The Interpretation of Static Cone Penetration Tests

by

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Introduction

The sounding method can be divided into two main groups *viz.*, dynamic and static. Dynamic penetration tests are most widely used for determining allowable soil pressures and relative densities of sandy soils, whereas the static cone penetration test (SCPT) has been originally developed and used in Holland and Belgium (1936) to obtain relative consistency of cohesive soils. Now, it is being used extensively in various parts of the world for developing empirical design parameters of piles in sands and silts and qualitative determination of relative density of sandy soils. It has also been used to estimate the bearing capacity and settlement of foundations on cohesive and cohesionless soils.

Generally, two types of static penetrometers are in use now-a-days *viz.*, (*i*) The Dutch cone penetrometer and (*ii*) Friction jacket cone penetrometer. The refined Dutch cone penetrometer operation with mechanical transmission has been shown in Figure 1. In this test a 60° cone with cross-sectional area of 10 sq cm is forced into the ground at constant strain rate of 1 cm/sec and provision is made to measure independently the point resistance and the total resistance. The friction or the mantle friction (f_c) is the difference between the total resistance (Q_t) and the cone resistance (q_c). The Dutch cone penetrometer is more popular and is generally performed for determination of the cone resistance.

The second type of penetrometer is the friction jacket cone developed by Begemann (1953), shown in Figure 2. The cone is 10 sq cm in circular base area, with a 60° apex. A 13.3 cm long friction jacket is provided between the cone and the casing in such a way that the underside of the friction sleeve is at 12 cm from the cone. At each test interval, the friction jacket cone provides three parameters *i.e.* cone resistance (q_c), friction ratio (FR) and mantle friction (f_c). Hence, the friction jacket cone can also be used for the normal sounding apparatus *i.e.* Dutch cone.

Factors Influencing the Penetration of a Static Cone

Grain Size and Gradation

Kahl and Muhs (1952) reported that the cone resistance in a non-uniform sand-gravel mixture is lesser than in a uniform sand of the same relative porosity. De Beer (1963) observed experimentally that the crushing of the grains increases with increase in the cone resistance. Shashkov

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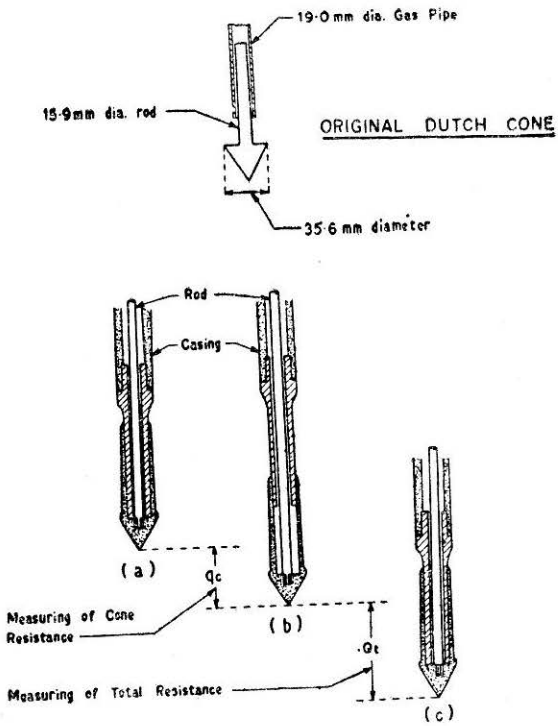

FIGURE 1 Refined Dutch cone penetrometer

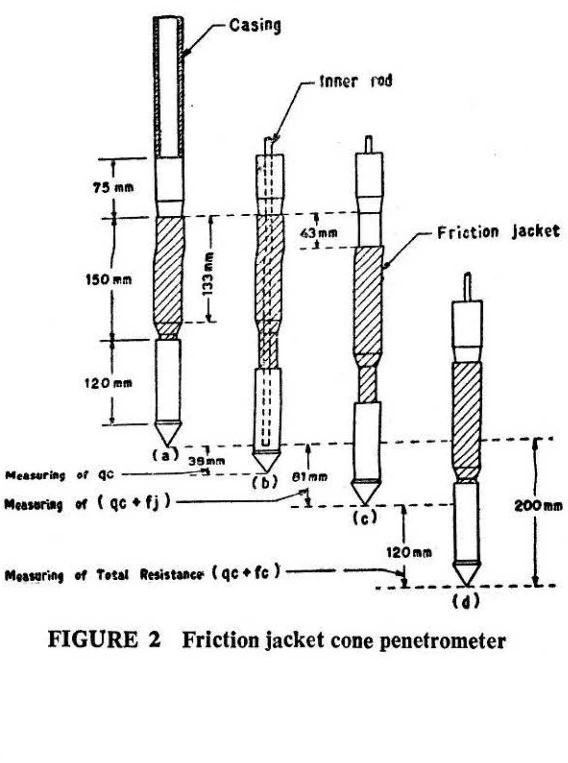

FIGURE 2 Friction jacket cone penetrometer

(1963) observed that the compacted and semi-compacted dusty sands have higher penetration resistance than fine, medium and coarse sands. Doscher (1968) observed a lower cone resistance in a non-uniform coarse sand and sandy gravel and higher cone resistance in uniform fine sand.

Density

Plantema (1957) observed that the cone resistance and the angle of internal friction increase with increasing density. He further observed that the cone resistance is greater for moist sand, least for saturated sand and intermediate in value for the dry sand. On the other hand, the angle of internal friction for a certain sand type and given dry density is greatest for moist sand and least for dry sand and an intermediate value for the saturated sand. Muhs (1965) reports that the cone resistance increases rapidly with the relative density in sand-gravel materials. Schultze and Melzer (1965) found from the laboratory controlled test results that the cone resistance increases rapidly with increasing relative density and overburden pressure. Doscher (1968) reports that the cone resistance increases with depth very quickly and reaches values of 300 kg/sq cm or more when sand was compacted by a vibroflot up to about 100 per cent relative density.

Position of Water-Table

Doscher (1968) found that the sand under G.W.L. has a lower cone resistance. He further observed that the influence of ground water decreases with depth and disappears below a depth of about 3.0 m. Schultze and Melzer (1965) found noticeable change in the measurements after the ground-water level was penetrated *i.e.* under ground-water level the sounding resistance decreased. Dahlberg (1974) observed that the cone resistance decreased by 40 to 50 per cent when the penetrometer point passes into the submerged sand from the capillary zone above the ground-water level.

Overburden Pressure Effect

It is widely recognised that SPT and SCPT-values are a reflection of both density and geostatic stresses. Gibbs and Holtz (1957) were among the first to give a chart, showing the effect of overburden pressure on SPT-value at different relative densities on sands.

Kerisel (1961) reports that the cone resistance in dense fine sand does not vary much after a certain depth has been reached and this critical depth increases with the increase of cone diameter. Chaplin (1963) suggests that the deep sounding results may be very strongly affected by stress ratio and overburden pressure. Doscher (1968) states that the grain-to-grain stress in the reach of the cone depends on the overburden pressure which results in increase of cone resistance with depth in sands having the same density.

Schultze and Melzer (1965) investigated the effect of the overburden pressure on the cone resistance by carrying out controlled laboratory tests on medium to coarse sand using a cylinderical steel shaft of 3.0 m diameter and 5.5 m height. They have given a chart, Figure 3, relating the cone resistance (q_c) to the relative density at different overburden pressure, which shows a very rapid increase in cone resistance even at very low

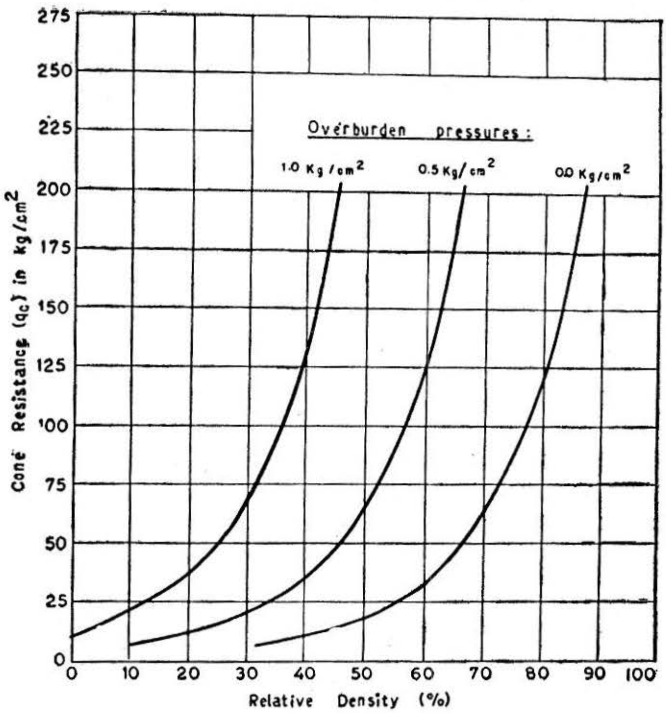

FIGURE 3 Relationship between relative density, cone resistance of SCPT and the overburden pressure in sands without groundwater (after Schultze and Melzer, 1965)

overburden pressures. Dahlberg (1974) carried out SCPT tests from the bottom of an excavation and from two additional levels at 1.4 m intervals in a preloaded natural fine sand. He observed that the cone resistance values obtained at the excavation levels are always lower by 10 to 17 per cent than those conducted from the natural ground surface due to the decrease in effective overburden pressure and lateral stresses.

Review of Established Empirical Correlations

Cone Resistance versus SPT-Value

Meyerhof (1956) and Meigh and Nixon (1961) have given a linear relationship, $q_c=4N$, between the cone resistance (tons/sq ft) and the SPT-value for sands and sandy gravels. Webb (1969) established a linear relationship, $q_c=2N$, between the cone resistance (q_c) in tons/sq ft and SPT-value for fine sand to clayey sands. Schmertmann (1970), Simons (1972), Lacroix and Horn (1973), Sutherland (1974), Peck, Hanson and Thornburn (1974) and Alperstein and Leifer (1976) summarized the q_c/N ratios for various soils and recommended different q_c/N values ranging from 2.0 to 16.0 without considering the influence of overburden pressure.

Dunn (1974) reported the q_c/N ratios from the in-situ results in fine sands at different depth levels below footing depth. He observed that the ratio q_c/N ranges from 3.9 to 5.5 and appears to increase with depth.

Cone Resistance versus Shear Strength

Begemann (1965) gives the relationship between cone resistance (q_c), local friction (f_j) and the apparent cohesion (c_u) by vane test in the clayey and clay-peat layers :

$$f_j = c_u = (q_c - p_0)/13.4 \qquad \ldots(1)$$

For shallow depth, ignoring the surcharge (p_0), he simplified the relation by :

$$f_j = c_u = q_c/14 \qquad \ldots(2)$$

Meigh and Corbett (1969) related the cone resistance (q_c) to overburden pressure (p_0) and undrained shear strength (c_u) for soft clays in the following way :

$$q_c = c_u . N_k + p_0 \qquad \ldots(3)$$

Where N_k is a bearing capacity factor or *cone factor* equal to 16 for this material.

Sanglerat (1972) and Alperstein and Leifer (1976) report a linear relationship, $q_c = 15\ c_u$, between the cone resistance and the undrained shear strength (c_u) for soft to stiff clays. Thomas (1965), Ward, Marsland and Samuels (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.

Cone Resistance versus Bearing Capacity

Meyerhof (1956) established empirical equations between the allowable bearing pressure (q_a) in tons/sq ft and the cone resistance (q_c) in tons/sq ft on the basis of the Terzaghi and Peck's (1948) penetration-allowable pressure chart and the correlation, $q_c = 4N$, for dry and moist sands:

$$q_a = q_c/30 \quad \text{for } B \leqslant 4 \text{ ft} \qquad \ldots(4)$$

or

$$q_a = q_c(1+1/B)^2/50 \quad \text{for } B > 4 \text{ ft} \qquad \ldots(5)$$

and

$$q_a = q_c/40 \quad \text{for rafts} \qquad \ldots(6)$$

where B is footing width in ft and q_c is average cone resistance within depth B below base level.

Meyerhof (1956) again made use of SCPT results for predicting the bearing capacity of piles. He worked out the approximate relations, $f_s = 2f_c = \bar{q}_c/200$, for unit skin friction (f_s) of piles, the static skin friction (f_c) on the penetrometer shaft and average cone resistance ($\bar{q}_c$) in tons/sq. ft within the depth penetrated by pile. Then on the basis of this correlation and the original bearing capacity equation of a pile, $Q_f = \bar{q}_p . A_p + f_s . A_s$, he presented the ultimate bearing capacity of driven displacement piles on sands the following relation:

$$Q_f = q_c . A_p + q_c . A_s/200 \text{ (Tons)} \qquad \ldots(7)$$

Mohan, Jain and Kumar (1963) reported similar approach for predicting the safe load on a pile by working out an empirical equation, $f_s = q_c/50$, for unit skin friction (f_s) on piles.

Scope of Investigation

Based on the published data, it can be said that the strict use of the existing correlations may be misleading and may probably give an incorrect estimate of the in-situ relative density, the angle of internal friction and allowable bearing pressure. From the work of Dunn (1974), it is indicated that the ratio q_c/N for one particular type of soil should increase with increasing depth, since the influence of overburden pressure on q_c and *SPT* values may be different being static and dynamic nature of test procedure. Similar reasoning may also be given for the ratio q_c/c_u, which ranges from 15 to 30 as indicated by various investigators.

The influence of the overburden pressure on the penetration resistance would not affect much any proposed approach to the bearing capacity of piles, since the penetration resistance increases with surcharge roughly in the same proportion as the bearing capacity of piles. Whereas the allowable bearing pressure of spread foundations, can be estimated by allowing the overburden pressure effect on the penetration resistance.

With a view to finding a suitable chart showing the relationship of cone resistance, relative density at different overburden pressure, it was felt necessary to carry out model studies in the laboratory on fine and coarse sands. The study will enable estimation of the allowable bearing pressure, relative density and other soil parameters to be made at certain depth level with reasonable accuracy.

Experimental Investigation

Test Equipment

The experiments consisted of conducting the static cone penetration tests into a prepared sand and clay sample in a cylindrical steel mould of 30.5 cm diameter and 30 cm height, Figure 4. The basic dimension of the penetrometer used were the same as of standard penetrometer having 10 sq cm base area and 60° cone. The penetrometer was coupled to a hydraulic jack of 5 tonne capacity, Figure 5. The complete rig system was connected to a steel frame, mounted on the reaction beam of universal triaxial testing machine (50 tonnes). Dayal and Allen (1975) investigated that the effects of penetration rates are insignificant for granular soils, and for cohesive soils the increase in penetration rate causes an increase in the cone and friction resistances. Hence a penetration rate of 1 mm/sec was adopted for all the tests.

The overburden pressure over the prepared sample was applied with the help of an electrical gear system of the triaxial compression machine using a 25 tonne proving ring and a circular steel loading plate of 300 mm diameter and 20 mm thickness. Two circular holes of 38 mm diameter were made on opposite sides of the centre of the loading plate for SCPT tests. The side of the steel mould was kept away at least 2.5 times the diameter of the cone from the centre of the hole. A hollow steel tube of 60 mm diameter and 375 mm height was used to fill up the gap between the proving ring and the loading plate. To minimise the frictional resistance on the wall of the steel mould during application of the overburden pressure, a thin layer of grease was generally applied on the inner wall side and then covered by a fine polythene sheet.

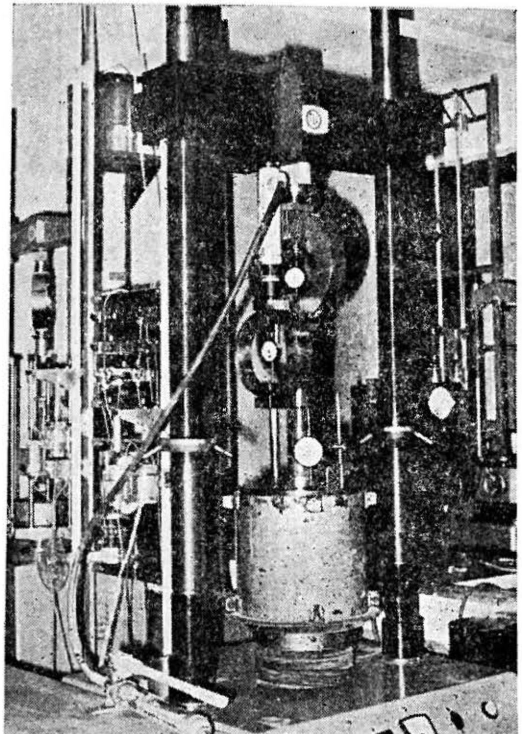

FIGURE 4 Experimental set-up for static cone penetration test

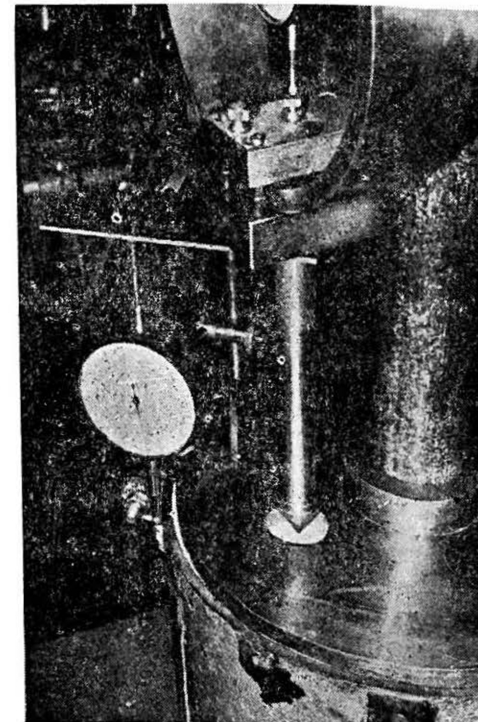

FIGURE 5 Close-up view of static cone penetration test

Test Materials and Sample Preparation

The tests were carried out with different uniformly graded Leighton Buzzard sand of England, Figure 6. The specific gravity of this sand was 2.66. The minimum dry density was determined by slow shaking and pouring by its own weight method in a 1000 ml glass cylinder (after Kolbuszewski, 1948). This was also compared with the pouring method through a small funnel from about 1 cm height in a standard compaction mould. The maximum dry density was obtained by compacting dry sand in 7.5 cm layers with a surface vibrator to the maximum degree of compaction. The diameter of the thin circular steel plate of the vibrator was slightly less than the inner diameter of the steel mould. The intermediate densities were prepared by placing dry sand in 7.5 cm layers and compacting with the aid of a surface vibrator to the required degree of compaction.

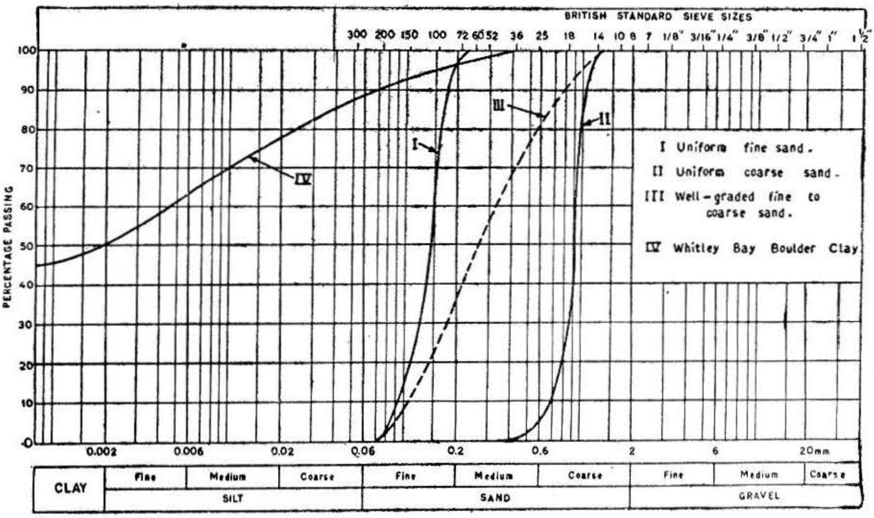

FIGURE 6 Particle size distribution curves

The second series of tests were carried out on Whitley Bay boulder clay of England, which is classified as silty clay, Figure 6. The specific gravity of this clay (LL = 42 per cent and PL = 21.8 per cent) was 2.64. The tests were carried out with two different densities under various overburden pressures. The clay sample, mixed with a measured amount of water, was placed in the testing mould and compacted with a hand hammer (modified AASHO) giving desired number of blows per layer.

Ancillary Tests

A number of direct shear tests were made on specimens of dry sand with three different gradings at the densest and the loosest conditions. The mean angle of internal friction was found to vary from 30° at the loosest condition to 44.6° at the densest condition.

Undrained triaxial compression tests were performed on clay specimens of 38 mm diameter and 76 mm height, obtained at the densities corresponding to those used in the SCPT tests. The confining pressures applied were in the range of 0.7 kg/sq cm. The results of the compression tests are given in Table 1.

TABLE 1

Soil type	*Dry density* kg/m^3	*Water content per cent*	*Cohesion* (c_u) kg/cm^2	*Angle of internal friction* (ϕ) *degrees*
Silty clay	1770	17.7	0.665	5
Silty clay	1570	26.2	0.187	0

Test Results and Discussion

Influence of Overburden Pressure

The static cone penetration tests (SCPT) were carried out on the dry sand of three different gradings and with various relative densities ranging from 25 per cent to 96 per cent. The tests were generally conducted at six different overburden pressures ranging from 0.0 kg/sq cm to 4.0 kg/sq cm. Figures 7 and 8 depict the static cone penetration test results for fine and

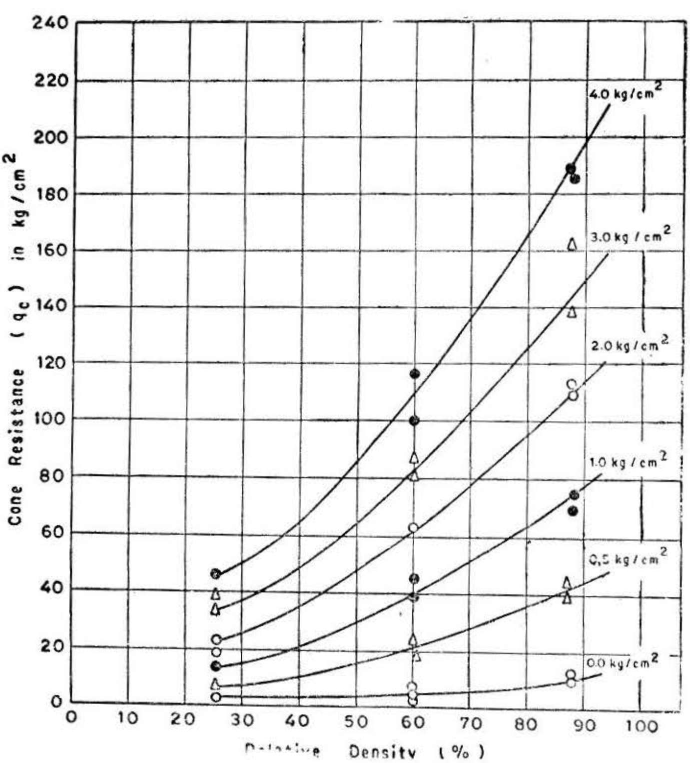

FIGURE 7 Correlation between relative density and cone resistance at different overburden pressure (uniform fine sand)

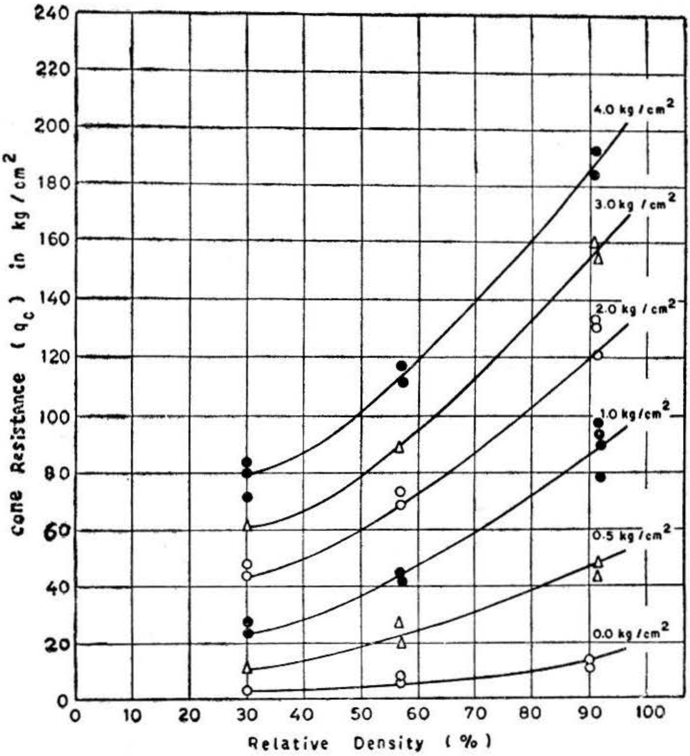

FIGURE 8 Correlation between relative density and cone resistance at different overburden pressures (uniform coarse sand)

coarse sand respectively, showing the relationship between the cone resistance and the relative density at different overburden pressures. Whereas Figure 9 represents a typical result for well graded fine to coarse sand and this chart may be generalised for all correlation purposes. These results show that the cone resistance increases more rapidly with depth than SPT value.

The second series of tests were carried out to find out the approximate relationship between the cone resistance and shear strength of clays at two different strengths found by changing the water content and density. Figure 10 shows the relationship between the cone resistance and the cohesion (c_u) at different overburden pressures for silty clay. From the result, it can be concluded that the cone resistance increases up to twice its value at the surface with increasing overburden pressure up to 4.0 kg/sq cm. With the help of these charts, Figures 9 and 10, we can interpret the SCPT results and hence the soil parameters at any depth.

Effect of Saturation on Cone Resistance

The effect of submergence in the well graded sand has been studied under different overburden pressure after saturating the prepared samples. The resulting relationship has been shown in Figure 11. The decrease in cone resistance due to submergence was found to be about 10 per cent near ground surface *i.e.* zero surcharge condition and about 25 per cent up to overburden pressure of 4.0 kg/sq cm.

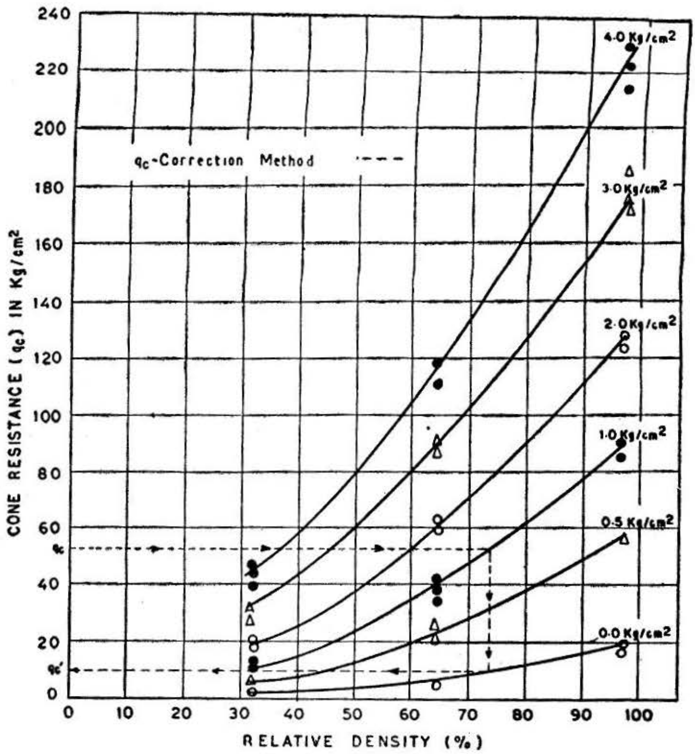

FIGURE 9 **Correlation between relative density and cone resistance at different overburden pressure in well graded sands**

Cone Resistance versus SPT-value

Comparing the SPT-values from Gibbs and Holtz's (1957) chart, with the cone resistance (q_c) values in Figure 9, for corresponding relative densities and overburden pressures, it is concluded that for values at the surface *i.e.* zero surcharge condition, the following relationship is given:

$$q_c \text{ (kg/sq cm)} = N \qquad \ldots(8)$$

This relation is valid for all degrees of compaction immediately near the ground surface. But up to the overburden pressure 2.8 kg/sq cm, the q_c/N ratio increases from 1 to 2.5 in dense sands and from 1 to 4.0 with increasing depth in loose sands. Since the overburden pressure effect in dynamic N-value and static q_c-value are different with depth for similar densities, one single relationship between q_c and N-value is not possible for one particular type of soil.

Penetration-Allowable Pressure Chart

Based on the above correlation, (Equation 8) and on the previous works

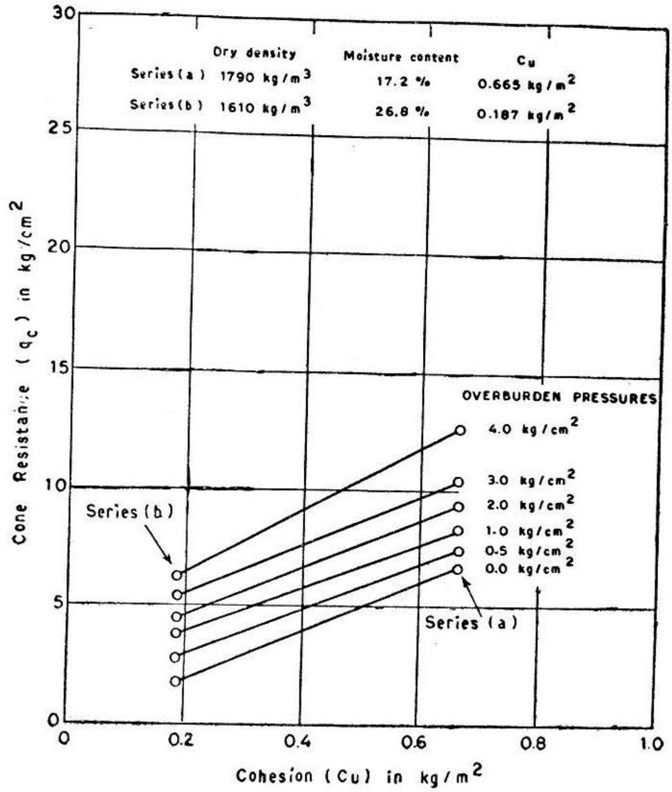

FIGURE 10 Correlation berween cohesion and cone resistance at different overburden pressures (Whitely Bay boulder clay)

of the author (1971 and 1972), a new penetration-allowable pressure chart, Figure 12, is given showing the relationship between allowable soil pressure and corrected q_c, SPT and N_c-values for zero surcharge condition. In Figure 12, the q_c refers to SCPT-values in kg/sq. cm the N refers to SPT-values, whereas N_c refers to the number of blows in dry dynamic cone resistance using 62.5 mm diameter cone.

Before using this new chart, Figure 12, the observed q_c-values at particular depths should first be adapted for zero overburden pressure by means of the SCPT-correction method given below. In this method, a line is drawn vertically downwards from the intersection of the point representing the measured q_c-value and the overburden pressure at that depth, to intersect the zero overburden pressure curve in Figure 9. The point so obtained is then projected horizontally to give the corrected q_c-value.

Interpretation of Relative Density and ϕ

The values of the angle of internal friction (ϕ) together with the corresponding relative densities can be directly correlated with the surface q_c-values in Figure 9, for zero overburden pressure, as both ϕ and relative density are independent of depth. Hence an empirical relation between

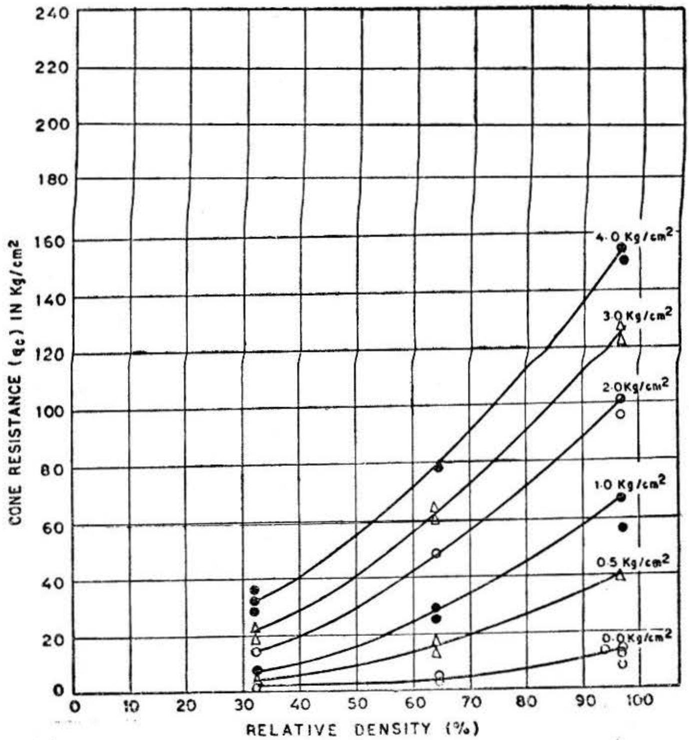

FIGURE 11 Correlation between relative density and cone resistance at different overburden pressures in sands with submergence

surface or corrected q_c-values, relative density and the angle of internal friction (ϕ) can be given as in Table 2.

TABLE 2

Relative density of sands according to the q_c-value corrected for zero overburden pressure

State of packing	*Relative density per cent*	*Corrected N-value blows*	*Corrected q_c-value kg/cm^2*	*Angle of internal friction (ϕ) degrees*
Very loose and loose	0-35	0-1	0-1	<30
Medium	35-65	1-7	1-7	30-36
Dense	65-85	7-14	7-14	36-41
Very dense	85-100	>14	>14	>41

Before using the above Table 2, the measured q_c-value at any depth should first be corrected for zero overburden pressure by the method given

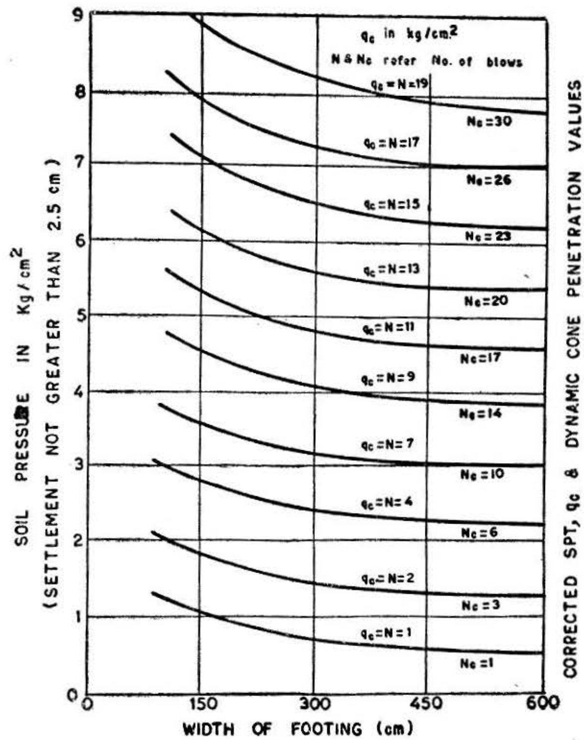

FIGURE 12 New peneration—allowable pressure chart for footings on sand

in the preceding paragraph. From the results, it can be concluded that the increase in cone resistance with depth is higher in uniform sands than in non-uniform sands for lower densities and the increase is more pronounced with increasing grain-size. But for dense sands, the increase in cone resistance with depth is similar irrespective of the gradings.

Cone Resistance versus Shear Strength

Comparing the cone resistance (q_c) values in Figure 10, with the cohesion (c_u) in Table 1, the following relationships are given:

(*i*) At zero overburden pressure;

$$q_c/c_u = 10 \qquad \text{...(9)}$$

where q_c and c_u are in kg/sq cm.

(*ii*) At overburden pressure of 4.0 kg/sq cm

$$q_c/c_u = 20 \qquad \text{...(10)}$$

Hence, one single empirical equation may be given considering the depth effect on cone resistance as :

$$q_c/c_u = 10+2.5\,p \qquad \text{...(11)}$$

where p is overburden pressure in kg/sq cm.

Summarizing the previous work by many investigators, it was indicated that q_c/c_u ratios were in the region of 14 to 30 for soft to stiff clays without considering the overburden pressure effect on q_c-values. However, Meigh and Corbett (1969) considered the surcharge effect on q_c-values as well as on shear strength (c_u) values, found by vane shear tests and the relationship was given by Equation 3. This equation reduces to $q_c/c_u = 16$ for zero surcharge condition, but at one certain critical stage when q_c-value equals surcharge pressure (p_0), the shear strength (c_u) becomes zero giving no definite correlation.

The study indicates that the present recommendations for the interpretation of the measured q_c-values at different depths are misleading. The influence of overburden pressure on q_c-values in sands and clays are given in Figure 9 and Figure 10 respectively. The measured q_c-value may first be corrected with the help of these charts and then related to various soil parameters through the following established correlations:

(*i*) The effect of submergence for sands was found to decrease the measured q_c-values by about 10 per cent near the ground surface and the decrease is more pronounced with increasing depth, decreasing to about 25 per cent at a depth of 4.0 kg/sq cm.

(*ii*) For all degrees of compaction immediately near the ground surface in sands, the q_c (kg/cm^2) is approximately equal to the *N*-value *i.e.* $q_c/N = 1$. But up to the overburden pressure of 2.8 kg/sq cm, the q_c/N ratio increases from 1 to 2.5 in dense sand and from 1 to to 4 in loose sands, with increasing depth.

(*iii*) Allowable soil pressure of sandy soils can be reliably predicted by Figure 12, using the corrected q_c-value for zero overburden pressure.

(*iv*) The relative density and the angle of internal friction of sandy soils may be extrapolated by means of Table 2 using the corrected q_c-values.

(*v*) It was observed that the increase in cone resistance with depth is higher in uniform sands than in non-uniform sands in loose conditions and the increase is more pronounced with increasing grain size. But in dense conditions, the cone resistance increase equally with depth in both types of gradings.

(*vi*) The q_c/c_u ratio for clays may be given by the equation $q_c/c_u = 10+2.5p$, where p is overburden pressure in kg/sq. cm.

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