. The linear regression correlations along with regression parameters for soils A, B and C are shown in Table 2. The correlation shown in Table 2 indicates a finite value of q_c (intercept) at ground surface. This may have resulted from the size of the mold and boundary effects, in addition to

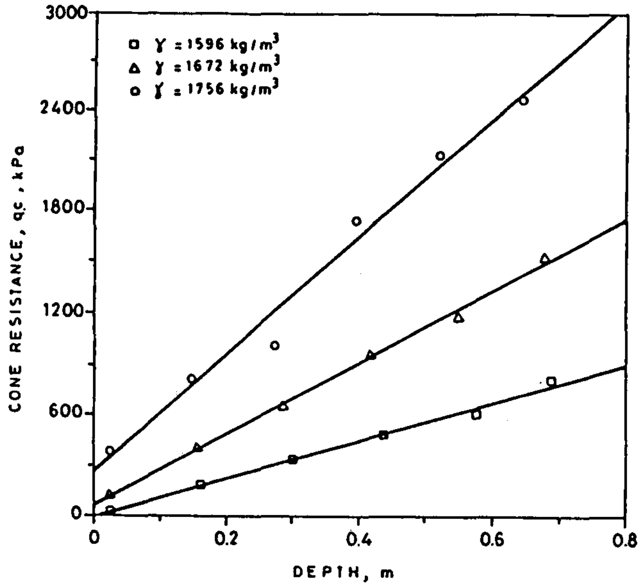

Figure 8: Relationship between depth and cone resistance; soil A; dry state

the effect of induced horizontal stresses resulting from compaction, with much of the horizontal stress remaining locked in after compaction. Been et al. (1986) applied the state parameter Ψ (defined as the difference between the voids ratio e of the sand and its voids ratio e_{ss} at the steady state at the same mean effective

Table 2: Statistical Parameters of Penetration - depth Curves for Soils A, B and C, Dry State. $q_c=a+b*D$

Soil	Density Kg/m^3	Linear Regression eq. : q_c (kPa), D=Depth, m		r (1)	No. of Points
A	1596	-3.01	$+1.132 \times 10^3 D$	0.996	6
	1672	57.54	$+2.1 \times 10^3 D$	0.999	6
	1756	267.80	$+3.42 \times 10^3 D$	0.991	6
B	1583	-3.81	$+9.26 \times 10^2 D$	0.998	6
	1670	1.31x102	$+1.17 \times 10^3 D$	0.983	6
	1760	8.08x102	$+2.71 \times 10^3 D$	0.989	6
C	1546	-1.49x102	$+1.36 \times 10^3 D$	0.963	6
	1593	-1.48x102	$+1.88 \times 10^3 D$	0.980	5
	1644	4.33x102	$+2.96 \times 10^3 D$	0.995	4

(1) r = Sample correlation coefficient.

pressure) concept in analyzing calibration chamber cone penetration tests results. In doing so, he readjusted cone tip resistance data for chamber boundary effects, which made the lines pass close to the origin. Such readjustment was not followed in this paper for the sake of simplicity and direct application of cone penetration results. It remains to be seen if such inaccuracy would hinder the prediction of density.

From those correlations, plots of q_c against dry densities were prepared for depth intervals of 0.1 m up to 0.7 m. It is proposed that these curves be used to predict soil densities for specific penetration depths, as suggested by Baghdadi et al. (1988). The curves so obtained were thus named density prediction curves (DPC), and are presented in Fig. 9 for the three soils.

All DPC's were found in the form:

$$q_c = a + b\rho \qquad (1)$$

where,

q_c = penetration resistance in kPa
a and b = regression constants
ρ = density in kg/m^3

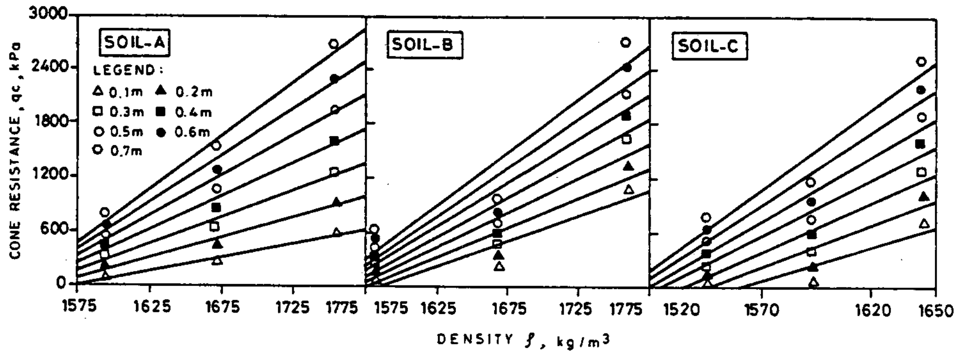

Figure 9: Density prediction curves; soils A, B and C; dry state

The correlations obtained by linear regression analysis for DPC's of the three soils are given in Table 3. Curves in the aforementioned figures resemble those shown by Baghdadi et al (1988) for other sandy soils. Equation (1) indicates finite value of q_c when ρ is set equal to zero. This could be due to boundary effects which were not corrected for from the outset, as discussed previously. Equation (1) represents results of compacted soils, which means that extremely low densities are not covered by the results.

As stated earlier, tests were conducted on partially saturated soils A, B and C. On the basis of the compaction curves shown previously in Figs. 5, 6 and 7, different dry densities and moisture contents were arbitrarily selected to run penetration tests on soil samples subjected to surcharge weights. The tests showed that these cohesionless free-draining soils were not easy to compact and reproduce samples of same water contents and dry densities. Each data point is a result of several trials ($\pm$20 kg/m^3) in order to make sure that the data are reasonably uniform.

The data obtained showed increased penetration resistance with increasing densities and higher surcharge weights. Furthermore, it indicated that penetration resistance was slightly affected by increasing the moisture content from the dry side of optimum to the wet side at same density, indicating virtually drained conditions. The decrease in penetration resistance, illustrated by Figs. 5, 6 and 7, is mainly due to the decrease of density as a result of increased moisture content. This observation is in line with a previous study by Bellotti et al(1988) who investigated three saturation techniques and concluded from the results of cone penetration tests performed on sand in calibration chambers that the resistance is slightly(10-15 %) influenced by saturation. Bellotti et al(1988) also cited other

Table 3: Statistical Parameters of the Density Prediction Curves for Soils, A, B and C. Dry State. $q_c = a + b^*\rho(q_c\text{:kPa}, \rho\text{:kg/m}^3)$

Soil	Line (depth, m)	a $x10^3$	b	No. of Points	r (1)
A	0.10	-4.93	3.14	3	0.984
	0.20	-7.11	4.57	3	0.990
	0.30	-9.28	6.00	3	0.992
	0.40	-11.46	7.44	3	0.994
	0.50	-13.64	8.87	3	0.995
	0.60	-15.81	10.30	3	0.995
	0.70	-18.00	11.73	3	0.996
B	0.10	-8.92	5.62	3	0.936
	0.20	-10.45	6.63	3	0.935
	0.30	-12.00	7.64	3	0.933
	0.40	-13.51	8.65	3	0.932
	0.50	-15.04	9.66	3	0.931
	0.60	-16.57	10.68	3	0.930
	0.70	-18.10	11.69	3	0.929
C	0.10	-11.74	7.53	3	0.904
	0.20	-14.37	9.30	3	0.923
	0.30	-16.79	10.95	3	0.935
	0.40	-19.20	12.59	3	0.944
	0.50	-21.61	14.23	3	0.950
	0.60	-24.03	15.88	3	0.954
	0.70	-26.44	17.52	3	0.958

(1) r : Sample correlation coefficient.

studies by Schmertmann(1976), Lhuer(1986), Caillemer(1976), and Last(1979) to point to the small influence of saturation on the cone penetration resistance (q_c).

The data of the partially saturated soils were treated in exact similar manner as that of the dry state condition discussed earlier except that dry densities were grouped according to their position relative to optimum water contents, i.e., dry of optimum and wet of optimum. Density prediction curves of soil A are only shown in Fig. 10; while the linear regression correlations and parameters for the three soils are given in Table 4. It may be noted that in all cases the correlation coefficients indicate very good correlations.

The final phase of the experimental work conducted in this investigation dealt with the field work. The procedure followed was explained earlier. Sections of soils A, B and C were compacted to selected dry densities on the basis of the dry

Table 4: Statistical Parameters of the Density Prediction Curves Partially Saturated, Soils A, B and C. $qc=a+b*\rho$ (qc: kPa, $\rho=kg/m^3$)

Line Depth, m	SOILS A a (1) x10⁴	A b (2)	A No. of Points	A r (3)	B a (1) x10⁴	B b (2)	B No. of Points	B r (3)	C a (1) x10⁴	C b (2)	C No. of Points	C r (3)
	DRY OF OPTIMUM											
0.10	-2.67	15.88	6	0.976	-3.20	18.85	4	0.910	-2.61	16.70	4	0.994
0.20	-3.316	19.81	6	0.973	-3.91	23.13	4	0.907	-2.95	18.90	4	0.991
0.30	-3.96	23.74	6	0.969	-4.62	27.40	4	0.905	-3.28	21.10	4	0.992
0.40	-4.64	27.85	6	0.963	-5.33	31.67	4	0.903	-3.61	23.30	4	0.989
0.50	-5.25	31.60	6	0.961	-6.05	36.00	4	0.901	-3.94	25.53	4	0.986
0.60	-5.89	35.49	6	0.958	-6.76	40.23	4	0.900	-4.29	27.83	4	0.983
	DRY OF OPTIMUM											
0.10	-2.26	13.44	5	0.995	-1.79	10.66	5	0.984	-3.03	19.41	4	0.992
0.20	-2.72	16.25	5	0.998	-2.182	13.01	5	0.990	-3.43	22.00	4	0.990
0.30	-3.18	191.06	5	0.999	-2.57	15.36	5	0.992	-3.83	24.60	4	0.977
0.40	-3.64	21.87	5	0.999	-2.92	17.47	5	0.989	-4.24	27.30	4	0.968
0.50	-4.10	2.47	5	0.998	-3.36	20.10	5	0.992	-4.62	29.80	4	0.960
0.60	-4.56	2.75	5	0.997	-3.74	22.40	5	0.991	-5.04	32.50	4	0.952

(1) kPa (2) kN.m/kg (3) Sample correlation coefficient

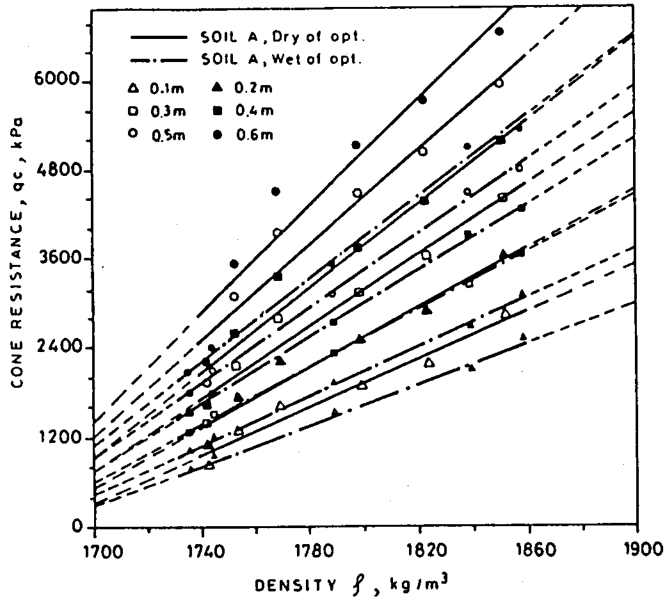

Figure 10: Density prediction curves; soil A; solid lines: range of test densities

of optimum side of the compaction curves shown in Figs. 5, 6 and 7. Field penetration resistances (q_{cf}) were recorded at mid-depth of the compacted section (0.2 m). Each q_{cf} value was the average of four readings and every density reading was the average of two sand cone tests. In this regard, there was little variation in q_{cf} and density readings for every density tested which indicated good uniformity in test conditions and good reproducibility in using the cone penetrometer. The test results are shown in Fig. 11.

The main field difficulty was the high temperatures (about 35 to 40°C), and the big temperature variation between morning and evening as compared to the laboratory temperature which was constant at 23°C. The high temperature and prevailing wind in the field caused high evaporation with the result that measured water contents were significantly lower than the initial ones, by the end of penetration testing. In some cases, the soils' densities were greater than gdmax and the field compactive effort was estimated to be higher than that employed in the laboratory. Such densities resulted in significantly higher penetration resistances and these compacted sections were abandoned and prepared again.

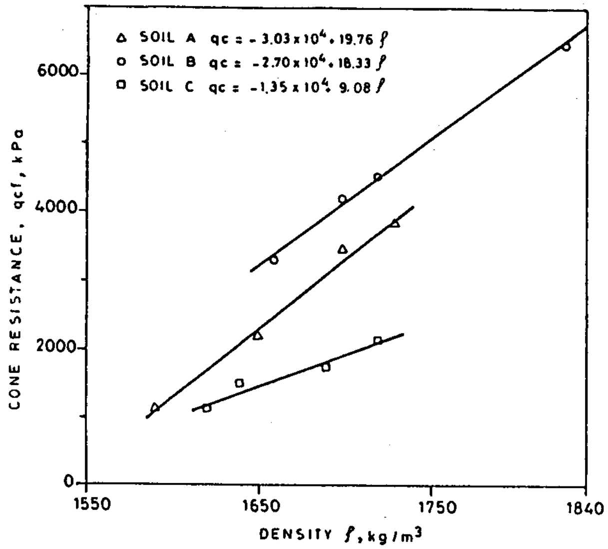

Figure 11: Field data of cone resistances and densities

Since the field data were taken at a depth of 0.2 m, the DPC's curves of 0.2 m, dry of optimum for the three soils (Fig. 12) were employed in subsequent discussion. Comparison of laboratory and field results, showed that at similar densities, field penetration resistances were higher than those of the laboratory. The difference is probably due to the climatic conditions (drying) in the field in contrast to the stringent control of testing conditions in the laboratory and imposed boundary conditions.

The main purpose of this work is to find a rather simple methodology to correlate laboratory and field results and assess the possibility of predicting field densities from laboratory-based density prediction curves (DPC), using field cone penetration resistances (q_{cf}). The measured field density (ρ_{dm}) and penetration resistance data along with ρ_{DPC} values (0.2 m depth) for the three soils (A, B and C) were combined in one set. After examining regression possibilities, multiple linear regression was tried first but resulted in multi-collinearity between ρ_{DPC} and q_{cf}. To solve the problem of multi-collinearity and thereby stabilize the coefficients of the regression model, ridge regression (Walpole and Meyers, 1989) is adopted.

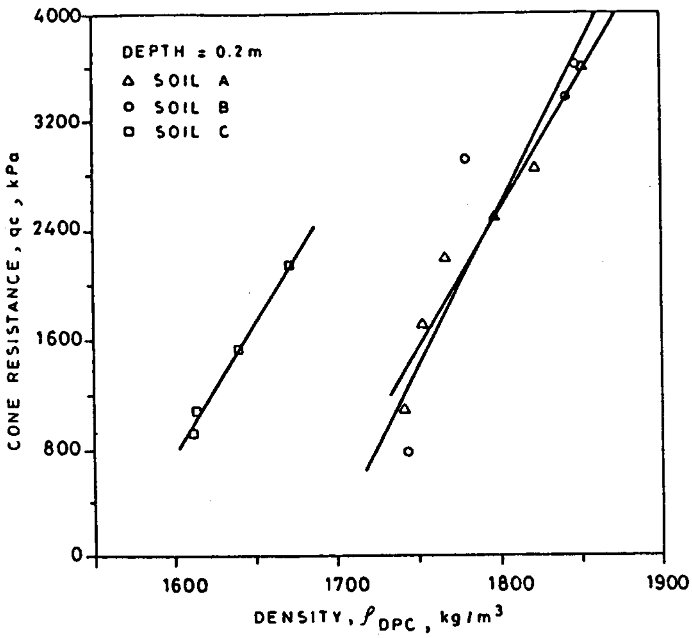

Figure 12: Density prediction curves of soils A, B and C; dry of optimum densities for the penetration depth of 0.2 m

The model obtained is:

$$\rho_{dp} = \beta_o + \beta_1 \rho_{DPC} + \beta_2 q_{cf} \tag{2}$$

where

ridge parameter estimate, K	= 0.0223
ridge regression coefficients:	
β_o	= 659.8631
β_1	= 0.5808
β_2	= -0.0029
standard error	= 57.97
correlation coefficient	= 0.999
F value	= 6.4

ρ_{dp} : predicted (dry) density in the field, kg/m^3

ρ_{DPC} : dry) density from DPC (Fig. 12), obtained on the basis of q kg/m^3

q_{cf} : field penetration resistance, kPa

Table 5 illustrates the application of equation (2) using data obtained in this experimental work. For the limited set of data used in developing and applying the correlation for the tested soils, the results are quite reasonable as indicated by the scatter diagram shown by Fig. 13. The average error in predicting field density

Table 5: Field Calculated Densities on the Basis of Correlation (2) for Soils A, B and C.

Soils	DPC kg/m³	qc kPa	ρ_{dm} kg/m³	ρ_{dp} kg/m³	Error %
A	1733	1128.1	1590	1663.1	+4.60
	1784	2138.6	1650	1689.8	+2.40
	1850	3433.5	1700	1724.4	+1.43
	1866	3776.8	1730	1732.7	+0.16
B	1833	3286.4	1660	1714.9	+3.31
	1872	4208.5	1700	1734.9	+2.10
	1887	4542.0	1720	1742.7	+1.32
	1970	6464.8	1830	1785.3	-2.44
C	1620	1118.3	1620	1597.5	-1.39
	1641	1520.6	1640	1608.5	-1.92
	1652	1716.5	1690	1614.4	-4.47
	1675	2158.2	1720	1626.4	-5.40

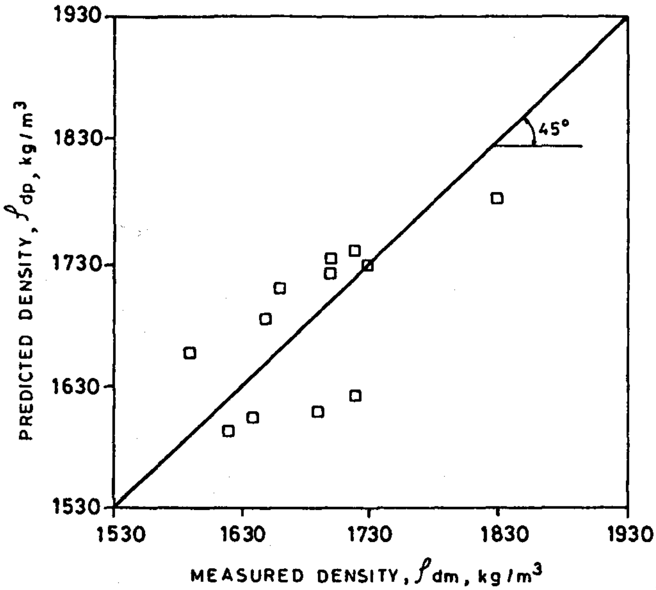

Figure 13: Field densities (ρ_{dm}) against predicted densities (ρ_{dp}) based on field penetration resistances and DPC readings, for all three soils

ranged from +2.19 to -3.12%. This compares well with other methods of field density tests such as the rubber balloon (+4.1 to -0.67%) and nuclear density meter (+8.9 to -1.3%) as reported by Alzaydi and Khalil (1988).

Having this model (2) all that is needed is to run field penetration tests on compacted soil layers. For the technique to be feasible, it should be applied to large areas where it could save time and effort. It should be realized that the obtained model is applicable solely to the tested soils and perhaps to other soils of similar characteristics and for penetration depths of 0.2 m.

CONCLUSIONS

The results and discussion presented in this paper show that it is possible to use static cone penetration testing to predict field densities of compacted soil layers on the basis of laboratory penetration tests. The proposed pilot procedure is simple and easy to apply. The obtained predicted densities are in good agreement with the measured ones. It is proposed that for soils of similar characteristics as those in this investigation, correlation (2) be checked and used. More work is definitely needed to include more granular soils and to fine tune the correlation.

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REFERENCES

1. **Ahmed, S.N., Ghazali, F.M., and Khan, A.M., 1986,** Classification of some soils of the Western Region of Saudi Arabia as a subgrade material for highway and airports, Proc., Second Saudi Eng. Conf., Dhahran, Vol. 1, pp. 267-284.

2. **Al-Ahmadi, S.M., 1989,** Use of static cone penetrometer for control of compaction of cohesionless soils, M.S. Thesis in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering, King Abdulaziz Univ., Jeddah, Saudi Arabia.

3. **Alzaydi, A.A., and Khalil, M.B., 1988,** Errors of in-place density measu rements for quality control in road construction, Proc., 3rd IRF Middle East Regional Meeting, Riyadh, Vol. 3, pp. 3.89-3.101.

4. **Baghdadi, Z.A., Ghazali, F.M., and Khan, A.M., 1988,** Density prediction using a static cone penetrometer, Proc., 1st Int. Symp. On Penetration Testing, Orlando, Vol. II, pp. 635-641.

5. **Been, K., Crooks, J.H.A., Becker, D.E. and Jefferies, M.G., 1986,** The Cone Penetration Test in Sands: Part I, State Parameter Interpretation, Geotechnique 36, pp. 239-249.

6. **Bellotti, R., Crippa, V., Pedroni, S., and Ghionna, V.N., 1988,** Saturation of sand specimen for calibration chamber tests, Proc., 1st. Int. Symp. On Penetration Testing, Orlando, Vol. 2, pp. 661-671.

7. **Caillemer, B.M., 1976,** An Experimental Study in the UF Static Cone Test Calibration Chamber, Part 1-Density Distribution of Plurially Placed Sand. Part 2-Pore Pressure Distribution Around Fugro Tip. M.Sc. Thesis - University of Florida.

8. **De Ruiter, J., 1982,** The static cone penetration test state-of the art report, Proc., 2nd European Symp. On Penetration Testing, Amsterdam, Vol. II, pp. 389-405.

9. **Eggestad, A., 1974,** A New Method For Compaction Control of Sand, Geotechnique 24, No. 2, pp. 141-153.

10. **Gibbs, H.J., and Holtz, W.G., 1957,** Research on Determining the Density of Sands By Spoon Penetration Testing, Proc. 4th ICSMFE, Vol. 1, London, pp. 35-39.

11. **Hausmann, M.R., 1990,** Engineering Principles of Ground Modification, McGraw-Hill, pp. 90-91.

12. **Jamiolkowski, M., Ladd, C.C., Germaine, J.T., and Lancelotta, R., 1985,** New Developments in Field and Laboratory Testing of Soils, Theme Lecture, Proc., XI, ICSMFE, Vol. 1, San Francisco, pp. 118-121.

13. **Last, N.C., 1979,** The Introduction of Cone Penetration Tests on Saturated Samples of Hokksund Sand in the NGI Calibration Chambers, Internal Report, Norwegian Geotechnial Institute.

14. **Lhuer, J.M., 1986,** An Experimental Study of Quasi - Static Cone Penetration In Saturated Sands, M. Sc. Thesis, University of Florida.

15. **Mitchell, J.K., 1986,** Ground improvement evaluation by in-situ tests, Use of In-Situ Tests In Geot. Eng., Geotechnical Special Publication No. 6, ASCE, pp. 221-236.

16. **Mitchell, J.K., and Lunne, A., 1978,** Cone resistance as measure of sand strength, Journal of the Geot. Eng. Div., ASCE, Vol. 104, No. GT7, pp. 955-1012.

17. **Parkin, A.K., and Lunne, T., 1982,** Boundary effects in the laboratory caliberation of a cone penetrometer for sand, Proc., 2nd European Symp. On Penetration Testing, Vol. II, Amsterdam, pp. 761-768.

18. **Parkin, A.K., 1988,** The Calibration of Cone Penetrometers, Proc., 1st Int. Symp. On Penetration Testing, Orlando, Vol. 1, pp. 221-243.

19. **Robertson, P.K., and Campanella, R.G., 1989,** Guidelines for Geotechnical Design Using The Cone Penetrometer Test and CPT with Pore Pressure Measurements, Hogentogler and Co., Inc. pp. 67-74.

20. **Sanglerat, G., 1972,** The penetrometer and soil exploration., Elsevier pp. 320-322.

21. **Schmertmann, J.H., 1970,** Sand densification by heavy vibratory compactor, Disc. Proc., paper 6656, Journal of the Geot. Eng. Div., ASCE 96, SM1, pp. 363-365.

22. **Schemertmann, J.H., 1976,** An Updated Correlation between Relative Density, Dr and Fugro-Type Electric Cone Bearing, q_c, Contract Report-23 DACW 39-76M 6646-Waterways Experimental Station, Vicksburg, Miss.

23. **Simone, P.D., 1988,** Theoretical analysis of the cone penetration test in sands, Proc., 1st Int. Symp. On Penetration Testing, Orlando, Vol. II, pp. 729-735.

24. **Sutcliffe, G., and Waterton, C., 1983,** Quasi-static penetration testing, Proc., of an extension course on in-situ testing for geot. investigations, A.A. Balkema, pp. 33-48.

25. **Walpole, R.E., and Meyers, R.H., 1989,** Probability and Statistics for Engineers and Scientists, 4th edition, Macmillan Publ. Co., pp. 452-460.