THE STANDARD PENETRATION TEST 2

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SYNOPSIS

From intuitions, through uses and abuses, into a state-of-the-art report, the SPT is analysed, establishing that it comprises a well-defined penetration phenomenon involving impact energy transmission problems and energy consumed in penetration against point resistance and skin friction forces. The theoretically indicated trends should orient bona-fide statistical regressions in substitution for the existing correlations at random which are confusing. Since in every practical foundation problem many unknown are at play, and it is quite hopeless to solve a single equation for multiple unknowns, it is proposed that the future rational development of the intuitively correct idea of penetration testing-sampling, should be based on the judicious simultaneous use of two or multiple samplers, with characteristics appropriately developed from the "casually invented" ones, for establishment of the necessary number of convenient equations. The reasons for much of the discussion around SPT values may be traced to random variations of significant parameters. Thus, rather than standardizing a single fortuitous equipment and procedure, it is concluded that the aim should be at simultaneous judicious standardization of a number of analogous, mutually complementary, techniques. The confidence limits of various present uses of SPT values are discussed.

I. INTRODUCTION

In the fairly crude manner in which many strictly empirical testing procedures sprouted, the "Standard Penetration Test" arose as an indispensable part of a preliminary subsoil reconnaissance program. Terzaghi's 1947, (297)* first mention of it is: "This test consists in counting the number of blows required to drive a primitive sampling spoon into the soil at the bottom of the hole for a distance of one foot". Indeed primitive and intuitive was the beginning, but, having thereupon been rather authoritatively presented to the practice of the profession [Terzaghi and Peck, (300)] the Standard Penetration Test although as yet recognizedly unstandardized in many respects [cf. Ireland et al., 1970 (126)] has acquired and presently retains a position of the most widespread use.

There are undisputably cogent reasons for some such rugged test of preliminary subsoil reconnaissance to persist in widespread use. Frequently in the face of the crudeness of the test it has been stated [e.g. Schmertmann (79)] that other penetrometer tests should substitute for the SPT: it will probably be conceded, however, that in the general case any such other tests can only be taken as complements to, and not

*The bracketed numbers, e.g. (297) refer to the bibliography listed at the end, and are used for greater convenience.

as substitutes for, the combined technique employing perforation-rugged sampling-consistency evaluation.

Any truly generalized condition of preliminary subsoil reconnaissance must needs satisfy at least three most important conditions. Firstly, from the exploratory boring one must obtain an appropriate indication of the types of soils being perforated, stratum by stratum: thus, without an anticipated knowledge or estimate of what materials will be sampled (and, therefore, the ruggedness required), "representative" samples must be retrieved of sufficient quality to permit routine identification and classification testing; moreover, since any given soil type may occur under widely different degrees of consistency or compactness, ideally each sampling operation must concomitantly furnish some indication of such consistency or compactness of the respective soil element in situ; finally, it is indispensable to determine the ground water level or the pressure conditions in the ground water. Rugged sampling for deep foundations in residual soils is well emphasized by Philcox, 1962 (233).

In short, the routine auger and wash-drill exploratory perforation with intermittent "standard" drive sampling must be regarded as an inescapable first indication. As we shall see later, the fact that point penetration resistance at depth vary very intensely as a result of but minor changes of soil properties, would make the test capable of detection of the smallest variations, so much so that micropenetrometers have been most successfully used (e.g. 325) for checking the homogeneity of artificially deposited soil deposits for research. On the other hand, it will be understood that too sensitive a measure of small variations may be a hindrance rather than a help, when preliminary indications on average properties of "strata" are sought; in this respect the intense zig-zaging of the results of the static (Dutch, et al.) penetrometers is frequently recognized as confusing, if one lacks prior or collateral information as to how to subdivide into horizons apt for averaging. Therefore it appears that the ruggedness of the dynamic penetration, absorbing much of the finer local variations, may indeed constitute a trend in the appropriate direction for the first step of subsoil exploration.

As a start it must therefore be firmly established that tests such as the Standard Penetration Test belong as a first, overall indication [e.g. Ireland, and Hough, 1966, (79)]. Moreover it must be recognized that from the very first phase of the process of advancing design decisions, all the important parameters that will require consideration at subsequent stages must already be available for appraisal, explicitly or implicitly, in a level of precision compatible with the remaining parameters and with the respective phase of study and solution: therefore, at a very preliminary stage of foundation investigation it would be wrong to sacrifice the necessary all-round coverage, by seeking an improvement in one facet of the investigation, to the detriment of other facets. Obviously, in subsequent phases the more significant parameters are pursued for more accurate determination, depending on cost vs. value relationships, and it must be conceded that in some areas where preliminary indications may be available, explicit or implicitly, it may thus appear that a more appropriate and refined subsoil investigation has been substituted for the reconnaissance boring technique.

As a second thought it may be noted that in a first degree approximation, essentially all factors may be found roughly correlatable (for instance, 129), as may be reminiscent of the Biblical statement "He that is not with me is against me" (Luke 11 v. 23). Strength, incompressibility, imperviousness, etc., obviously tend to increase concomitantly with increases of "denseness" of soils, and so in general one may reason that certain properties vary together with, while others vary antithetically to, the apparent dry density (or some measure of the content of solids). Thus the indispensable point resides in establishing the confidence limits of such first approximation correlations: only by knowing the widths of variations encompassed by appropriate confidence limits (90% or 95%) can one discuss whether a given correlation has any applicability to engineering, or gains in such applicability by increased effort in numbers or refinements of tests.

Finally, as a third very important consideration it seems necessary to recall that the spoon-sampler dynamic penetration phenomenon is inevitably a very complex one, so that the corresponding lumped parameter must be a function of several of the simpler intrinsic parameters of soil behaviour. On the other hand, most of the phenomena of behaviour of foundations or of earth masses are also complex phenomena similarly embodying several simpler geotechnical parameters. If one faces separate complex lumped parameters of behaviour, $X = f\ (a, b, c, d, \ldots)$ and $Y = f'\ (a, b, c, d, \ldots)$, it may prove quite hopeless, in general to attempt direct correlations between Y and X, by random search of "statistical" correlations $Y = F\ (X)$ unless one begins by attempting to sort out analytically or pseudo-analytically the parameters $(a, c, f, \ldots)$ of more significant interference in X and in Y, and in each case attempting to establish at least approximate laws of the trends of the interconnecting functions. One might in this connection distinguish between "analytic" correlations, such as, for instance, correlations of SPT values with unconfined compression strengths q_u of clays, and, on the other hand, "direct synthetic correlations" such as would be the correlation between SPT values and "allowable bearing pressures for footings", or earth pressures on retaining walls (not yet attempted, but conceptually analogous).

It may be concluded, therefore, that on the one hand the subject of the Standard Penetration Test is all-embracing and therefore tantalizing, and, on the other hand it will inevitably prove frustrating because of the abyss that separates a preliminary overall indication from the gradually increasing specificity and refinement of solutions pertaining to subsequent stages of design decisions. Even if under certain circumstances we may be forced, as engineers, to continue resorting to the same crude test in later stages, either for lack of, or because of costs of, better techniques, obviously we can only count on some measure of narrowing of the confidence limits insofar as greater numbers of tests become available, and it be valid to apply to the engineering problems at hand the statistics of averages, as for instance in the case of settlements of large footings, wherein cumulative behaviour, and not the "trigger behaviour" under extreme conditions, is really involved.

The present report will therefore attempt to analyse the character of the several correlative applications attributed to the Standard Penetration Test, investigating as far as possible the confidence limits attachable to each.

2. SAMPLER PENETRATION IN CLAYS, AND FACTORS AFFECTING PENETRATION RESISTANCES

It is felt that one should begin by attempting to analyse the phenomenon of penetration of the spoon sampler in the case that may appear to have been accepted as least controversial. One of the basic preoccupations in the face of any test procedure is to establish to what extent it is reproducible, and therefore reliable as long as significant parameters are controlled: simultaneously one must investigate which are the significant parameters, and, if possible, what mathematical functions might express the laws governing the respective significant interferences in the observed results.

A hopefully complete reference survey reveals that without exception the spoon penetration resistance in homogeneous "purely cohesive" soils has been accepted as a simple function of the respective undrained shear strength. Thus one might begin by attempting a simplified analysis of the penetration phenomenon for such an ideal soil. Unfortunately there is no record (with the exception of Hvorslev 1949, 123, Figs. 111 - 114) of any systematic laboratory or field research into the static or dynamic phenomenon of penetration of sampling spoons, analogous to the meticulous research that characterized the introduction of the static cone penetrometers (4, 7, 10, 16, 18, 36, 37, 43, 60, 82, 84, 99, 139, 156, 234, etc.). Thus one of the approaches to the desired analysis comprises the rough comparisons of results as reported in publications in which some of the probable interfering factors have been altered, either consiously, in accord with intuitions, or unconsciously, because the basic ideas concerning spoon penetration resistances sprouted fairly universally while the Standard Penetration Test left many details unstandardized.

It may appear confusing to undertake comparisons of great numbers of different penetrometer procedures. Unfortunately, however, the conclusion from the reference survey is that the enormous confusion already exists. Moreover, since no publication has systematically analysed and/or taken into account the numerous parameters of first-order interference, often enough mentioned qualitatively, the only way of attempting to establish the probable physical and mathematical relationships at play comprises the recourse to such indirect comparisons, even if relating to penetration devices and procedures that have nothing to do with the would-be "Standard" Penetration Test.

The important interfering factors may be discussed summarily under the following groupings.

2.1 UNRELIABILITY DUE TO HUMAN FACTORS

The problem includes several obvious factors that have been repeatedly mentioned (e.g. Fletcher, 79, Mohr, 201, ...). Unfortunately there can be no solution to the problems, other than: (1) exercising due care, constantly, to avoid systematic errors; (2) relying on the bonafide statistical averages (where applicable) to exclude random errors. Conceptually the criticism (1) is not restrictive to any single test, and, ironically, if it has to occur to a greater degree in field tests because of the type of personnel in such activity as compared with laboratory or office, let us take cheer in the pity that field tests have to be done in the field; and, as regards (2), the principal advantage in SPT is that at practically no extra cost great numbers of tests may be performed, so that statistical averages may well be freed of random errors.

It may indeed appear strange that one should level selectively at SPT such suspicions as that based on "carelessness in counting the blows and measuring penetration" (Fletcher, 79) when any similar carelessness connected with the two main observations of a test would equally thwart any alternative test. It cannot be overlooked, however, that Mohr (79 disc.) "believes it wrong to infer or imply that the so-called SPT or any similar data are or can be made reliable and consistent". In diffident witness to the contrary (cf. Schnabel 79, disc.) it may be suggested that over the past 25 years thousands of skyscrapers and other important foundations in Brazil (and elsewhere, no doubt) have been designed and executed exclusively (in possibly 99% of the cases) on the basis of penetration resistance interpretations, such local interpretations have generally found published recommendations and codes from abroad comparatively conservative, and yet the apparent record of unsuccessful cases attributable to penetration resistances may be of the order of one or few per thousand. Perhaps as regards human factors a barely-literate labourer who faces a reconnaissance boring as a very important task in his humble life may be much more reliable than a labourer with misplaced broader vistas who faces his job as a nuisance of a means to other ends.

2.2 VARIATIONS OF DRIVING ENERGY AND PENETRATION ENERGY TRANSMITTED TO THE TIP.

The subject includes the efficiency connected with the applied energy e W H, the effect of changes of W and H, the problem of impact momentum equation and coefficient of restitution λ, and the effects of length, weight, and type of rods transmitting the impact energy to the penetration device.

2.2.1) Efficiency e. Regarding e there is no information. Qualitatively one is alerted against the problem of drag of the rope, and equipments that use wire line wound on drum. (e.g. Fletcher 79). There may also be a difference of friction losses in the use of the drop weight with the central hole as compared to the pin-guided weight depicted by Fig. 1 of Fletcher. Eccentric striking of the drive cap is also mentioned. (loc. cit.) On the one hand it would appear desirable and relatively easy to quantify statistically the average values of e and the respective dispersions, on the other hand the appropriate introduction of automatic devices (cf. Stubbings, 79 disc.) may greatly reduce the dispersions.

2.2.2) Errors in H. Other factors remaining constant obviously and error of ± 3 ins. in the height of drop results in a variation of ± 10% in the applied energy and thus one may conclude with Palmer and Stuart (1957, 228) that "the set is virtually inversely proportional to the height of drop, so that a variation in drop of an inch or two would affect SPT only slightly".

2.2.3) Changes of the impact-restitution conditions. By reference to dynamic pile formulae it should be obvious that the energy transmitted to the penetration device depends on the coefficient of restitution λ, among other important factors hereinafter discussed. Fletcher's (79) pin-guided weight possesses an important hard-wood cushion which Stubbings (79 disc.) claims "to the best of his knowledge is not used in the United Kingdom" and meanwhile Ireland (79 disc.) warns with regard to the drop weight with the center hole that "this involves a metal to metal striking contact and the possibility that the drive weight may chatter as it falls guided by the casing ... (which) is believed to have a major effect on the results of the SPT". Such comments appear obvious, the pity being that there is no reference to an attempt to quantify the order of magnitude of such "systematic error" effects.

2.2.4) Changes of W, energy WH, and length and type of rods. A number of references have from the very beginning attempted to consider separately some of these effects, thereby leading to the present situation wherein the entire problem is fraught with mistakes. Before analysing the individual references it is therefore believed worthwhile postulating the generalized theoretical reasoning for the problem, so that the differences may be interpreted.

From a similarity with the problem of pile-driving one may expect that the basic law governing the dynamic penetration of the sampler at the bottom of a set of rods will be that of work equilibrium, moreover, in close analogy to the important rejoinder that Terzaghi 1946, (296 a) makes with respect to piles, since the penetration involves a problem of failure of the soil around the tip, for a condition of constant energy transmitted, the phenomenon must also be a function of the maximum compressive stress transmitted to the tip by the wave.

In accordance with the equations of Newtonian impact between solid bodies we conclude that of the applied energy e W H, the energy transmitted to the penetrometer would be reduced to

$$(e\ WH)\ W.\ Wp\left(\frac{1+\lambda}{W+Wp}\right)^2 = (e\ W\ H)(1+\lambda)^2\ \frac{W.\ Wp}{[W+Wp]^2} \quad \text{.................(1)}$$

Where Wp = weight of the penetrometer plus the length of rods, and λ = coefficient of restitution.

Although the coefficient of restitution is generally assumed to be low [the value of 0.552 cited by Sikka (275) appears to be the highest admissible] it may be seen that the influence of differences of the coefficient may account for a ± 60% variation of the blowcounts.

λ	0 (perf. inelastic)	0.1	0.2	0.3	0.4	0.5	0.552
$(1+\lambda)^2$	1	1.21	1.44	1.69	1.96	2.25	2.4

Such an effect, doubtless very significant, must be considered when comparing results from different boring procedures. Within a given boring, however, the influence of the increasing length of rods, or of changes of rod type and weight, must be assessed, as regards the purely Newtonian impact of solid bodies, by analysis of the weight-ratio factor.

In order to permit rapid visualization of the importance of this interference the curves of Fig. 1 are presented for the weight-ratio energy function

$$R = \frac{W.\ Wp}{[W+Wp]^2} \quad \text{..............................(2)}$$

The appraisal of these indications may be based on the assumption that in a homogeneous soil the blowcounts should be, in a rough manner, inversely proportional to the Energy-function plotted. It may be seen that this factor introduces very significant variations with depth of boring; the blowcounts may easily double within typical depths of borings.

The additional factor that quite probably exerts an influence concerns the maximum compressive stress produced by the impact on the base of the rod-penetrometer assembly: within the range in which simplified equations may be employed (cf. Terzaghi, 1946) the corresponding variation of this maximum compressive stress is presented in Fig. 2, for appraisal of another factor that should be, in a rough manner, inversely proportional to the SPT blowcounts. Assuming all rods to be of steel of the same unit weight, a hypothetical variation of effective Young's modulus has been admitted in the range of (1 to 2) x 10^6 kg/cm^2 to account for differences in couplings: and, for lack of any information on

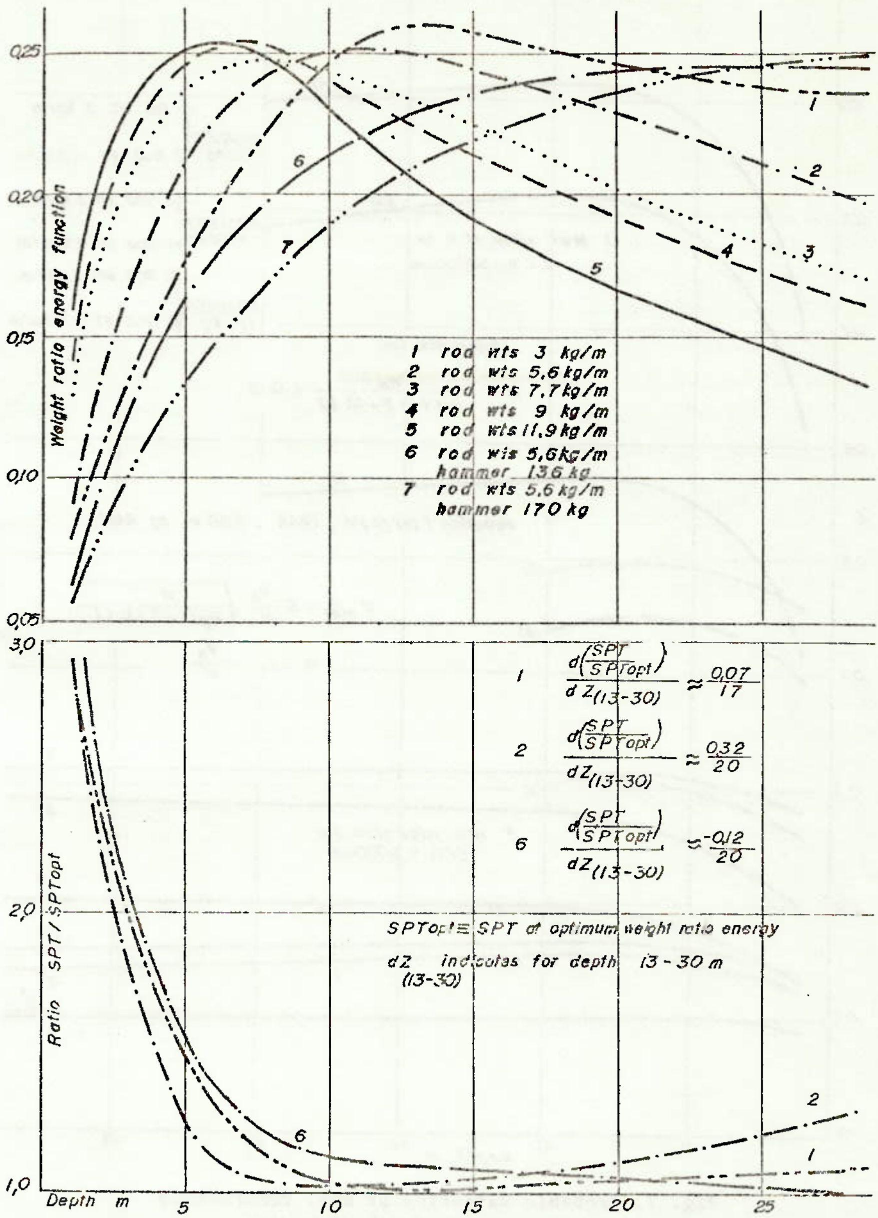

Fig. 1. Depth effects through weight-ratio energy function.

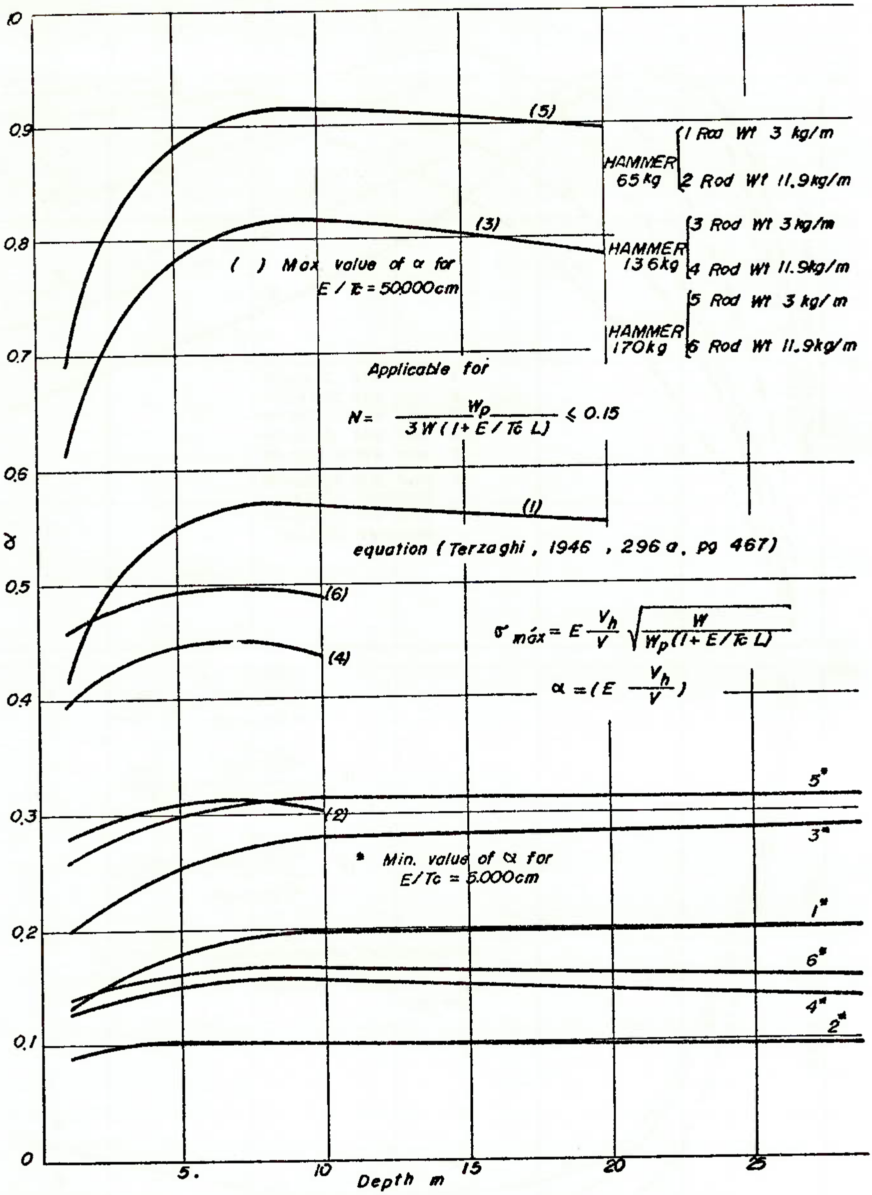

Fig. 2. Probable variation of max. compressive stress with depth of boring.

values of the stiffness constant Tc, a hypothetical range of variation has been assumed between 100 and 500 kg/cm^3. It may be seen that this factor could introduce a very major variation in comparing one case with another, but variations with depth within a given boring are smaller, particularly beyond the first few meters.

In the light of the above theoretical formulations it may be seen that the Burmister formula 1948, (40) for adjustments as regards variations of driving weight W and energy e W H is erroneous because of the oversimplification that would make penetration resistances proportional merely to W H. (N.B. The part of the Burmister correction intuitively assumed to prevail in direct proportion to the solid crosssectional area of the sampler will be discussed later). Sowers 1954, (283) furnished a tabulation for conversions of penetration resistances to values close to the SPT, but it is not indicated how these factors were established, and since they did not take any account of the weights of rods, onde concludes that the tabulation should be further questioned. [cf. Guerra 1963, (94), comparing the conversion factors with corrections as computed by the Burmister suggestion].

Sampler		Sampler areas sq. in.			Hammer Wt.	Drop	E	Conversion factor	
I.D"	O.D"	Ai	Ao	A	lbs.	in.	lbs. x in.	(1)	(2)
1	1.3	0.78	1.33	0.55	140	30	4.200	1.5	3.0
1.4	2.0	1.5	3.15	1.65	140	30	4.200	1.0	1.1
2.0	2.5	3.15	4.9	1.75	300	18	5.400	1.0	1.2
2.5	3.0	4.9	7.1	2.2	375	18	6.750	1.0	1.2

A = Asteel; E = Energy; (1) = Sowers; (2) = Guerra

Machado and Magalhães, 1955 (168) employ an oversimplified pile-driving formula to estimate the decrease of energy with increased depth of boring for rods weighing 3 kg/m: on the other hand Palmer and Stuart 1957, (228) claim that for rods of 11.9 kg/m, analysing by St. Venant-Boussinesq, the "weight of the driving rods in relation to the impact forces is very low, and even if the actual stresses transmitted to the soil are as little as one tenth the theoretical values, the dead weight only makes up a few (5) per cent of the total" and suggest that the SPT values "may not be influenced substantially when the length of rods is over 20 ft." The subject is further mentioned by Fletcher, 1965, and discusses Geisser and Renau (79) principally in connection with the limiting depth (around 30 m) beyond which the blowcounts would become "too high and unreliable".

Special attention must be focused on the U.S.B.R. research, Gibbs and Holtz 1957 (86), which has been widely quoted and used to justify the small attention paid to the problem of rod weights and lengths: moreover, special interest falls upon the data furnished by Golder, 1961 (89) to emphasize the fallacy in interpreting penetation resistance phenomena as directly related to the energy applied by the hammer. Both references were recently analysed by Sikka 1966 (275) in emphasizing the need to consider both the coefficient of restitution (apparently overlooked in the U.S.B.R. comparative tests) and the weight ratio: the possible interference of the effect depicted on Fig. 2 has not been considered.

In conclusion it appears that (1) pending further and more meticulous investigations of practical intervening parameters (e, λ, Tc, and "losses") the orders of magnitude of effects may be assessed by reference to the two basic equations mentioned (Figs. 1 and 2); (2) unless evidence arises to prove to the contrary, one must assume in all statistical comparisons, etc., that there should be a significant effect of length of rods in a given boring in homogeneous material; (3) special attention must be paid to the very rapid changes shown to occur, on both counts (Fig. 1 and Fig. 2), over the range of depths of 0 to 5 or 10 meters (depending on weight of rods) since a great deal of research [e.g. Gibbs and Holtz (86), Zolkov and Wiseman (353)] and the main interest of practical applications of SPT values has generally concerned shallow footings.

2.3 DIAMETER OF HOLE AND CLEANING OF HOLE

Most of the interferences mentioned in this connection appear qualitatively obvious. Usual description of the test is considered as associated with 2 1/2" or 4" casing holes [cf. Fletcher (79), and especially Parsons (79 disc.)] emphasizing exclusion of diameters bigger than 4". Improper cleaning of the hole may vitiate results by increased friction, and principally by the sludge at the bottom bearing against the upper ball check valve, increasing blowcounts (Fletcher 79), and by plugging the lateral vents with similar effects to a possible 50% increase in blowcounts as cited by Machado and Magalhães (168). One must exclude from discussion such a gross error as when "drillers sometimes fail to clean out the casing to the bottom" (Fletcher, 79).

The principal variation in this connection has been introduced by the general practice prevalent in the United Kingdom, employing 8, 10 and even 12" casing, alongside with perforation by "shelling" [cf. Rodin (244), Palmer and Stuart (288), Thomas (169) etc.]. The stress release "bulb" at the bottom of the hole will obviously be greater with such large-diameter holes, and consequent SPT values be smaller. But the principal factor may be the "unavoidable disturbance created by the shelling operation" due to the pumping action of the up-and-down travel of the shell. Thus, for instance, the observation that "in general, the Standard Penetration Tests in the borehole compare with the lower values of static cone resistance in the various horizons" (Rodin, 244) may be associated with this disturbance: moreover, as reminded by Thomas (169), drillers may well resort to loosening dense sands in order to facilitate lowering (and finally raising) such heavy, large-diameter casing, without too heavy a lateral friction. On the other hand, apparently the precautions exerted to avoid the loosening [cf. Rodin (244) using "smaller than usual shell", and Palmer and Stuart (228) cleaning out" with the lining tubes slightly advanced below the bottom of the boring, and the penetrometer lowered to the bottom"] may in instances lead to opposite effects by the driving of the penetrometer under confined conditions.

Gupta and Aggarwal (95), and Narahari et al. (213) present data quantifying the effects of diameter of hole (4, 8, 12", and open pits) to shallow depths: the effect is obviously much more noticeable (cf. 140, 142, 220, etc.) in dense than in loose soil, amounting to about 30% decrease in the bigger diameter boreholes, and about 50% in shifting from boreholes to open pits. With a slightly conservative simplification it is suggested (213) that for both static and dynamic cone penetrations a rectangular hyperbolic variation $q = K_1 / x$ and $N = K_2 / x$ is applicable, where x is the ratio of borehole to cone diameters.

2.4 USE OF DRILLING MUD VS. CASED HOLES.

Granger (91) considers the drilling mud responsible for increased SPT values, and Mohr (201) reminds that the wash-water mud may have similar effects depending on details of the holes at the top of the sampling spoon. However, the effect may be more complex than would appear at first sight. The use of drilling mud should be especially favourable for attenuation of the unbalanced head effect below water table, and consequent obvious "quicksand" loosening of material at the bottom. However, Schmertmann (79, disc.) reports equally unexplained scatter in a very fine sand above and below water table, and suggests that "perhaps the difficulty is caused by the dynamic nature of the SPT". Meanwhile Parsons (79, disc.) furnishes data whereby auger holes with aqua-jel gave roughly 50 blows while the casing boring had given around 20 blows, and proposes that "the auger boring resistances, in this case, were obviously more reliable": it may be noted that the preferences for acceptance of one or the other set of data may not be quite so obvious, if the only bases for them are from indirect comparisons with what be expected; if the sand was indeed a 50 blow material, it may appear strange that there is no increase of SPT with depths between the 80 and 145 ft. depths of the borings. Sutherland 1963 (289) mentions a case of "fine to medium sand to substantial depth" in which average SPT blowcounts in borings changed from 22, 24, 29 to the order of 64, 88, 94, presumably by elimination of boiling. It must be emphasized that rapid raising of the drill-rods can cause a marked suction effect and consequent boiling.

2.5 TYPE OF SOIL AND SOIL CONDITIONS.

Obviously there are soils and soil conditions in which the method of perforation must be specified. Machado and Magalhães (168) recommend that auger perforation be employed down to the water table in the fine clayey sands (residual from sandstones) and silty clays (residual from basalts) wherein the mere contact with wash-drill water drops the penetration resistances to about one-half. A similar effect is mentioned by Fletcher (79).

At the other extreme lies the case (Fletcher, 79) where in "gravel too large to pass the opening, added blows are needed to drive the spoon"; the consequent recommendation was "in such cases it is advisable to select the lowest number of blows recorded in the formation to evaluate its density". Such a recommendation does not appear acceptable, firstly because it fails to place the SPT in the conceptually correct position of any test available to the engineer, as a tool to be used with due specific interpretation; secondly because it would fail to admit employing one of the greatest advantages of the SPT in comparison with non-sampling penetrometers, which is the specific information of the soil being perforated (for instance, the fact that gravel did block the penetrometer would immediately be detected on subsequently resuming perforation); finally, because as stated it fails to consider point area effects, depth effects, etc.. It is a fact that "all penetration tests become unreliable as the maximum particle size approaches the diameter of the penetrometer or sampling spoon" [Meyerhof 1956, (190)], and when a test becomes unreliable because of use outside its intended range, there is really no way of getting by the unreliability.

In summary, one should develop special procedures, precautions, and recommended interpretations, possibly quite local, for the cases of special soils. Hopefully none of the persons who really work with penetration tests, and the SPT, nurture any hope akin to the philosopher's stone complex, of standardizing a minutely detailed testing and interpreting procedure usefully applicable to all soils and all conditions. Further mention will be made of the very important case of dense silty very fine sands below the water table for which from the start [Terzaghi and Peck, 1948 (300)] a special procedure of interpretation was suggested.

2.6 DISTURBANCE AND DEPTH OF "DISTURBED ZONE".

From the beginning [Terzaghi and Peck 1948, (300)] it was required that "after the spoon reaches the bottom, the drop weight is allowed to fall on the top of the drill rods until the sampler has penetrated about 6 in. into the soil, whereupon the penetration test is started ...". In comparison with earlier analogous techniques [cf. de Mello et al., (182)] this recommendation anticipated avoiding the recording of low penetration resistances due to the "pressure bulb" of disturbed soil.

Naturally, as diameters of boreholes have greatly varied and perforation techniques have greatly altered disturbance effects, a significant number of variations on the theme have been introduced. One group includes the alteration of procedure: ex. Schnabel (79 disc.) mentions "seating on the bottom of the hole with a few light taps", but since presumably the reaching of the bottom may be established on the basis of forcing to a given depth, and since in a wash-drilling technique the bottom may accumulate a concentration of the coarser grainsizes, it is of interest to caution, with Fletcher (79) that erroneously high blowcounts may be caused by "excessive driving of the sample spoon before the blowcount". Lo Pinto (79 disc.) mentions that "the spoon and drill rods ... the last 10 ft. increment, at least, is allowed to drop to reach bottom". In Brazil (cf. de Mello et al, 182, etc.) because of the tradition connected with blowcounts for the first 12 in. of penetration for both the Mohr-Geotecnica sampler (IRP blowcount index) and for the I.P.T. sampler (IPT blowcount index), it has been customary to seat the weight on the rods to force an initial "static" penetration, and then to begin the blowcounts which are recorded for three consecutive penetrations of 6 in.: for the SPT sampler the result reported is the sum of the blowcounts for the last two 6 ins.

Another group has proceeded to increase the length of sampler to 30 in. [e.g. Palmer and Stuart (228)], and to record the blowcount for the second foot of penetration. Depending on the parameters at play, significant differences may be introduced in any of the above cases.

A third group has undertaken to investigate the depth of the disturbed zone by means of tests in which the number of blows were recorded at every few inches [e.g. 3", Granger (91)]. The respective results merit careful examination, since they are conceptually in disagreement with the continuity of the penetration phenomenon, which is used hereinafter for pseudo-analytical and statistical derivation concerning the penetration phenomenon. Both Granger (91) and Gupta and Aggarwal (95) interpret the penetration as comprising two distinct straight lines, with point of intersection or inflexion, so that "in any case, the SPT value should be evaluated from the phase of penetration vs. number of blows curve beyond the point of inflexion" (95). The discontinuity is recorded by Gupta and Aggarwal at 10-14" (average

12" penetration) using different perforation techniques and diameters of holes, but is encountered at greatly varying penetrations by Granger (91).

Theoretical reasoning, however, would lead one to expect, in a given soil, no discontinuities, and no linear relationships, even for the "final" phase of penetration. And, in discussing the topic one must distinguish between a) discontinuity due to bottom disturbance; b) discontinuity due to soil change, and the attempt to employ and refine such detection; c) final linearity as employed for extrapolation (conservative) of blowcounts in the case of very dense materials.

In examining thousands of boring results subjected to statistical analyses (de Mello et al., 181, 182) the disturbance discontinuity has rarely been found: perhaps a part of the difference of opinions results from different degrees of precision of observations, it being more recommended for such studies to record the number of cms. or ins. penetrated under each consecutive blow, rather than the number of blows required for predetermined (3" or 6") penetrations. Practical evidence to the continuity and non-linearity of the penetration phenomenon in a non-discontinuous soil may be observed in Narahari et al (213), as well as in all the pile or penetrometer penetration curves reaching depths of the order of 10 times the diameter.

As regards discontinuity due to soil changes within a penetration test, de Mello et al. (191) suggested simple statistical quality control for rejection of such results. Moreover, inspection of samples retrieved must not be overlooked. It appears quite beyond the degree of precision of the SPT to attempt investigating differences of subsoil to the scale of 6 ins. [Fletcher, Lo Pinto, Schnabel, Geisser, 79)] or further down to 3 ins. [possibly implicit in Granger (91)]. Also, if the penetration phenomenon is continuous, accumulating gradually greater number of blows per unit length of penetration, as is herein contended and will be hereinafter discussed, obviously one cannot accept (cf. Schnabel, 79) the use of would-be "conservative" interpretative procedures of considering the sum of the 6" penetrations giving the lowest total (Fletcher, 79), or the like.

Finally, as regards the assumed linearity of the penetration phenomenon within the homogeneous virgin soil mass, once again a significant difference of observations and interpretations occurs between much of the published information (79, 91, 213, 228) and the experience connected with the analyses above mentioned (181, 182). Granger (91) states "The number of blows for each inch of penetration in uniform soil is constant. Under these conditions the number of blows for 12 inches can be extrapolated from any intermediate value"; Lo Pinto (79) presumably expects the last two 6" blowcounts to differ only "in those rare instances where the spoon penetrates two different soil strata"; Palmer and Stuart (228) state "model experiments suggest that the side friction effect is probably very small, and show that even when the penetration is doubled in uniform material the value of N for the second foot of penetration is only some 2% greater than for the first foot"; Gupta and Aggarwal (95) indicate the two straight lines; Sikka (275) indirectly implies "final" linearity.

Such differences point to the importance of an experimental and pseudo-analytic investigation of the sampler penetration phenomenon.

2.7 INTUITIONS ON THE PENETRATION PHENOMENON. USE OF COMPARISONS OF DIFFERENT SAMPLERS.

From the start it was assumed that in clays the SPT would be directly related to the undrained shear strength, and all publications have reported empirical correlations SPT = (n) q_u. The basic tabulation for classification of consistency of clays as suggested by Terzaghi and Peck 1948 (300), implies a correlation SPT = 8 q_u (kg/cm^2).

Burmister's, 1948 (40), much cited equation for conversion of penetration resistances of different samplers to a common basis implies purely a point resistance phenomenon proportional to the sampler's annular area of steel.

In the case of sands on the one hand Terzaghi, 1953 (299) states: "This number SPT depends chiefly on the friction along the walls of the tube. If the sand is dense the number SPT of blows increases with increasing depth below the surface, whereas in very loose sand SPT is practically independent of depth". Simultaneously, Peck at al, 1953 (230) implicitly admit, for use in connection with shallow footings, that the SPT values reflect the sand's ϕ values. Meyerhof, 1956 (190) establishes a linear correlation of SPT with static penetration tests, thereby connecting the blowcounts to point resistances .... Rp (kg/cm^2) = 4 SPT and thus to ϕ values.

It can be seen, therefore, that in general the blowcounts have been intuitively related to a shear resistance problem, with some confusion as to the influences of point resistance and side friction contributions.

However, no systematic investigation has been conducted to confirm or revise such a first approximation intuition. Moreover, all attempts at correlations have been strictly empirical, and with few exceptions (e.g. 168, 181, 182) none of them statistical. Since the separate contributions of point and skin friction effects doubtless vary from sampler to sampler, some interest will fall on the statistical correlations published regarding different samplers or penetrometers. Meyerhof, 1956 (190) for instance, states that "The dynamic cone resistance (using a 60^o cone of 2 in. base diameter on an uncased 1 5/8 in. diameter sounding rod driven under 350 ft. lb energy) is about twice the standard penetration resistance under the same energy, which may at least partly be explained by the area of the cone base being about twice the net sectional area of the sampling spoon": meanwhile Schultze and Knausenberger, 1957 (261) mention a ratio of only 1.25 for the 2" cone and 2" rods, and a careful examination of the results presented by Mohan et al, 1970 (199) show that for a 62.5 mm cone and A rods the ratio of the SPT varies from about 1.4 to about 1.8, continuously, for depths between 1.5 and 7.2 m. Such are the correlations and intuitive indications that are available for interpretative study. No attempt will be made herein to summarize and discuss the great numbers of correlations published (5, 63, 65, 89, 91, 94, 95, 118, 131, 137, 168, 169, 181, 182, 183, 184, 190, 228, 239, 250, 261, 283, 289, 301, 337). After establishing a pseudo-theoretical background to the problem, those that have a bearing on the case will be discussed.

2.8 ATTEMPT AT ESTABLISHING A SAMPLER DYNAMIC PENETRATION RESISTANCE EQUATION FOR A HOMOGENEOUS, INSENSITIVE SATURATED CLAY.

In previous studies (de Mello, 183, 184) it was postulated that the principal reason for the very wide range of single-parameter correlations of the type SPT = n q_u in saturated clays arises from neglecting, as a minimum second important parameter, the effect of partial remoulding in sensitive clays: therefore it would be desirable to begin by investigating an insensitive clay. Moreover, for a long time it had been noted [de Mello et al., 1959, (182)] that "one important factor ... is the length of sample that enters the sampler", the penetration phenomenon being thus characterized by at least two distinct phases, the first being of penetration of the annular volume of the sampler-wall, and "the remainder of the penetration probably being achieved by displacement of the soil as if a solid bar were driven". Nonetheless, it was proved statistically to satisfaction, within 95% confidence limits, "that there is no real discontinuity in the penetration behaviour" of the samplers through the 45 cm of driving (loc. cit.). Finally, it had been proven since the earliest studies (181, 182) that there are statistically significant interferences of several factors (such as depth, for instance) in the principal correlations, and that the best statistical regressions for the SPT either do not really pass through the origin if linear, or would be quadratic.

Thereupon, it was felt since the last studies in this connection [de Mello 1967, (183)], when penetration resistances were interpreted as related to bearing capacity problems, that the desired correlations might be more appropriately investigated through judicious use of energy equations similar to those of pile driving, at least with a view to establishing the reasonable trends for statistical regressions. Upon recent reperusal of earlier publications, it may be seen that the idea is not new; for instance, Meyerhof 1956 (190) mentions that "dynamic tests are usually extrapolated by means of a pile driving formula", and Malcev 1964 (169) in quoting a highly simplified formula attributed to Haefeli probably resorts to the same reference. Dobry 1957 (65) adopts analogous concepts analyse results of the dynamic cone penetration tests.

Finally, since in any such theoretical deduction there are inevitably a great many unknown parameters, it was thought necessary to retain the concomitant use of different samplers in order to obtain the minimum necessary number of simultaneous equations.

At two sites comprising a moderately thick layer of low-sensitivity preconsolidated São Paulo clays, clusters of borings and static penetration tests were conducted in 1967 by courtesy of Geotecnica S.A. under the author's direction. In one case at distances of 2 m center-to-center Boring A employed the SPT procedure (rods weighing 3 kg/m), Boring B extracted 2" diameter shelby samples, and Boring C employed the Mohr-Geotecnica sampler of D_o = 1 5/8", D_i = 1" driven by the same weight and fall, through the same 45 cm of penetration. Moreover, the samples of virgin soil retrieved within the spoons were measured, and insofar as possible subjected to unconfined compression tests. The logs of these borings are summarized in Fig. 3. In the other case within center-to-center distances of 0.8 to 1.3 m, Boring 1 was a static cone penetration test; Boring 2 extracted 2" diameter shelby samples; Borings 3 and 5 determined SPT values and obtained unconfined compression tests on the samples retrieved; Boring 4 employed the Mohr-

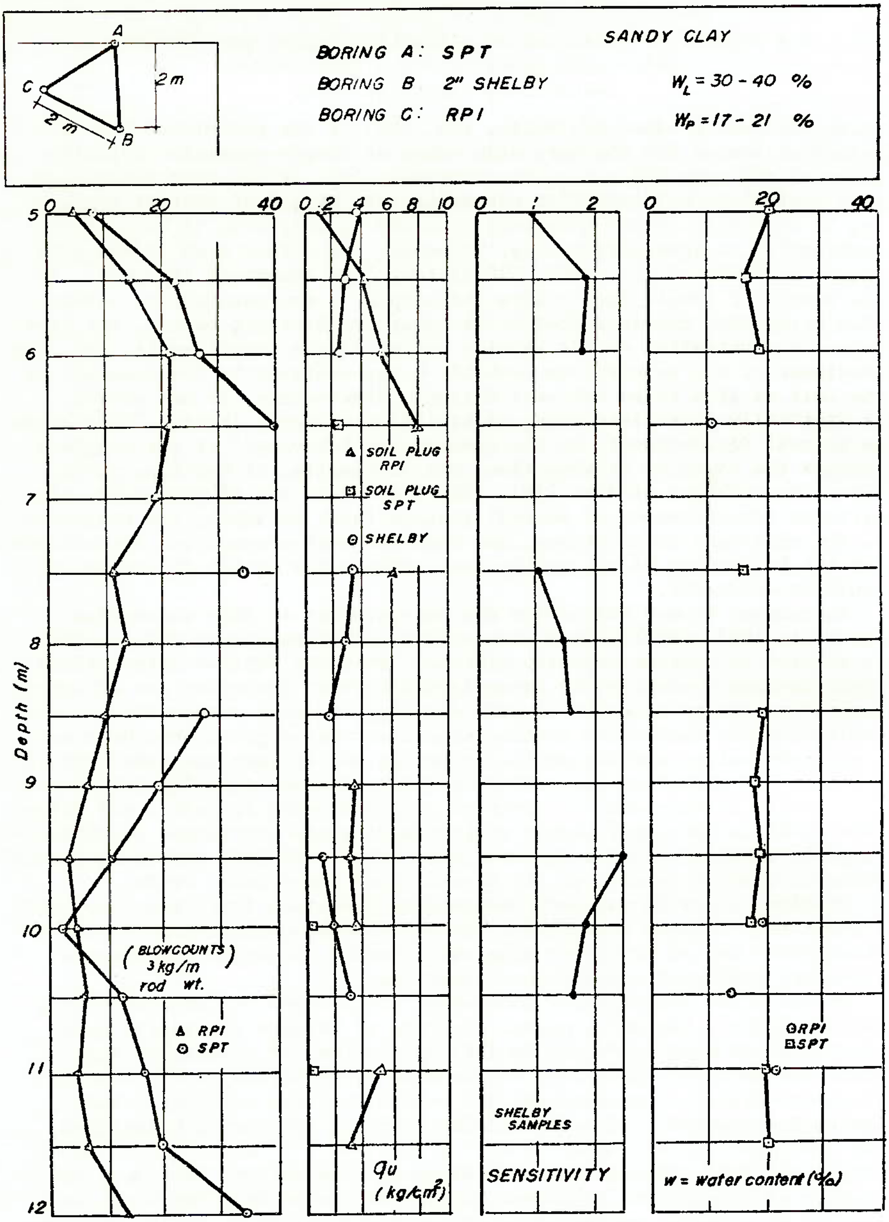

Fig. 3. Subsoil data from Site 1, S. Paulo clay.

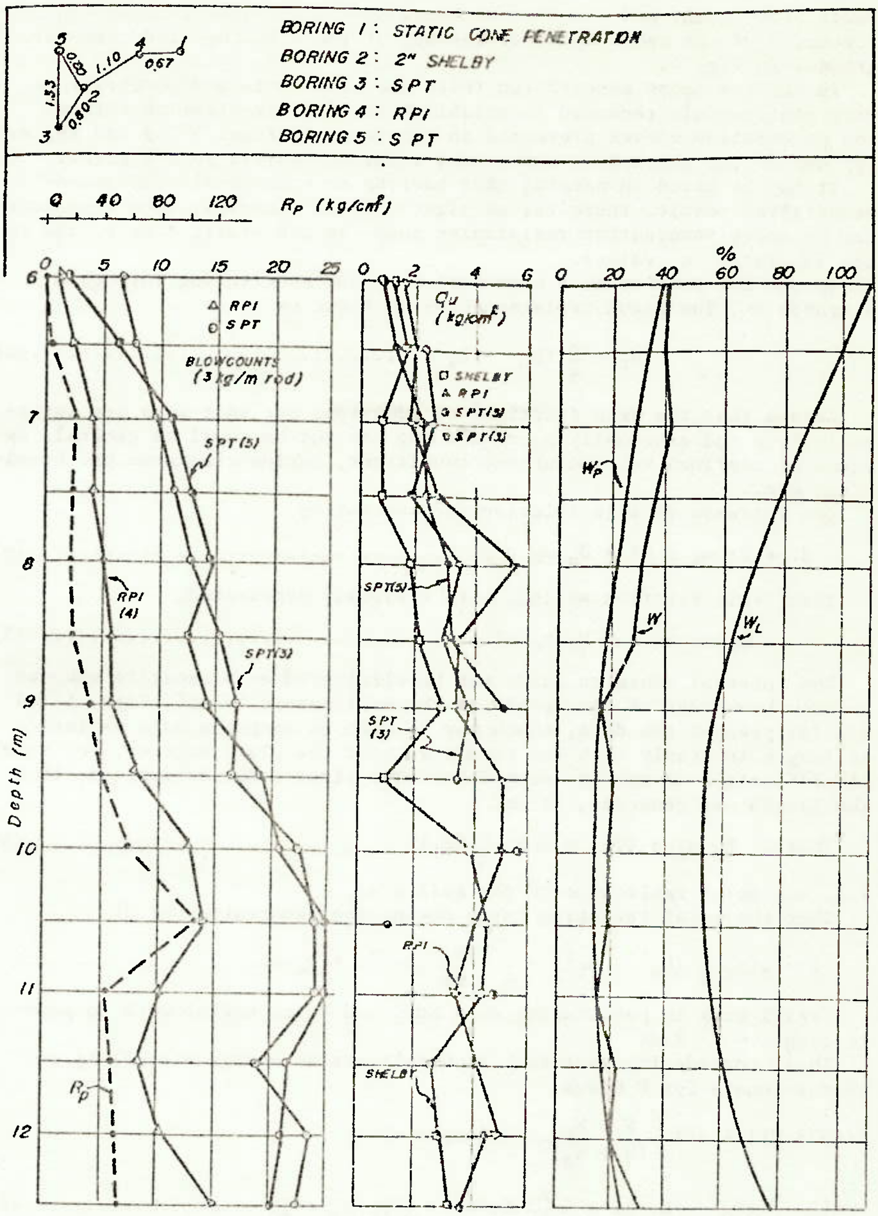

Fig. 4. Subsoil data from Site 2, S. Paulo clay.

Geotecnica spoon, and likewise obtained some unconfined compression strengths of the spoon samples. The log of these borings and tests are grouped in Fig. 4.

In all the spoon penetration tests the blowcounts and penetrations were continuously recorded to establish the full penetration curves. The penetration curves presented in two forms in Figs. 5 (a) and (b) are typical of the absolutely consistent results observed in all cases.

It may be noted in passing that barring an occasional wild spoon-penetration result, there is, as might well be expected, much less scatter in these penetration resistances than in the static cone R_p and in the laboratory q_u values.

Assume the sampler at a penetration ℓ, and penetrating through a distance $d\ell$. The point resistance may be taken as

$$N_c\ c\ \frac{\pi}{4}\ (D_o^2 - D_i^2) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Assume that the skin frictions or adhesions per unit area are internally m_i c and externally m_o c: the two may not be equal in general, because of confined vs. unconfined conditions, inside clearance vs. bevelling, etc..

The increase of skin friction in penetrating

$$d\iota = D_i\ m_i\ c\ d\ell + D_o\ m_o\ c\ d\ell \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Total skin friction acting, with ℓ already penetrated,

$$= \quad D_i\ m_i\ c\ell \quad + D_o\ m_o\ c\ell \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

The internal adhesion force may be eliminated as a separate unknown through knowledge of the length of the soil sample "plug". Figs. 5 (c) and (d) present the data, which may be seen to comprise some variation of length inversely with the consistency of the clay: however, for some simplification it may be assumed for this first-order derivation, that the length was constant, 25 cm.

$$\text{Thus} \quad D_i\ m_i\ c\ (25) = c\ N_c\ \frac{\pi}{4}\ (D_i)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6),$$

i.e. the point resistance of the soil plug.

Thus the total resisting force during the penetration of $d\ell$

$$R = c[N_c\ \frac{\pi}{4}\ (D_o^2 - D_i^2) + \frac{\pi}{4} \cdot \frac{N_c}{25}\ (D_i)^2\ell + D_o\ m_o\ell] \quad \ldots\ldots\ldots\ldots (7)$$

Useful work in penetrating $d\ell = Rd\ell$, and total useful work in penetrating $\ell = \int_0^{\ell} R.d\ell$

Thus, considering that each blow delivers an energy e(4875) kg.cm, we may equate for N blows

$$e(4875)N(1 + \lambda^2)\left[\frac{W\ .\ W_p}{(W + W_p)^2}\right] - \text{losses} =$$

$$= c[N_c\ \frac{\pi}{4}\ (D_o^2 - D_i^2)\ell + \frac{\pi}{4}\frac{N_c}{50}\ D_i^2\ell^2 + \frac{\pi}{2}\ D_o\ m_o\ \ell^2]_o^{\ell} \quad \ldots\ldots\ldots\ldots (8)$$

The integration constant is annulled by the fact that the penetration energy is zero for $\ell = 0$.

The losses will be assumed as directly proportional to N only, and

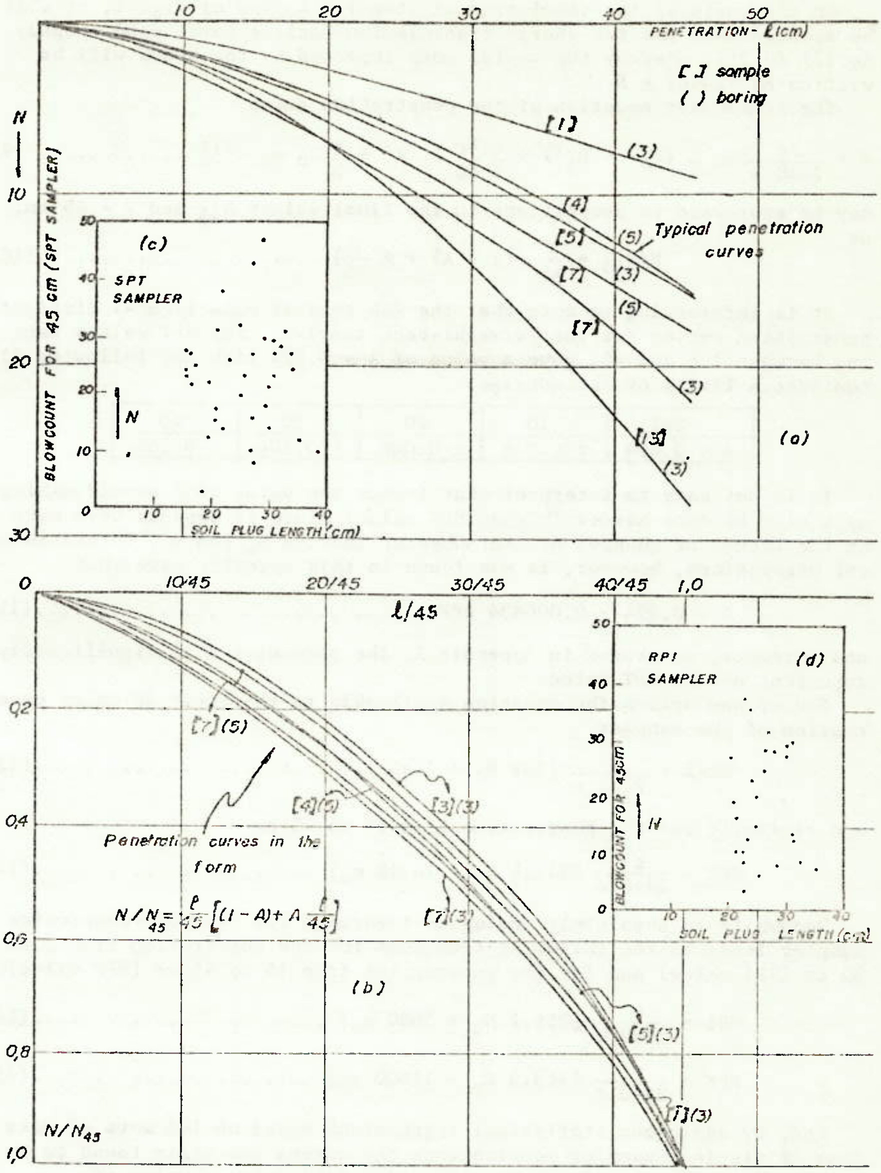

Fig. 5. Continuity of penetrations in clay.

therefore may be suppressed from the equation because they would merely alter the value of e.

On the basis of the tabulation of item 2.2.4, and of Fig. 1, it will be assumed that the two energy transmission factors correspond roughly to (2) (0.25), whereby the useful work imparted by the blows will be written as (2440) e N.

The consequent equation of the penetration curve

$$N = \frac{\ell c}{2440\ e}\left[N_c \frac{\pi}{4} (D_o^2 - D_i^2) + \frac{\pi}{4} \frac{N_c}{50} D_i^2 \ell^2 + \frac{\pi}{2} D_o\ m_o\ \ell^2\right]_o^\ell \quad \ldots\ldots\ldots\ldots (9)$$

may be expressed in proportions to the final values N_{45} and ℓ = 45 cm, as

$$N/N_{45} = \frac{\ell}{45}\left[(1 - A) + A \frac{\ell}{45}\right] \quad \ldots\ldots\ldots\ldots (10)$$

It is interesting to note that the 246 sets of data from 41 distinct penetration curves for the Terzaghi-Peck sampler, for SPT values ranging between 1.6 and 40, gave a value of A = 0.694 with the following 95% confidence limits of the average:

SPT	10	20	30	40
A = 0.694	± 0.078	± 0.006	± 0.104	± 0.166

It is not easy to interpret what trends the value of A should exhibit as a clay becomes harder (higher SPT value) since it depends very much on the trends of changes of the adhesion indices m_i and m_o. By statistical regressions, however, it was found in this specific case that

$$A = 0.694 - 0.000434\ SPT \quad \ldots\ldots\ldots\ldots (11)$$

and moreover, as proven in Appendix I, the parameter A is significantly dependent on the SPT value.

Now if one writes the equation applicable to the first 30 cm of penetration of the sampler

$$SPTI = \frac{c}{2440\ e}\left[332\ N_c + 7155\ m_o\right] \quad \ldots\ldots\ldots\ldots (12)$$

and similarly for the penetration from 15 to 45 cm

$$SPT = \frac{c}{2440\ e}\left[345.5\ N_c + 14310\ m_o\right] \quad \ldots\ldots\ldots\ldots (13)$$

Meanwhile an absolutely analogous treatment for the Mohr-Geotecnica sampler leads to the following equations for the penetration from 0 to 30 cm (RPI value) and for the penetration from 15 to 45 cm (RPF value).

$$RPI = \frac{c}{2440\ e}\left[256.2\ N_c + 5800\ m_o\right] \quad \ldots\ldots\ldots\ldots (14)$$

$$RPF = \frac{c}{2440\ e}\left[265.3\ N_c + 11600\ m_o\right] \quad \ldots\ldots\ldots\ldots (15)$$

And, by analogous statistical regressions based on 145 sets of data from 29 distinct sampler penetrations the curves are again found to be expressed in the same form as in equation (11), with a value of

$$A = 0.600 - 0.0024\ RPF \quad \ldots\ldots\ldots\ldots (16)$$

again a dependent function of the consistency of the clay as indicated by the RPF value.

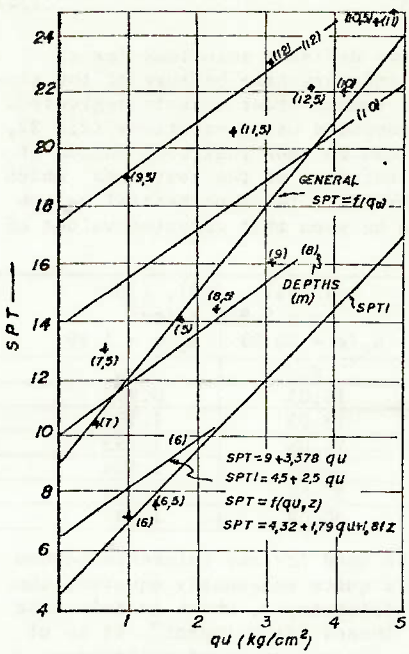

Fig. 6. Statistical analyses of data from Figs. 3 and 4, on SPT sampler, in S. Paulo clay.

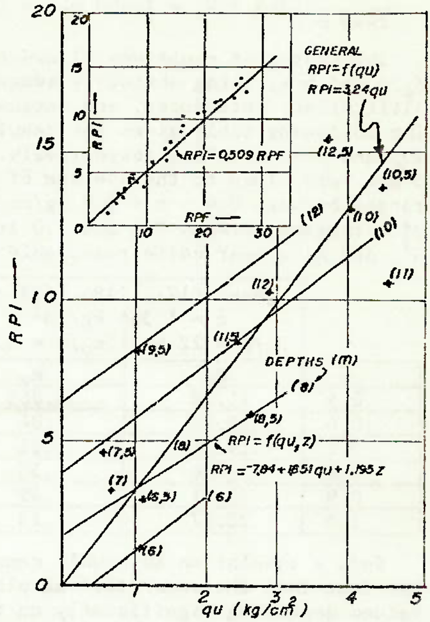

Fig. 7. Statistical analyses of data from Figs. 3 and 4, on RPI sampler, in S. Paulo clay.

From the available data, as plotted on Figs. 6 and 7, it can be seen that the following statistical regressions are available, comparing the penetration resistances to the c values as determined from q_u tests on the 2" shelby samples.

$$\text{SPTI} = 4.50 + 5.01\ c \qquad (17)$$

$$\text{SPT} = 9.00 + 6.76\ c \qquad (18)$$

$$\text{RPI} = 6.48\ c \qquad (19)$$

$$\text{RPF} = 1.97\ \text{RPI} = 12.75\ c \qquad (20)$$

The equations interconnecting the empirical results with the pseudo-theoretical indications would therefore be

$$\frac{c}{2440\ e}\,[332\ N_c + 7155\ m_o] = 4.50 + 5.01\ c \qquad (21)$$

$$\frac{c}{2440\ e}\,[345.5\ N_c + 14310\ m_o] = 9.00 + 6.76\ c \qquad (22)$$

$$\frac{c}{2440\ e}\,[256.2\ N_c + 5800\ m_o] = 6.48\ c \qquad (23)$$

$$\frac{c}{2440\ e}\ [265.3\ N_c + 11600\ m_o] = 12.75\ c \quad \ldots\ldots\ldots\ldots\ldots\ldots (24)$$

The system of equations allows for two definite solutions for c, N_c/e and m/e, being obviously somewhat indeterminate because of the simplifications introduced, and because of second-order effects neglected. The following table gives the results computed using equations (21, 22, 23) and (21, 22, 24), respectively. It can be seen that both values of c are very close to the average of the universe of the test data, which ranged between $0.4 < c < 2.8\ kg/cm^2$. Moreover, for hypothetical values of e ranging between 0.5 and 1.0 it can be seen that computed values of N'_c and m_o appear quite reasonable.

	Eqs. (17), (18), (19): $c = 1.366\ kg/cm^2$		Eqs. (17), (18), (20): $c = 0.960\ kg/cm^2$	
	$Nc/e = 22.57$	$m_o/e = 1.73$	$N_c/e = 30.09$	$m_o/e = 1.99$
e	N_c	m_o	N_c	m_o
0.5	11.28	0.865	15.05	0.995
0.6	13.54	1.04	18.05	1.194
0.7	15.76	1.21	21.06	1.393
0.8	18.06	1.38	24.07	1.504
0.9	20.31	1.49	27.08	1.791
1.0	22.57	1.73	30.09	1.99

Such a tabulation obviously cannot be used for any inference beyond the fact that the parameters at play are quite reasonably equated, the values depending significantly on the efficiency e, which in this case has been made to incorporate also the "losses after impact". It is of interest, however, to note the separate contributions of resistances that obtain in different samplers, since in order to improve future determination of different unknown parameters by simultaneous regression equations, it would be desirable to develop samplers that did accentuate the differences of the contributions. Thus, presently with the Terzaghi-Peck sampler (in the clay investigated) the point resistance contribution decreases from about 40% in the initial 30 cm of penetration to about 30% in the final 30 cm; meanwhile in the Mohr-Geotecnica sampler the drop is from about 39% to about 25%.

It may be noted that apparently the losses are somewhat greater in the final blows, since the values of N_c and m_o obtained for $e \approx 0.8$ using the equation (19) for the first 30 cm of penetration (RPI) tally with those obtained for $e \approx 0.6$ using the analogous equation (20) for final 30 cm (RPF). The somewhat surprising results of m_o values predominantly higher than unity may yet concur with the data, possibly, reflecting appreciable compressions of the clay, since the samples retrieved from the spoons yielded, in this particular case, roughly 30% to 60% higher q_u values than were obtained with the 2" shelby samples.

The most important effect, however, that was neglected in the above regressions and deductions, was the effect of depth. To begin with one may note that the U.S.B.R. Report, 1960 (312) claims "the effect of rod weight and length upon penetration resistance is not fully understood and it has not been definitely proven in this study that penetration resistance is a function of shear strength alone, although this study shows interesting and useful trends". At any rate, the effect of depth had to be neglected in these preliminary analyses because of the unwieldyness of the number of equations and unknowns in the face of insufficient sets of data for the statistical regressions; it may be noted, however, that it had already been recognized and pointed out by statistical regressions [cf. de Mello et al. 1959, (182)]. In order to evaluate the order of magnitude of the depth effect, which is quite apparent in the plots of Figs. 6 and 7, the statistical regressions were repeated to include a linear variation of SPT and RPI not merely in function of q_u, but also in function of z. Both regressions indicate an appreciable variation in function of z, and as summarized in Appendix I for the case of Fig. 6, the dependence on z is statistically highly significant (N.B. For the case of Fig. 7, on RPI, because of the smaller number of points and comprehensible greater scatter, the similar analysis furnishes a factor but slightly higher than the value that proves the significance of the dependence).

Since it appears that the effect of z on SPT or RPI values should be principally a measure of the energy factors discussed under item 2.2, and since both the samplers were driven under essentially identical conditions of hammer and rods, it may be expected that the rates of change of SPT and RPI values with z should be in roughly the same proportion as the proportions of SPT/RPI values: by the regression equations these rates of change are about 1.81 and 1.195, with a ratio of 1.51 from the pseudo-analytical equations and tabulated values of N_c and m_o, it is concluded that the average ratio of SPT/RPI = 2.0, which checks with the value published by Machado and Magalhães, 1953 (168). The somewhat smaller ratio deduced may be justified as compatible with the fact that the "losses" are not merely proportional to the blowcounts, as assumed, since "refusal" will be met at essentially the same hardness of clay which would correspond to smaller RPI blowcounts than SPT ones.

In short, therefore, without any intention of lending undue importance to the pseudo-analytical treatment herewith suggested as a bare minimum, it is postulated that the dynamic sampler penetration phenomenon may be analysed to advantage, in search of indications of the more significant parameters and trends.

2.9 SPT AND UNDRAINED SHEAR STRENGTH OF CLAYS.

Some important conclusions may be summarized and compared with published information, in which the undrained shear strengths of clays have been defined by:

a) unconfined compression strengths q_u of "undisturbed" samples;
b) static cone penetration values R_p;
c) in situ vane shear tests.

2.9.1. It appears to be convincingly established that in saturated insensitive clays SPT values are a direct measure of the undrained shear strength, and may in first order approximations be connected theoretically through conservation of energy principles analogous to those of judiciously employed dynamic pile formulae.

2.9.2. The effects of all the impact energy transmission parameters may be more significant than has been presumed, and should be duly considered in comparisons between distinct procedures and equipments, and in analyses of depth effects in a given boring. For most cases the variations should apparently be especially abrupt at very shallow depths. Moreover, since in many cases exemplified in Figs. 1 and 2 the probable factors of interference are not simple functions of z, depending on the range of depths considered the effects of depth may be undetectable by simple statistical correlations.

2.9.3. The SPT value reflects significant participation both of skin friction and of point resistance: these components obviously vary from sampler to sampler and may be varied consciously (e.g. by altering area ratios, diameters, inside clearances etc..) so that one may well develop a) more suitable samplers and driving procedures for such special cases as soft clays wherein the SPT blow constitutes too rough a measuring unit; b) suitable pairs or clusters of samplers so as to furnish more appropriately the multiple simultaneous equations for deduction of the many first-order unknowns.

2.9.4. As emphasized earlier [de Mello 1969, (184)] the sensitivity of clays should have a significant influence on the correlation of SPT = f(q_u), so that it is absolutely unacceptable to employ a single indication of tabulated SPT values as a basis for classification of clays as very soft, soft, medium, stiff, very stiff, and hard (300). The correlations should be separately established for representative clay strata (regionally); such correlations might be linearized, but not passing through the origin. Moreover the interfering effect of depth should be considered or duly minimized.

2.9.5. In Fig. 8 are presented the correlations hitherto published, earlier collected in Fig. 27 of de Mello, 1969 (184), and herein updated. Moreover, for the sake of completeness Fig. 9 reproduces herein the indications submitted earlier [de Mello, 1967 (183), 1969 (184)] on the influence of the clay sensitivity. It may be noted that for the

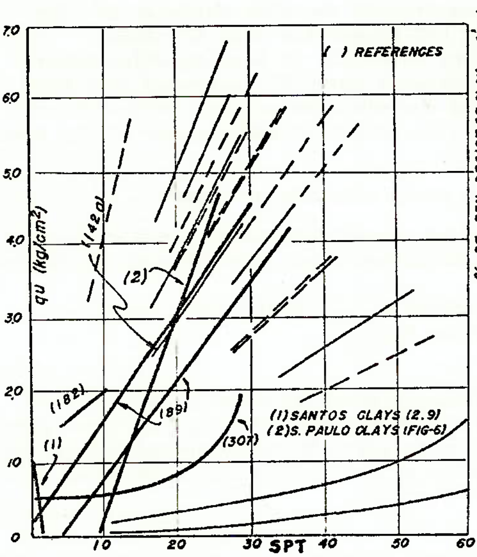

Fig. 8 Correlations q_u(kg/cm^2 = f(SPT) for clays

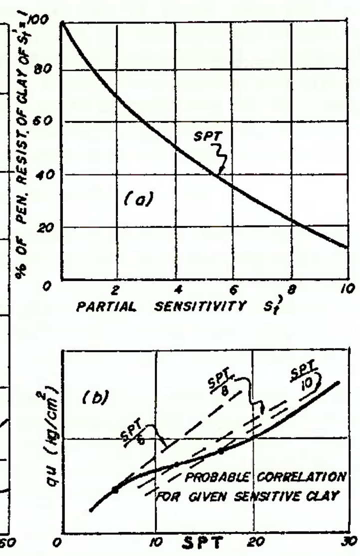

Fig. 9 Probable effect of sensitivity of clays on q_u = f (SPT)

[Apud de Mello, 1969 (184)]

average q_u value of about 1.5 kg/cm^2 the S. Paulo insensitive clays above discussed would give roughly SPT = 14 whereas for the sensitive Santos clays (LL = 120%, Sensitivity of the order of 10, Partial Sensitivity estimated around 5) the corresponding blowcount for q_u = 1.5 kg/cm^2 is SPT = 1. The sensitivity correction is therefore probably even greater than shown on Fig. 9: the data available for the earlier analyses were but rough single-parameter correlations.

2.9.6. With respect to the depth effect it must be understood that since the functions represented on Figs. 1 and 2 are not simple functions of z, first-order attempts at statistical evaluation of the interference of z are likely to result confusing. It is highly recommended that for the sake of proving, disproving, or revising, this very important question, all data in the future furnish every detail necessary to evaluate the functions suggested under item 2.2.4, so that statistical regressions on z may be used on such functions as a start. To illustrate the point two extensive sets of data were analysed. The U.S.B.R. report, 1960 (312) yields statistical regressions SPT = 1.7 + 0.848 c - 0.0473 z (c in psi and z in feet) for the data of high degree of reliability and SPT = 0.36 + 0.664 c - 0.0551 z for the data of high and medium reliabilities: but if one analyses the significance of the dependence on z, the result is negative; it may be noted that the data are for depths between 5 and 48 ft. (1.5 to 15 m) which, depending on the rod weights, straddle the hump of Fig. 1. On the other hand, for the highly plastic normally consolidated Santos clays, for depths varying between about 10 and 25 m using the 3 kg/m rods, the statistical regression of SPT = 1.955 - 0.352 q_u, kg/cm^2, on 68 pairs of values, obviously proves unacceptable because of the crudeness of the test for $0.1 < q_u < 1.2$ kg/cm^2 and 0.6 < SPT < 4. A concomitant regression with respect to in situ vane shear strengths proves somewhat better, SPT = 0.87 + 1.464 c (on 92 pairs of values). Both regressions indicate a very marked improvement upon introduction of the simplest (linear) function of z (m), SPT = - 16.93 + 5.96 q_u + 1.155 z and SPT = 0.51 + 1.066 c + 0.046 z: the important fact is that both show that there is a high degree of significance of the dependence on z, as is illustrated in Appendix I for the case of the vane c.

3. PENETRATION RESISTANCE IN PURE SANDS.

3.1 IMPORTANCE OF PENETRATION RESISTANCES IN SANDS.

Whereas the use of the SPT in clays could be, and was readily, substituted by "undisturbed" sampling and laboratory testing whenever any difficult problems were at stake or the test was questioned, in the case of sands, penetration tests and in particular the SPT acquired a place of exaggerated importance because of the difficulty, expense, and frequent unreliability of special procedures for extracting and testing undisturbed samples, particularly from greater depths and from below the groundwater level. Thus the SPT is eminently implied as a test for granular materials, in which, moreover it had been intuitively assumed that dynamic resistances should not differ much from the static ones.

The initial reference to the use of SPT in sands [Terzaghi and Peck, 1948 (300)] comprises basically the presentation of a tabulation of blowcounts for classification of the relative denseness of sands (very loose, loose, medium, dense, very dense), and without any attempt at connecting with the shear resistance, states that "the relative density of sand strata has a decisive influence on the angle of internal friction of the sand", but errs in failing to mention that the blowcount for a given sand and relative denseness should vary with depth. Burmister 1948 (40) also claims that "for granular soils the driving resistance of the sampler is an index of compactness". Only in Peck et al. 1953 (230) does one find the early indication of a closer connection with ϕ values, but still without incorporation of effects of overburden pressures.

It is presently well understood that if SPT or penetrometer tests have any relation to shear resistances, since shear resistance in sands cannot be dissociated from the normal pressures at play, obviously there was a lapse in establishing the classification of denseness of sands from SPT values irrespective of depth.

Thus it appears appropriate to begin by considering the U.S. Bureau of Reclamation tests [Gibbs and Holtz, 1957 (86)] which really constituted the first set of systematic tests on SPT in sands.

3.2 INTERPRETATION OF THE U.S.B.R. TESTS, EMPIRICALLY, IN FUNCTION OF ϕ.

The research has been very much discussed, and continues to be a source of considerable argumentation, in part because no other systematic tests on this all-important subject have been carried out. The general tendency (1, 5, 46, 51, 52, 57, 72, 79, 85, 93, 149, 169, 262, 289, 353, etc.) has been to relate the (SPT, σ) data to Relative Densities, possibly in part because of the context created by the Terzaghi-Peck (300) indications, in part because greater concern lay with problems of footing foundations, and, doubtless, in part because of the principal direction followed in the original publication itself. It may be noted however that Gibbs and Holtz (86) stated "The increase in penetration resistance resulting from increasing overburden appears to be related to shear resistance, since shear resistance increases proportionally with overburden in sand". Moreover, there has meanwhile been a continuous flow of correlations between SPT values and static cone penetrometer Rp values, which would indicate the possibility of correlating SPT with point resistance and therefore with (σ, ϕ). In short, it appeared worthwhile attempting a close-cycle empirical correlation of the U.S.B.R. data of (SPT, σ) vs. the sands' ϕ values, through some point resistance equation: the statistical regressions would simultaneously indicate if the correlation was at all significant.

As a first step it may be accepted that for a given sand the variation of ϕ with void ratio can well be represented by equations of the type $\tan \phi = A/\varepsilon$ (often mentioned) between the limiting values ε max and ε min separately determined. The discussions as to determinations of ε max and ε min, which of the modernly elusive ϕ values of a sand to employ, the interferences of crushing of sand grains and of in-situ minute cementations between grains, and so on, and so forth, will either be considered hereinafter, or may be set aside as effects not generally relevant to the crude first-order relationships seekable or sought from the SPT. The applicable (ϕ, ε), ε max and ε min values for the two U.S.B.R. sands, as indicated by personal communication, are represented in Fig. 10. The curves of best fit of the type $\tan \phi = A/\varepsilon$ were thereupon established for transforming the RD values into ϕ values. Unfortunately the (ϕ, ε) data are scant and covering only a narrow central range, and some of the data may appear strange: for the sake of comparison the data from the Zolkov and Wiseman, 1965 (353) sands are plotted in the same graph.

The basic data (for the air-dry sands) of the U.S.B.R. tests were thereupon extracted [de Mello, 1967 (183)] as sets of values of (SPT, σ, RD, ϕ) for statistical regression of (SPT, σ, ϕ): for the cases recorded as pertaining to zero overburden pressure the σ value was interpreted as the average pressure (vertical) due to the weight of the sand at 30 cm of depth. The reason for employing the results pertaining to the air-dry tests was purely statistical, because of the greater number of points available for the regressions. .

Because of the intention of analysing such data statistically in the light of a bearing capacity problem, it was necessary initially to postulate a simplified equation of the type desired, despite the recognition that the very subject of bearing capacities of "deep foundations" underwent and continues to undergo very intense discussion, research and development: moreover, it was reasoned that as a first step friction and point resistance parameters could be assumed to enter in the same trend, since the basic intention was merely to establish an empirical, but statistically valid, correlation, along a theoretically acceptable trend. Notwithstanding the very significant differences in point bearing capacity factors for sands [de Beer 1965 (11), Berezantzev et al. 1961 (20), Vesic 1965 (323) and 1967 (325), et al.] it was anticipated that the statistical regression parameters would ably absorb such differences of opinion, if used under acceptably closed-cycle conditions, for first-order estimates.

It was thus resolved for feasibility of the statistical formulation, to resort to the Prandtl-Caquot-Buisman idealized theory, assuming that SPT values could be interpreted with relation to a general shear resistance equation $s = c + \sigma \tan \phi$ and to the values of σ, through a function

$$SPT = f[\sigma \tan^2 (\frac{\pi}{4} + \frac{\phi}{2}) e^{\pi \tan\phi} + \frac{c}{\tan\phi} \{\tan^2 (\frac{\pi}{4} + \frac{\phi}{2}) e^{\pi\tan\phi} - 1] \quad \ldots\ldots\ldots (25)$$

The results of the statistical regression were

$$SPT = 4.0 + 0.015 \ \frac{2.4}{\tan\phi} [\tan^2 (\frac{\pi}{4} + \frac{\phi}{2}) e^{\pi\tan\phi} - 1] +$$

$$+ \sigma\tan^2 (\frac{\pi}{4} + \frac{\phi}{2}) e^{\pi\tan\phi} \pm 8.7 \quad \ldots\ldots\ldots (26)$$

Appendix I summarizes the results of the analyses conducted separately for the coarse sand, and for the fine sand, as well as jointly both sands (giving the above equation) and further summarizes the statistical proofs that the regressions are highly significant, to 95% confidence limits.

The results obtained are more conveniently summarized in Fig. 11. Moreover in Fig. 12 (a) and (b) the results computed separately for the coarse and the fine sand, for two distinct values of ϕ (35^o and 40^o) are superimposed to indicate that within 90% of confidence the equation determined for one sand comprises the data pertaining to the other sand.

It may be noted that in the basic equation the value of c (cohesion) was purposely retained with a view to allowing the statistical regression to determine the result without any preconceived imposition. It might appear odd that a value of $c = 2.4\ t/m^2$ should have been revealed: it can well be seen, however, that such a value is quite plausible. For $\phi = 0^o$ the expression between brackets {} in equation 26 degenerates to (π + 2) c and thus one concludes that the average value of SPT would be SPT = 4.0 + 0.015 (2.4 x 5.14) = 4.0 + 0.185, in which the contribution of the apparent cohesion is quite negligible, and quite plausible for pure sands

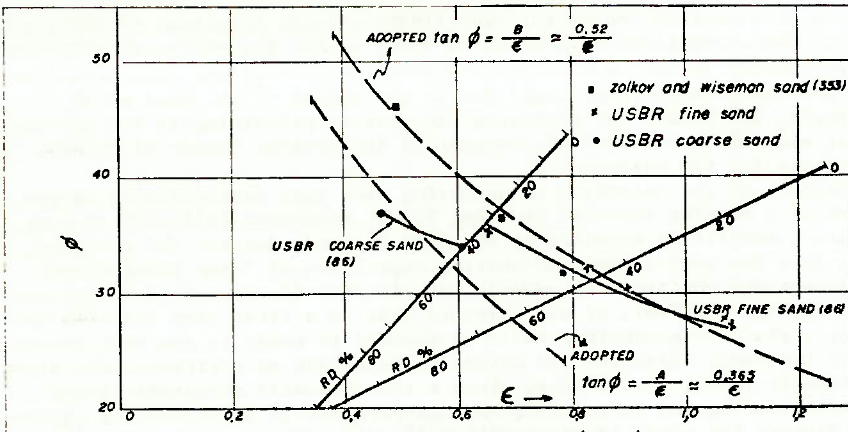

Fig. 10 Basic data for SPT = f(σ, φ)

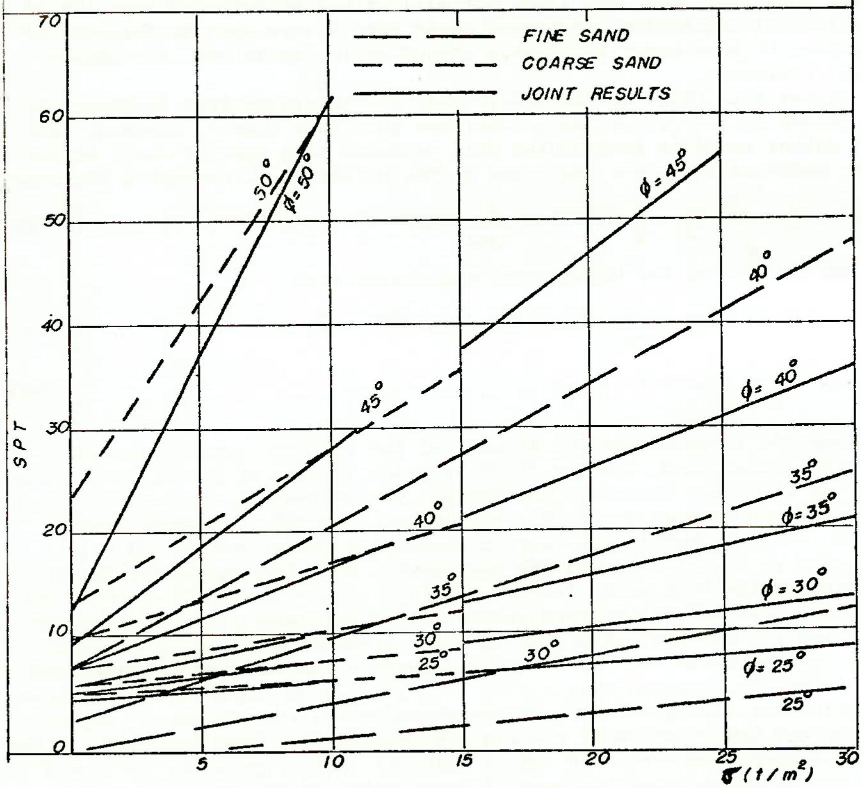

Fig. 11 Results of statistical analysis of USBR tests (86)

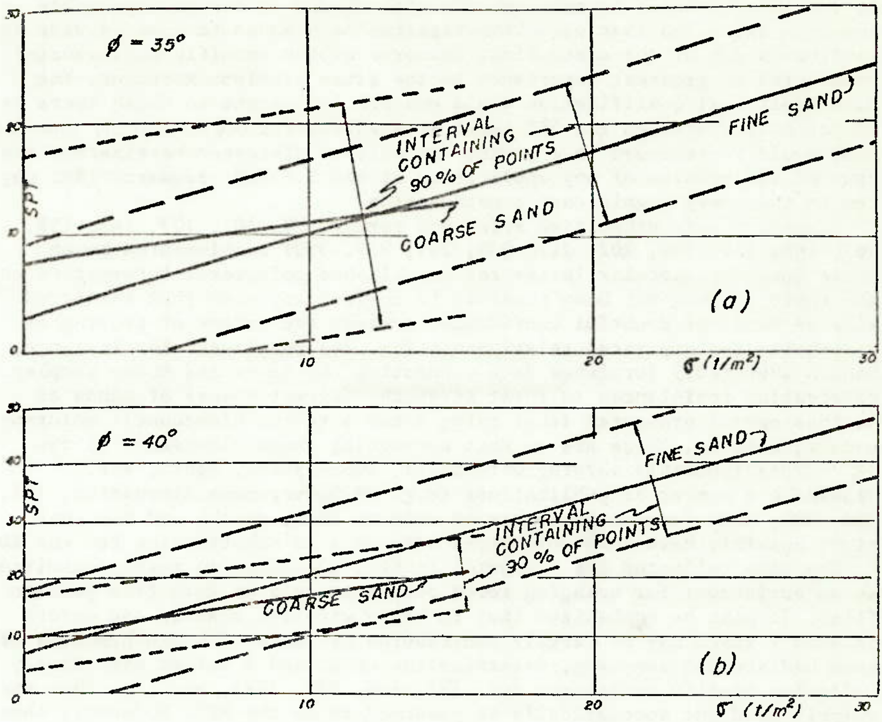

Fig. 12 Confirmation of similarity of statistical regressions SPT = f(σ, ϕ)

Apud de Mello, 1967 (183)

considering the slight curvature of the Mohr-Coulomb envelope, particularly bearing in mind the crushing of grains that is probably involved [de Beer, 1963 (10)].

In conclusion, it may be seen that there is a good statistical correlation between the pairs of values of (σ, SPT), and the probable nominal ϕ values from routine triaxial tests, such a statistical correlation being of a type analogous to point resistances of deep foundations. Since the proposed correlation is strictly empirical, connected with measured vertical σ values and triaxial test ϕ max values, it may be expected that the applications of the correlation in closed-cycle conditions should yield satisfactory results: thus, if one knows vertical σ values and SPT values in a given sand, one should be able to extract nominal triaxial test ϕ max values, insofar as the sampler penetration phenomenon may be reasonably related to deep foundation failure phenomena and the (σ, ϕ) values really prevalent during this phenomenon bear a consistent similar relationship to the nominal (vert. σ, ϕ max) as obtained in the test values adopted.

It must be conceded that although the sands are presumed to be descriptively different (fine vs. coarse), the two may indeed have been nearly identical in the face of the phenomenon at play: little is known on the finer details to reassure one with regard to the indispensable research intention that each investigation be planned to span as wide as possible a gap of the statistical universe of the specific parameter(s) considered of greatest importance to the given problem. Moreover, the main additional qualification would concern the extent to which there is compatibility between the SPT tests as executed for the research, and those that would be recorded in a boring. As will be discussed hereinafter the crux of the problem of any application of the U.S.B.R. research (86) may lie in this very significant consideration.

Despite a very exhaustive reference survey (79, 101, 107, 141, 159, 167, 169, 170, 180, 202, 217, 239, 291, 339, 353) complemented by an ample quest by circular letter for unpublished collateral information on the topic, it has not been possible to collect any more than scattered sets of data, of doubtful confidence, towards the intent of proving or disproving the empirical relationship (eq. 26) developed. For instance, Rendon 1969 (239) furnishes data connecting the Dames and Moore sampler penetration resistances to shear strengths (direct shear) of sands at various normal pressures (thus tying σ and ϕ to the blowcount): unfortunately, however, there are no data connecting these blowcounts to the SPT values (possibly varying with soils, submergence, depth, etc.). Meanwhile a number of publications (e.g. 79 Schmertmann discussion, 101, 202, 339, 353) furnish good sets of data on (SPT, depth, and RD) that might possibly have some collateral data on ϕ in substitution for the RD.

The data collected are presented in Fig. 13, which is merely submitted as an enticement for bringing forth scattered sets of data from private files. It must be emphasized that to begin with the scatter and errors in such a graph may be largely contributed by the well-known problems of sand undisturbed sampling, determination of RD and ϕ values effectively valid for in-situ conditions (cf. 141, 149, 271, 292), etc., so that the culprit need not automatically be presumed to be the SPT. Moreover, there are sand deposits (e.g. of glacial or colluvial origin) that can automatically be anticipated to be very heterogeneous in denseness, or relative denseness [cf. Wu, 1959 (344)], from point to point, and wherein the scatter of results of any density tests may be expected to be altogether frustrating for an investigation of such a nature. The important hope, however, is that the data brought forth may furnish information on the problems raised under item 2.2, in order to substantiate, revise, or overthrow the presumed correlation <u>as applicable borings</u>. It is of utmost importance to this study, to emphasize that if the discussions of item 2.2 are valid, it cannot be expected that the U.S.B.R. (86) test results, however they be interpreted, be found applicable to borings and SPT values at various depths: superimposed on the σ effect reflected in eq. 26 there should be the depth effects of Figs. 1 and 2, depending on the rods etc., especially considering that the SPT value were principally recorded for zero rod length.

By far the most valuable data for the present appraisal were extracted from the boring profiles (Figs. 68, 70) supplied by Vesic, 1967 (325). These data can be accepted as pertaining to a sand apparently similar to the U.S.B.R. sands. It can be seen in Fig. 13 that the data confirm the expectation, based on the theoretical reasoning, that measured triaxial ϕ values should be consistently greater than those derived from the U.S.B.R. data, through the statistical regression Eq. 26, Fig. 11. The "measured" values were derived from the ε values by adopting $\tan\phi = 0.59/\varepsilon$, similar to the Chattahoocchee River Sand, in accordance

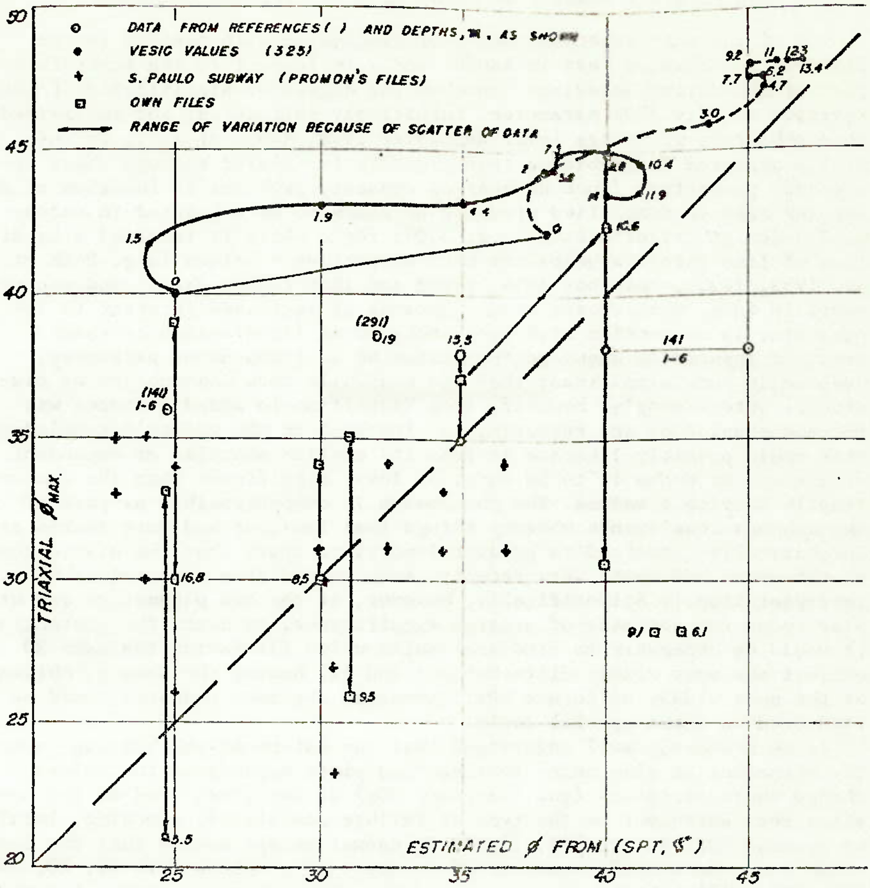

Fig. 13 Comparison of triaxial ϕ_{max} values with ϕ estimated from (SPT, σ) by Eq. (26).

with a personal indication of the author. If there were no effect differentiating routine borings from the U.S.B.R. penetration tests, there should be equivalence of the ϕ values determined by the two approaches, since the application is absolutely closed-cycle: if, however, the problems of shallow depth driving of the SPT are as postulated, the U.S.B.R. tests should require greater SPT blowcounts for any given condition, and, therefore from any given (SPT, σ) condition the interpretation via SPT using the U.S.B.R. data should lead to low estimated ϕ values.

In conclusion, it appears proven that the U.S.B.R. data lead to underestimates of "equivalent triaxial ϕ" values of the sands. Moreover, the Standard Penetration Test appears to be subject to greater than usual (and possibly unacceptably great) variations and errors in sands at very shallow depths.

3.3 RELATIVE DENSITY OF SANDS AS A SIGNIFICANT PARAMETER

One of the most important areas of discussion with respect to the Standard Penetration Test in sands, and with respect to the interpretation of foundation behaviour concerns the degree of significance of the Relative Density (RD) parameter. Intuitively introduced, and subdivided into arbitrary groupings [e.g. Burmister 1948, (40), et al.], it initially appeared in problems traditionally formulated through shear resistance parameters (such as bearing capacity problems in function of ϕ) for the sake of simplified grouping of cases to be tabulated in recommended design criteria etc. (e.g. 300); for a while it retained a condition of free interchangeability with respective ϕ values [e.g. Peck et al. 1953, (230), Meyerhof 1956, (190) and 1959 (191), etc.]; and imperceptibly in a more recent stage, because of increased interest in the parameter in connection with such problems as liquefaction of sands, etc., it apparently acquired the status of an independent parameter, eventually more significant than the meanwhile more commonplace or elusive ϕ. Interestingly, however, this transition to added statures was not accompanied by any research, to the best of the writer's knowledge, that could possibly liberate it from its earlier shackles of dependent parameter, or prove it to be more, or less, significant than the concomitantly varying ϕ values. The phenomenon is comprehensible as part of the subconscious trends whereby things less familiar and more feared are automatically totemized to greater importance [note that the static cone penetrometer had until very recently been spared from such trends of interpretation.]. Scientifically, however, if the two parameters are at play vying for position of greater significance, to clear the contention it would be necessary to find some soils which (1) having the same RD exhibit the most widely different ϕs; and (2) having the same ϕ, obtain at the most widely different RDs: thereupon the moot research would be conducted on these special soils.

It is presently well understood that the nature of the bearing capacity phenomena at play under footings and piers depends on the volume-change characteristics (and therefore RDs) of the given sand at its density: yet, whichever be the type of failure postulated, punching, local, or general [cf. Vesic 1967, (325)] it cannot escape notice that the formulae are consistently established in terms of ϕ values [cf. 11, 20, 141, 191, 325, 347], which, moreover, are also, in each given sand a function of RD.

The writer will temporarily (within the present report) set aside the criticism that even if the RD tests were well standardized they represent a conceptually unsatisfactory solution to the real problem, because they cannot be applied to the vast majority of soils (clayey sands, silts, etc.) that are situated in between the idealized cases of "pure" sands and clays, and have accentuated problems of in-situ structure etc.. If the fundamental parameters desired are shear strength, and "compressibility", and the former can be established directly, the best approach is to try to establish the latter also directly (e.g. the cone penetrometer Rp approach by Buisman, et al.) rather than persisting in a completely circuitous approach altogether reproachable both by the theory of errors and by the accumulated knowledge of soil behaviour.

As a first step it was decided to investigate the confidence limits of the eventual universality of the relation between ϕ and RD, ever consciously denied, but always implicitly employed: this was undertaken simply through an extensive reference survey of the greatest possible number of data on variation of $\phi = f$ (RD) for different sands. The vast amount of information (12, 21, 23, 24, 32, 33, 36, 40, 44, 48, 50, 57, 60, 76, 90, 105, 108, 115, 125, 127, 128, 130, 140, 142, 146, 148, 151, 160, 168, 169, 172, 175, 177, 178, 179, 190, 195, 214, 226, 232, 247, 268, 270, 288, 289, 292, 306, 310, 325, 334, 340, 344, 353, and data gratefully received through personal communications) has been analysed statistically to furnish two important bands of 95% confidence limits around the average function: for the statistical regression the form of equation chosen was $\tan \phi = A/\varepsilon$ as is generally quoted as valid.

These data are summarized in Fig. 14. Because of the very great number of points available, extracted directly or indirectly from data as routinely published, it can be seen that the 95% confidence limits of the average curve are very narrow. Moreover, by coincidence the U.S.B.R. (86) sands lie close to the general average: thereupon it may be reasoned that in problems in which the sands are similar to the U.S.B.R. sands, or the phenomenon at play involves cumulative effects (e.g. settlement problems of large footings) of a variety of sands, the average behaviour may be well simulated by that of the U.S.B.R. sands. However, because of the very wide scatter of the points available, it is simultaneously concluded that the 95% confidence limits of representing any single point (i.e. a given case or soil) are absurdly wide. That means, obviously, that if any sand is not closely similar to the U.S.B.R. sands, the chance of adequately representing the behaviour by analogy with the U.S.B.R. results will be very small; moreover, if any behaviour depends principally on a local "trigger phenomenon" or if each individual test value is considered along profiles of ϕ vs. RD results, the scatter will be very wide unless the sand deposit has been naturally rendered homogeneous through highly selective deposition pertaining to the geologic processes.

It must be explained herein that it is well recognized that (1) both the maximum and the minimum void ratios of a sand are subject to much discussion as to test procedures [cf. Kolbuszewski, 149, 150, 151, 152, and a recent overall appraisal by Tavenas and La Rochelle (1970), 292]; (2) there even occurs some confusion of definition of RD (cf. 292); (3) values of ϕ of sands are also subject to significant variations dependent on a great variety of test and interpretative procedures; and so on. However, it was reasoned that in any current discussion of practical application of ϕ and RD values, the above difficulties obtain inevitably, and therefore the range of variation depicted in Fig. 14 comprises a valid indication to assess the current practical usefulness of any universal $\phi = f$ (RD) relationship, irrespective of incompatibilities of some of the data used, and of invalidations of other parts of the same data through more meticulous tests and interpretations.

From Fig. 14 one sees immediately that if ϕ is the more significant first-order parameter controlling (SPT, σ) values, RD cannot be generally substituted for it, since at the same RD different soils exhibit ϕ values within a range of $\pm 12^{\circ}$ (or even more) from the average curve. On the contrary, if RD is the more significant first-order parameter, and especially non-average sands had been tested, it would be found that a regression based on ϕ would not be found applicable.

Incidentally, since the U.S.B.R. sands are really quite average and similar, it is unnecessary to confirm the applicability of a statistical regression of (SPT, σ) on RD quite as significantly as the regression on ϕ: it can be accepted as proven by foregone conclusion. The only problem is that one would be somewhat at a loss to invent the appropriate

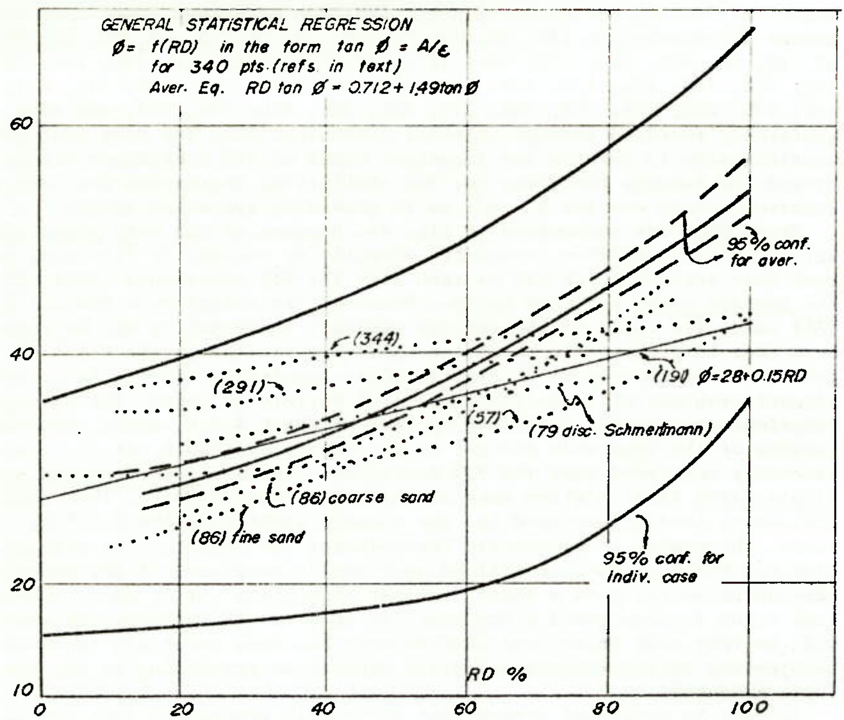

Fig. 14 Dispersion around universal relation $\phi = f(RD)$.

function for the regression: a function such as that of eq. (25) in which one substituted $\tan \phi = A/\varepsilon$ would be found applicable.

3.4 INTERPRETATION OF THE U.S.B.R. (86) RESULTS ON (SPT, σ) IN TERMS OF RELATIVE DENSITIES, AND DISCUSSIONS.

As is well known, Gibbs and Holtz, 1957 (86) chose to interpret their results in terms of strictly empirical curves of variation of SPT as a function of RD and σ values. The results of these tests have been intensely discussed, automatically accepted by some, criticized by others with respect to some apparent errors, and mechanically readapted for more convenient use by yet others: to the best of the writer's knowledge, however, they have never been broadly discussed with respect to the fundamental problems at stake, and with respect to some of the concepts that are explicit or implicit in the line of interpretation adopted.

It seems useful to subdivide into several distinct steps the reasoning incorporated into the said line of interpretation, in order that one may analyse more profitably the discussions waged around the subject.

3.4.1. The (SPT, σ) data are taken as indicative of the RD of the two sands.

Obviously for a given sand there is a very close correlation between ϕ and RD; moreover, for two sands that are quite similar as regards the relationship ϕ = f (RD), there continues to be a single close correlation between ϕ and RD, so that if any phenomenon is dependent on ϕ, it is simultaneously correlatable to RD. With respect to this step the only problem lies in the fact that no indication is given on the confidence band covering the observed points [cf. Tavenas and La Rochelle, 1970 (292)]. It appears unnecessary to mention such a further error (possibly second-order and probably compensatory) as that of determining the RD after the six penetrations were driven (loc. cit.) although it might be responsible for a systematic error of presumption of RD values somewhat higher than obtained, under the looser conditions.

3.4.2. The assumption that by analogy the (SPT borings, σ overburden effective) for similar sands will bear out the same relations to furnish RD.

As shown under item 3.3 the U.S.B.R. sands fall close to the average function ϕ vs. RD for sands (Fig. 14): therefore, to the extent that average values are extracted, from great numbers of data and for average sands, the same relationship (SPT, σ) vs. RD should be expected in any tests similar to those conducted by the U.S.B.R.. The principal questions are: to what extent are the U.S.B.R. applied σ values analogous to the σ_{oe} (overburden effective) that prevail at the bottom of a boring at the moment when the penetration test is conducted; to what extent are such σ_{oe} values significant to the spoon penetration resistances, when one of the important stresses at play should be the horizontal σ [which may be affected by precompressions etc. cf. Zolkov and Wiseman, 1965 (353)]: finally, to what extent are the U.S.B.R. SPT blows with zero rod length analogous to SPT blows at various depths in a boring?

As regards the first question, it is indeed a pity that no "holing condition" was tested: indeed the 4 ft. deep tank precluded such testing, but the need was not remembered or tended to when Schultze and Melzer, 1965 (263) apparently did not profit of their bigger tank to evaluate this factor with reference to their penetration devices. One must exclude from discussion the "errors" already briefly covered under items 2.3 and 2.6 whereby the material and stress disturbance at the bottom of a boring may completely vitiate results: the question remains as to how stresses readjust at the bottom of the hole, with or without proximity of the casing etc., since one knows from theory and experiment that very minor strains may alter the lateral stresses quite significantly. Pending further investigations it will be assumed, however, that for the 2 1/2 to 3" hole the state of stress after the initial 6" penetration is sufficiently close to in-situ conditions.

The second question, concerning the σ_h (horizontal) stress with or without precompressions etc., has come to be very important, meriting special attention with regard to sand strata. Tavenas and La Rochelle, 1970 (292), raise this point as a significant criticism to the U.S.B.R. tests, reporting to Zolkov and Wiseman, 1965 (353), and would therefore suggest that at higher RD values the SPT observations are vitiated by progressively greater errors on the high side. Therefore if this error exists, it should be patented by steeper d SPT/dσ relationships, with no error at the low RD values. It is the writer's opinion that depending on the manner in which the higher relative densities are achieved, no such precompression effect takes place: since in the present case "the soils were compacted by a vibrator working up and down the center axis ... (and) also attached to the sides of the tank", the dense sands are truly "normally consolidated" and merely packed in a denser grain to grain arrangement, with lower K_o (cf. $K_o \approx 1 - \sin \phi$) than obtain in the looser conditions.

The third important question concerns the acceptance of zero rod-length blowcounts as valid for varying lengths of rods, as occur in any boring. The analysis summarized under item 2.2.4 would indicate this assumption to contain a fundamental, significant error. Quantification of the effect purely on the basis of the formulae (e.g. Figs. 1 and 2) would be unwarranted: but it appears that the unquestioning acceptance of the U.S.B.R. SPT blowcounts as tantamount to blowcounts in borings may be a source of significant error (despite the fact that the said research checked for the effect of rod-length at three specific lengths: note that three experimental points are insufficient for defining a curve), responsible for some of the discussions e.g. (57) disc.. Based on the trends illustrated on Figs. 1 and 2, and assuming that no additional important revision occurs at z = 0 m, one would conclude that the ratio of U.S.B.R. - SPT to boring-SPT would always be greater than 1, except for the depth of equal penetration efficiency. Therefore, at a given RD for a given depth $z \neq 0$ and presumed normally consolidated σ_{oe}, the trend should be for extrapolated U.S.B.R. blowcounts to be higher than the real SPT; in other words, the U.S.B.R. relations would lead to an underestimation of the sands' RD on this count.

Fig. 15 presents the U.S.B.R. curves of (SPT, σ) vs. RD. Superimposed on it are plotted the corresponding data from borings, in which one may be assured that the ϕ = f (RD) function is analogous, and the sand may be assumed to be "normally consolidated". The best data located were carefully extracted (with some exclusion of questionable results) from the boring profiles of Figs. 68 and 70 of Vesic 1967 (325), for which the function $\tan \phi = 0.59/\varepsilon$ was indicated, quite close to the corresponding equation of best fit for the U.S.B.R. fine sand, which was $\tan \phi = 0.52/\varepsilon$. It is patent in Fig. 15 that the SPT values in the borings are consistently lower than indicated by the U.S.B.R. research, so that, for instance the real blowcounts are 17 and 30 vs. predicted 25 and 37, and so on: meanwhile predicted RD values are lower than measured ones, as for instance 71 and 79% vs. 83.5 and 86.5% respectively, as may be read off the graphs.

Note that Peck and Bazaraa, 1969 (57 discussion) furnish data on 7 cases, five of which would oppose the above conclusion by indicating predicted values of RD between 137 and 174% of the measured values, while two would indicate predicted values of 83 to 90% of the measured ones: the authors state that the Gibbs and Holtz relation "generally overestimates the relative density of the deposits". The same form of interpretative analysis is employed by Tavenas and La Rochelle, 1970 (292), who transcribe the "average" points quoted by Peck and Bazaraa, and complement them with additional data, but conclude "that it is not only difficult, but virtually impossible, to compute RD from the knowledge of SPT only". It is far from easy to assure oneself "similar, uncemented, normally consolidated sands" for comparing U.S.B.R. predicted ϕ (or RD, quite interchangeably, if they are similar sands) values with observed, fide-dignae, values: since the data are scant, very scattered, and individually subject to question of significant proportions on certain points, these divergences of opinion may be discussed below after consideration of the other very significant intervening factors.

3.4.3. The assumption that by analogy the (SPT borings, σ_{oe}) will be indicative of RD of natural deposits of sands in general.

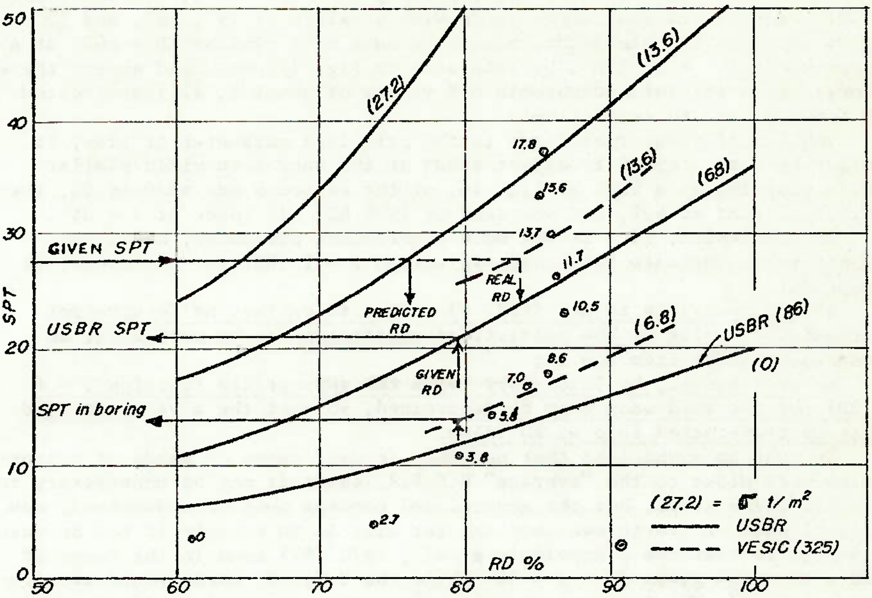

Fig. 15 Comparing Gibbs-Holtz (SPT, σ) vs. RD with data from borings (e.g. Vesic 325).

Let us consider, for a start, only the case of uncemented normally consolidated sands. The first point must be the acceptance that the statistical analysis of (SPT, σ) vs. ϕ, Eq. 26, yielded a highly significant regression analogous to pile or static cone penetrometer point bearing capacity: meanwhile no satisfactory function has been found for an independent regression of (STP, σ) vs. RD. Although this is not proof positive, it is good circumstancial evidence, accumulated on the analyses and statistical regressions available on clays, etc..

To begin with, the fact that some sands permit satisfactory use of the (SPT, σ) vs. RD U.S.B.R. relation, does not constitute proof of the dependence of SPT on RD as principal parameter. Any sand with a function ϕ = f (RD) satisfactorily similar to that of the U.S.B.R. sand should yield the anticipated trends, as already discussed under item 3.4.2.

Now if the premise of the predominance of the shear resistance parameter prevail, it is obvious by a glance at Fig. 14 that since there is no universal function ϕ = f (RD) at all practicable within the widest tolerable limits of confidence, there cannot be a satisfactory relation of (SPT, σ) vs. RD for all sands.

The U.S.B.R. sands would exhibit an "equivalent* triaxial ϕ" value of 36° at RD = 60%; the "average" sand would exhibit ϕ = 38°.5; but at the same RD there are sands with ϕ values presumably ranging between 19°

*It must be emphasized that this may not be at all closely similar to the ϕ value really at play in reacting to the spoon's penetration (cf. item 3.6).

and 50° (on the basis of present data, fraught with errors). Obviously if shear resistance is at play, depending on σ and tan ϕ, it cannot possibly be expected that three sands with ϕ values of 19°, 36°, and 50° give the same SPT blowcount, merely because of a similar RD = 60%. At a hypothetical σ = 10 t/m², by reference to Fig. 11 one would expect these three sands to yield comparable SPT values of about 2, 12 (uncorrected U.S.B.R.), and 60 respectively.

Indeed, if shear resistance is the principal parameter at play, it might be more logical to expect sands at the same ϕ to yield similar SPT: that is, by a look at Fig. 14, at the extremes one sand at 0%, the U.S.B.R. sand at 60%, and one sand at 100% RD, all three at ϕ = 36°.

In conclusion, if ϕ is the more significant parameter, and since there is no adequate universal expression ϕ = f (RD) for all sands, it appears:

a) indispensable to use (SPT, σ) values to extract an "equivalent triaxial ϕ" value by the statistical regression (to be corrected, as concluded under item 3.4.2);

b) as a sequel, by laboratory tests the appropriate function ϕ = f (RD) for the said sand must be determined, so that the ϕ value derived may be transformed into an RD value.

It must be emphasized that possibly in many cases of sands of properties very close to the "average" U.S.B.R. sands it may be unnecessary to follow these steps, but the generalized concept must be understood, and established or overthrown once and for all. As an example it can be seen in Fig. 14 that the D'Appolonia et al., 1970 (57) sand in the range of 60 < RD < 80% gives 32 < ϕ < 36° while the U.S.B.R. sands would average 36 < ϕ < 42°. The increase of about 5° in the average ϕ would automatically imply, upon acceptance of the interpretation of Fig. 11, that the correct use of the Gibbs and Holtz (86) data would require adjustments of the blowcounts of just about the double of what was used (considering only the ΔSPT/$\Delta\sigma$ slopes), i.e. 4 to 6 blows instead of the 2 and 3 blows.

Moving on to consider sand deposits in general, it is today well established that most sands develop a grain-to-grain cement with age (61). Thus, without a change of RD, and aged in-situ sand may possess an increased shear resistance that should also reflect in higher SPT blowcounts in the field.

Finally, it is presently considered that overconsolidation or precompression of sands affects SPT values significantly because of "locked-in" higher horizontal stresses [cf. Zolkov and Wiseman, 1965 (353), Bazaraa, 1967 (5), Tavenas and La Rochelle, 1970 (292), etc.]. Thus, under a given σ_{oe} vertical stress the real stresses affecting the spoon penetration resistance may be quite variable, presumably under essentially unchanged RD values. This topic will be further discussed (item 3.5).

Under this item, therefore, one finds two factors tending to raise the field measured SPT blowcounts, in comparison with U.S.B.R. anticipated values (vs. the one factor of opposing trend of item 3.4.2); correspondingly field predicted RD values should be higher than the real values in such cases. All three effects, however, are probably minor in comparison with the effect incumbent upon possible major differences of ϕ = f (RD) relations for different cohesionless materials.

The discussion as to whether or not RD values estimated by (SPT, σ_{oe}) values, using U.S.B.R. data, are generally valid, appears altogether sterile from a theoretical premise, irrespective of errors. Fig. 16 reproduces in a more explicit form the data behind the "average points"

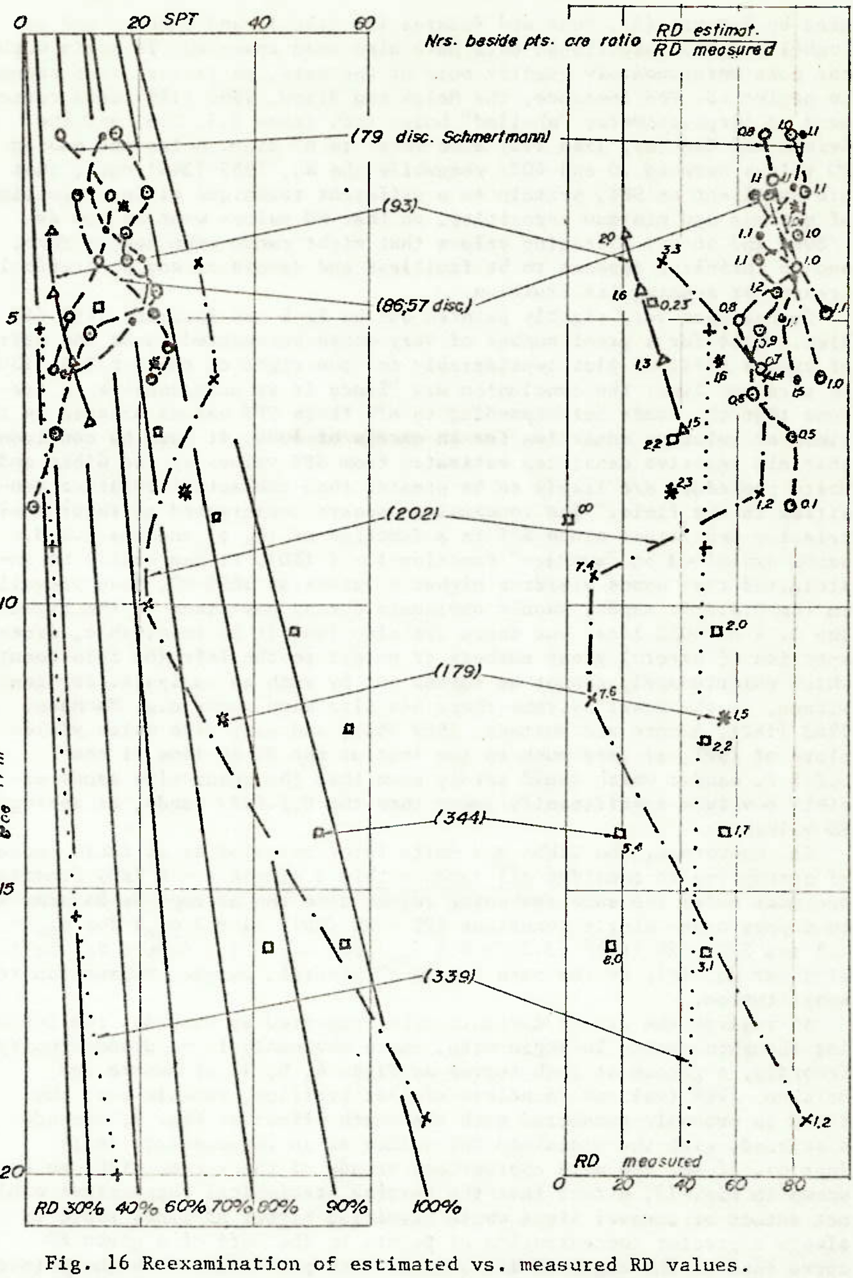

Fig. 16 Reexamination of estimated vs. measured RD values.

used by Bazaraa (5), Peck and Bazaraa (57 disc.), and Tavenas and la Rochelle (292): additional data have also been inserted. To begin with, one must unfortunately qualify some of the data, on factors that cannot be neglected. For instance, the Meigh and Nixon, 1961 (179) data correspond to large-diameter "shelled" holes (cf. items 2.3, 2.6) and the Grimes and Cantlay, 1965 (93) also refer to 6" diam. holes and quoted RD values between 20 and 40%: meanwhile the Wu, 1957 (344) data, that are excellent on SPT, pertain to a different technique of determination of maximum and minimum porosities, so that RD values went as low as - 80%. And so on: averaging values that might range even beyond zero, and to infinity, appears to be fruitless and devoid of any theoretical reason for seeking its fruition.

It has been very rightly pointed out by Peck and Bazaraa, 1969 (57 disc.) that for a great number of very dense uncemented sands the pairs of values (SPT, σ) plot considerably to the right of the U.S.B.R. 100% RD straight line: the conclusion was "Since it is unreasonable to presume that the sands corresponding to all these SPT values existed in the field at relative densities far in excess of 100%, it must be concluded that all relative densities estimated from SPT values by the Gibbs and Holtz procedure are likely to be greater than the actual relative densities in the field." The conclusion appears unwarranted as regards all relative densities: since SPT is a function of (σ, ϕ) and the U.S.B.R. sands exhibited an "average" function $\phi = f$ (RD), it can easily be anticipated that sands yielding higher ϕ values at 100% RD, than prevailed in the U.S.B.R. sands, should obviously occupy positions to the right of the G. & H. 100% line: but there are also (and it is inevitable, irrespective of errors) great numbers of points to the left (on this count) which unfortunately cannot be sorted out by such an analysis. For instance, at the lower extreme there are also many sands e.g. Machado, 1963 (167), Basore and Boitano, 1969 (4a), and many like which yield plots of (SPT, σ) very much to the left of the 0% RD line of the U.S.B.R. sands: which would merely mean that the respective sands exhibit ϕ values significantly lower than the U.S.B.R. sands, at analogous RD values.

If, therefore, the Gibbs and Holtz interpretation is at fault because of attempting to consider <u>all sands within a single</u> $\phi = f$ (RD) function, one must under the same reasoning reject also the attempt by Bazaraa (5) to supply other single functions $SPT = 20\ (RD)^2\ (1 + 2\ \sigma_{oe})$ for $\sigma_{oe} \lessgtr 1.5$ and $SPT = 20\ (RD)^2\ (3.25 + 0.5\ \sigma_{oe})$ for $\sigma_{oe} \lessgtr 1.5$ (where σ_{oe} is in kips per sq. ft), as the more "correct" general, unique, expression in substitution.

As regards the linear discontinuity suggested by Bazaraa, the following thoughts occur. To begin with, there obviously is no discontinuity. Secondly, a glance at such curves as Figs. 4, 9, 11 of Basore and Boitano, 1969 (4a) and countless similar profiles, reveals that the trend is probably connected with the depth effect of Fig. 1, already discussed, with the minimized SPT values at an intermediate depth (not σ). If so, the more appropriate trends of the curves might be as shown in Fig. 17, a fact that the Bazaraa statistical regressions could not detect or unravel since while examining higher RD lines there is always a greater concentration of points to the left of a given RD curve than to the right of it, because such points mix with the points belonging to the right of the next lower RD curve. Below the troughs in SPT values the nearly straight lines of SPT vs. σ_{oe} may be appropriately examined for $\Delta SPT/\Delta\sigma_{oe}$ (including also the $\Delta SPT/\Delta z$ effect of Fig. 1): such regressions, corrected for z, may possibly furnish the best means of establishing in "homogeneous" sand deposits, the "average" ϕ value.

3.4.4. The decision to connect numerically established RD values to the qualitatively described states of relative denseness which Terzaghi and Peck, 1948 (300) associated as conditioning settlements of shallow footings on sands.

So considerable an amount of effort has been expended in discussing this fourth step implicit in the Gibbs and Holtz, 1957 (86) presentation that the topic may be summarized herein, although it cannot be disregarded as a really distinct step, especially since it has been incorporated into many mechanistic design adaptations (e.g. 52, 289, 296, 303) as to how to "correct" measured SPT blowcounts into closer agreement with the Terzaghi-Peck footing design recommendations.

It was quite clearly established by the Gibbs and Holtz research that Terzaghi and Peck had slipped in neglecting the fundamental parameter σ of the shear strength of sands, when they indicated the classifications of the denseness of sands by SPT blowcounts irrespective of depth. The subsequent step, however, of the criticism of the T. & P. preliminary recommendations was somewhat sidetracked because of the form of graph adopted by G. & H. to present their data. If the same data were interpreted directly (independently of the qualitative descriptions of states of denseness, and the quite arbitrary subdivisions in terms of RD), through a presentation such as Fig. 11, assuming that shallow footings pertain to σ_{oe} values of the order of 1 to 3 t/m^2 at least, we obtain the following correspondence of SPT vs. ϕ values. (see table).

In the same table can see, without having to relinquish the "average" U.S.B.R. sands, the possible results of a geologic precompression effect to pressures of the order of 10 t/m^2.

<table>
<tr><td>ϕ^o</td><td>25</td><td>30</td><td>35</td><td>40</td><td>45</td><td>50</td></tr>
<tr><td>STP_1</td><td>1.7</td><td>3.5</td><td>6</td><td>10</td><td>13</td><td>24</td></tr>
<tr><td>STP_3</td><td>2.2</td><td>4.5</td><td>7</td><td>12</td><td>17</td><td>28</td></tr>
<tr><td>T.P. classif.</td><td colspan="6">0 —— 4 —— 10 —— 30
V. Loose Loose Medium</td></tr>
<tr><td>STP_{10}</td><td>3.6</td><td>7</td><td>11</td><td>19</td><td>29</td><td>61</td></tr>
<tr><td>T.P. classif.</td><td colspan="6">0 —— 4 —— 10 —— 30
V. Loose Loose Medium Dense</td></tr>
</table>

Indeed for "average" sands there is an excessive conservatism in the T. & P. recommendations, as is already universally accepted, and consequently there should be every effort towards consciously and judiciously narrowing this margin of over-conservatism [cf. for instance, Meyerhof 1956 (190) and 1965 (192)]. The fundamental question is how to apply the corrections.

It certainly should not be via the "corrected-SPT" (SPT_c) approach, which was tackled graphically by Thorburn, 1963 (303) and Sutherland, 1963 (289), and which is epitomized by the Coffman 1960 (52) and Teng 1962 (296) linearizations yielding the recommended use of

$$SPT_c = SPT \frac{50}{p + 10} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (27)$$

where $p = \sigma_{oe}$ in psi at the depth of the penetration test.

As regards the classification of states of relative denseness, it must be remembered that: (1) it is conceptually unacceptable to abandon a position gained, of greater specificity of knowledge, to retreat to one of more qualitative and preliminary evaluation. Thus, if the (SPT, σ) information can give estimated values of ϕ (or even of RD in cases, as already discussed) it is incomprehensible that one should retreat to "groups" or classification adjectives that are quite arbitrary. There are, indeed, fields of engineering (such as highways, earthwork, etc.) wherein strategy dictates that it is better to retreat to see the forest rather than lose oneself amid tree-trunks: such is not, however, the case in foundation engineering. Moreover, not even the argument of compactness of communication holds, since the recorded SPT values, and the presumed σ_{oe} and nominal ϕ values, may be summarized beside a boring profile much more telegraphically and meaningfully than through any adjective. A quite similar conceptual discussion affects the use of clay-group classifications (CH, MH, etc.) for foundations, if, as is usually the case, the engineer has to go through w_L and w_P determinations, and might much more specifically transfer the information in the form w_L 110 w_P 45 rather than merely CH;

b) to the best of the writer's knowledge there is yet no established experimental information covering the wide range of heterogeneities of sands, to differentiate important parameters of behaviour (e.g. compressibility, liquefaction potential, etc.) as functions of RD as the first-order parameter: moreover, it would appear prudent to shy from the separation of such categories of behaviour by the hopeful linearized RD subdivisions:

c) the adjectivation of states of relative denseness belonged to a period prior to the more advanced knowledge fostered by the U.S.B.R. tests: henceforth if one can evaluate relative denseness through other means (e.g. geophysical, etc.), it is reasonable that one attempt to move from such information to the assumption of SPT values, and thence, from appropriate direct correlations, to other important parameters. To go from SPT to states of denseness is really going the other way, backwards.

As an important conclusion it is recommended herewith that classification of states of denseness be used, appropriately, where (SPT, σ) data are not available: where they are available, along borings, either one advances towards quantifying ϕ (or even RD) values, or it is better to forego employing adjectives that have only caused confusion and discussion. This conclusion tallies with the recommendation made for classification of consistency of clays via SPT (item 2.9.4).

3.4.5. The revision of the Terzaghi-Peck prescriptions of allowable q_a values on shallow footings in order to avoid excessive differential settlements.

As a final step of the trend of interpretation initiated by the Gibbs-Holtz tests one finds the conclusion "the Terzaghi-Peck correlations are on the conservative side for shallow footing work for which they were intended". And, thereupon a number of authors have followed the line of suggesting "corrections to the correlations" (e.g. 52, 289, 296, 303, etc.). They were really not correlations, but possibly prescriptions: and, whenever appropriate, a prescription or recommendation can and should be altered directly [e.g. Meyerhof, 1965 (192)], without any circuitous "correction" of the data on which it is supposedly based.

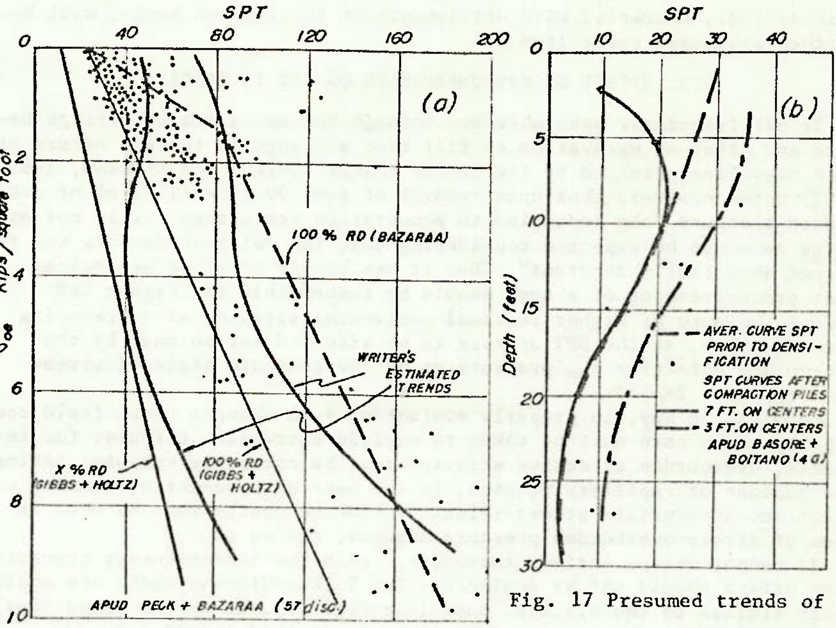

Fig. 17 Presumed trends of SPT (a) vs. σ_{oe} (const. RD), for curves such as (b).

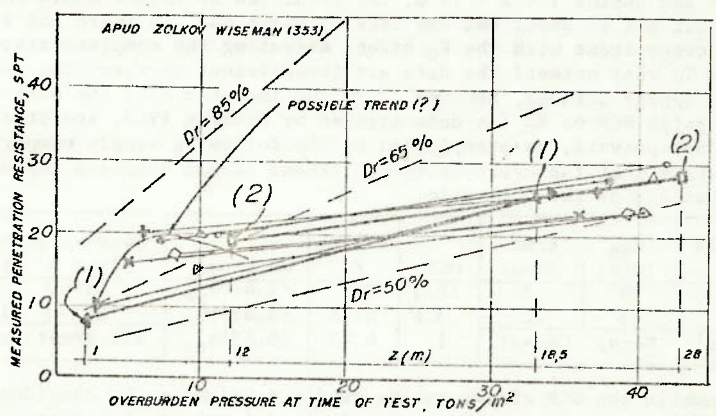

Fig. 18 Tentative interpretation of Zolkov-Wiseman (353) data as due to depth effects rather than precompressions (?).

This subject, connected with settlements of footings on sands, will be further discussed under item 6.1.

3.5. EFFECT OF PRECOMPRESSION ON SPT IN SANDS.

It has frequently been observed through the execution of borings before and after an excavation or fill that a change in the SPT occurs at each elevation affected by the stress change. Zolkov and Wiseman, 1965 (353) noted however, that upon removal of some 30 t/m^2 (15.5 m) of overburden pressure "the reduction in penetration resistance ... is not as large as would be expected considering that the relative density has remained essentially constant". Thus it has become accepted as logical that precompression of a sand should be responsible for higher SPT values, because of higher residual horizontal stresses after removing the surcharge, as the SPT appears to be affected not so much by the overburden effective σ_{oe} pressure as by the complete state of stress $\sigma m = \sigma_{oe}\ (1 + 2K_o)/3$.

Needless to say, in properly evaluating such changes under field conditions every care must be taken to exclude extraneous effects: for instance, overburden effective stresses must be carefully computed taking due account of capillary tension, in the case of basement or footing excavations appropriate stress-release influence charts must be used in lieu of direct overburden pressure changes, and so on.

It cannot escape notice, therefore, that the impact-energy transmission effect should not be neglected. The Zolkov-Wiseman sands are apparently similar to the U.S.B.R. ones (see Fig. 10). It may be noted thus, in Fig. 18, that the initial SPT values determined for $33 < \sigma_{oe} < 43\ t/m^2$ and $18 < z < 28$ m indicated values of RD of about 56 - 50%: and at the same elevations after the general excavation, for $2 < \sigma_{oe} < 12\ t/m^2$ and depths $1 < z < 11$ m, the predicted RD values would range from about 60% to about 80% and back to about 65%. Is there not a depth effect concomitant with the K_o effect affecting the complete state of stress? To what extent? The data are insufficient to clear the question. It does appear strange, however, that the importance of the overconsolidation ratio OCR on K_o (as demonstrated by Hendron 1963, and others) should not prevail, as exemplified by the following simple comparison which shows that the overcompression effect should decrease rapidly with depth because of the OCR ratio.

Fig. 18 Case	σ_{oe} Init.	t/m^2 Final	OCR	estim. K_o	σm final	Approx. depth, m Init.	Final
1	33	2	16.5	1.6	$(1.4)\ \sigma_{oe}$	18	1
2	43	12	3.6	0.75	$(0.83)\sigma_{oe}$	28	11
Initial	Norm.	Consol.	1	0.45	$(0.63)\sigma_{oe}$	all-great depth	

Meanwhile the OCR effect on the sand's ϕ value may be considered null if the SPT penetration occurs under drained ϕ values, but there may be significant effects if undrained strength behaviour is at play [cf. Bishop and Eldin, 1953 (23)].

It appears important at the present state of the art of SPT interpretation to employ several concomitant indications to evaluate the intervening effects. On the one hand, in the analysis of subsoil profiles advantage must be taken of preconsolidated clay layers (desiccation excluded) to evaluate overburden precompressions of the sand strata too. On the other hand, the "systematic" effects affecting SPT in a homogeneous deep sand deposit may be interpreted as follows, by judicious use of the (SPT, σ) and (SPT, z) plots vs. ϕ and vs. RD as shown in Figs. 15, 16, 17.

In Fig. 17, the steepness of the slope $\Delta STP/\Delta\sigma$ of the lower "linear" stretch is a good indication of ϕ except for depth effects: this effect generally leads to flatter slopes, i.e. greater ΔSPT, for a given RD or ϕ line. Greatly flatter than usual (U.S.B.R. average sand) lines can only indicate a material of such grain and grading as to give bigger ϕ values. A precompression effect tends to accentuate the upper "hump" by shifting it to the right, while simultaneously steepening the lower linear stretch somewhat (clockwise rotation). Moreover a nearly vertical lower stretch can only mean a very low ϕ value: incidentally, unless there are equipment and procedure differences, it seems difficult to accept for an average $55 \lesssim RD \lesssim 75\%$ of Basore and Boitano, 1969 (4a), as shown in Fig. 17, a nearly vertical lower stretch; no wonder that the authors found the U.S.B.R. corrections quite inapplicable because the latter at RD $\approx$ 65% correspond to $\phi \approx 37^{o}$ and the subvertical lower stretch of $\Delta SPT/\Delta z$ corresponds to $\phi \lesssim 25^{o}$.

3.6. ATTEMPT AT ESTABLISHING THE FUNDAMENTAL GEOTECHNICAL PARAMETERS AT PLAY IN THE PHENOMENON OF DYNAMIC SPOON PENETRATION IN DRY SANDS.

In a manner analogous to that undertaken under item 2.8 for saturated insensitive clays, it was decided to begin by postulating a pseudoanalytic treatment for the phenomenon of dynamic spoon penetration in pure dry sands. Once again, it need not be repeated that such an analysis should not be interpreted as implying any attempt to underestimate or to despise the real complexity of the problems: it is merely advanced, despite the tremendous present limitations, with a view to fostering the thesis advocated, that the chaotic and sterile discussion on the uses of the SPT in sands will only begin to be settled as one establishes a first-approximation analysis to understand the relationships between the more significant intervening parameters.

Once again, the first thought is that the penetration phenomenon in sands is also absolutely continuous, as already suggested by earlier investigations (181, 182) and despite several references to the contrary. Fig. 19 summarizes some typical results of meticulous observations of penetration per blow conducted by Geotecnica S.A. at the writer's request: the continuity of the penetration curves can hardly be questioned. It was found, however, that the type of penetration curve does not follow the function used (Eq. 10) successfully for penetration in clay.

The apparently successful treatment for clay based on the equation of useful work in penetrating against point and skin friction forces, suggested adopting an analogous treatment in the hope of profiting of the Gibbs and Holtz data. Some adaptations are indispensable because of insufficient information. It is known that in sands the point resistance only reaches a constant ultimate value [e.g. Vesic, 1967 (325) Fig. 13] after several "diameters" (variable with RD) of penetration. Lacking any means of estimating the analogous behaviour of the hollow sampler, it was decided to adopt as two limiting possibilities: (a) the assumption of a constant ultimate point resistance from the start of penetration: (b) a point resistance varying linearly from zero to the above constant value.

The Vesic results were reported in terms of the adjacent vertical

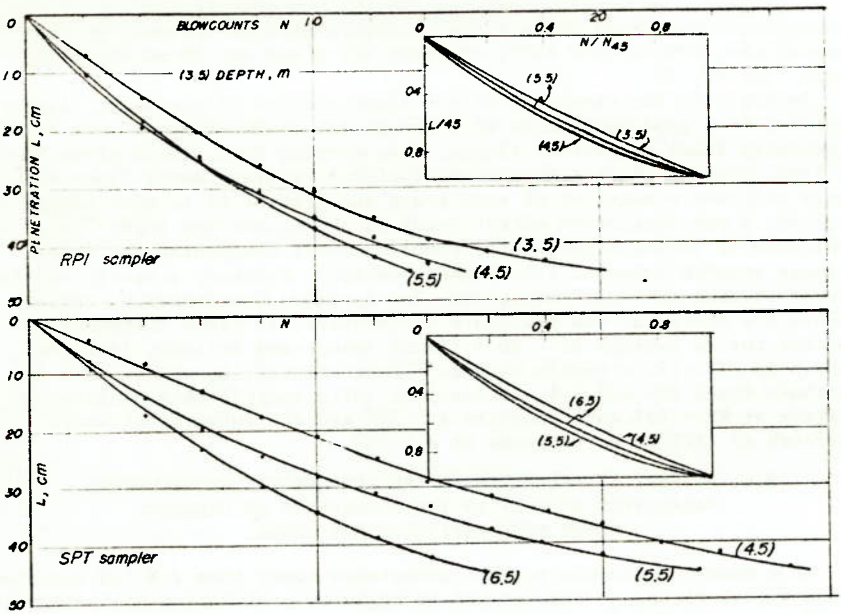

Fig. 19 Continuity of penetration in pure fine sand, Santos.

confining pressure q_f, which is an interesting pragmatic approach because of data available for solution of foundation engineering problems. Moreover, from Vesic's Fig. 63 one may estimate the overall friction f_o over the "pile length" in terms of the point resistance q_o: if we assume that the friction developed on the inside and outside walls of the sampler is αf_o, we obtain the friction already developed at a penetration ℓ as

$$\alpha q_o/m\ (\phi)\ [\pi\ (D_o + D_i)]\ \ell, \text{ where } q_o/f_o = m\ (\phi) \quad \ldots\ldots\ldots\ldots\ldots \quad (28)$$

The point resistance at the same penetration ℓ will be equated to $q_o\ \pi/4\ (D_o^2 - D_i^2)\ \ell/45$ for case (b) above, and automatically the factor $\ell/45$ will be unity for case (a). Thus the work in penetrating $d\ell$ can be formulated, and, if we assume a shape factor F, to equate $q_o = q\ N_q\ F$, the useful work of penetration, for case (b) will be =

$$q\ N_q\ F\ [\tfrac{\pi}{4}(D_o^2 - D_i^2)\ell^2/2x45 + \frac{\alpha\pi}{2m\ (\phi)}\ (D_o + D_i)\ell^2]_{15}^{45} \quad \ldots\ldots\ldots\ldots \quad (29)$$

For the two hypotheses we therefore obtain the work equation

$$\beta(SPT) = 318\ q\ N_q\ F + \frac{2.42 \times 10^4}{m\ (\phi)}\ q\ N_q\ F + \text{losses} \quad \ldots\ldots\ldots\ldots \quad (30)$$

or

$$\beta(SPT) = 212\ q\ N_q\ F + \frac{2.42 \times 10^4}{m\ (\phi)}\ q\ N_q\ F + \text{losses} \quad \ldots\ldots\ldots\ldots \quad (31)$$

The N_q and m (ϕ) parameters were extracted from Vesic's (325) experimental data (Figs. 41 and 63) based on the reasoning that it might still be preferable to work with closed-cycle correlations using reasonably significant, even though nominal, parameters: therefore the triaxial ϕ_{max} values extracted from RD values appear indicated, resulting in the following tabulation of approximate values:

ϕ^o	31	32	33	35	36.5	30	40	42.5	44
Nq	15.5	20	26.5	39	60	100	180	320	580
m (ϕ)	45.5	47.5	53	64	74	90	104	136	178

Interestingly m (ϕ) would vary from 3 N_q to about 1/3 N_q.

The principal problems lie in estimating the β value applicable for the U.S.B.R. tests, and the intervening losses.

The losses apparently should bear some relation to SPT: lacking any information to orient on this point one could assume the losses as $\lambda(SPT)^n$ allowing n to define itself by suitable regressions and simultaneous equations. In the present case, however, since the experimental program was not especially "delineated as a sampling program" for furnishing suitable simultaneous equations (as would occur if different spoons, etc., were used, to force the separate contributions to stand out), it transpires that the introduction of any more than two or three unknowns already leads to unwieldy and extremely sensitive equations (analogous to the problem of intersection of nearly parallel straight lines), with values easily changing from positive to negative, and thereby losing any physical meaning. Therefore two conditions were assumed, n = 1 and n = 2 just to illustrate the problems. If n = 1, the "losses" become incorporated into the left-hand term of Eq. (30) or (31) with a new $\beta_c = \beta - \lambda$, and thus the number of unknowns for the simultaneous equations is reduced to two. With n = 2 we retain 3 unknowns in the statistical regressions, which may be developed in the form.

$$(SPT) = a + b\ F + c\ (\alpha F) + \lambda (SPT)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (32)$$

with values on the left-hand side presumably known, and on the right-hand side, values of b, c, and SPT known. The unknowns F, αF, and λ are determinable by three sets of simultaneous equations dictated by the statistical regressions for best fit, and the value of "a" results from a linear combination of the other three.

A great number of groups of the U.S.B.R. data were investigated, as illustrated by the results on the following table, and all regressions yield very good representations of the experimental points. But it can be seen that the attempt at pseudotheoretical analysis fails, because of the extreme sensitivity of the equations. It is not known to what extent some of this sensitivity may be physical, pertaining to the SPT test itself: the sensitivity due to the very poor delineation is obvious. The physical meaning of α leads one to expect values close to 1. The factors F and λ depend on the estimate of β, which incorporates the energy factors discussed in item 2.2. Assuming a β factor of the order of 1.500 to 3.200 kg.cm, the values of the shape factor βF should be expected to reach physically meaningful values of the order 0.5 to 3. The only reason for including this discussion is to indicate a possibility and a need to be investigated, particularly as regards the eventual physical sensitivity of the tests (in sand at shallow depths).

Case (a) Constant point resist.	I Losses = λ(SPT) (affecting value of β) in Eq. (29) or (30)		II Losses = λ(SPT)^2		
	F	α F	F	α F	λ
(1) $26^o < \phi < 32^o$	-0.00012	0.00072 α ?	0.00010	0.00019 α = 1.9	0.05413
(2) $32.4 < \phi < 36.7$	0.00186	0.00338 α = 1.8	-0.01978	0.00732 α ?	0.56605
(3) $38 < \phi < 41$	0.00036	0.00011 α = 0.31	0.00040	-0.00006 α ?	0.03771
(4) $\phi > 41$	0.00026	0.00038 α = 1.5	-0.00366	0.00541 α ?	0.10486
(5) $\sigma < 0.1$ kg/cm^2	-0.00132	0.00698 α ?	-0.00344	-0.00556 α = 1.6	0.18251
(6) $\sigma > 1.5$ kg/cm^2	-0.00007	0.00055 α ?	0.00021	0.00032 α = 1.5	0.00572
(7) General	0.0032	-0.0029 α ?	-0.00366	0.00541 α ?	0.10486
(8) General, point resist. varying	0.0048	-0.0029 α ?	-0.00549	0.00541 α ?	0.10486

Tabulated values must be multiplied by β or β_c of the order of 2.500 to 3.200 kg.cm.

3.7. APPLICABILITY OF A CRITICAL VOID RATIO CORRECTION TO SPT VALUES IN SUBMERGED, SATURATED, VERY FINE OR SILTY SANDS.

It has been satisfactorily confirmed (262.5) that there is no noticeable effect of submergence and saturation on SPT values of fine to corase sands, and gravels. On the other hand, the very first published indications (300) on the use of SPT blowcounts furnished "the following rule, which represents the present state of our experience: if the number of blows SPT is greater than 15, it should be assumed that the density of the soil is equal to that of a sand for which the number of blows is equal to 15 + 0.5(N - 15). Pending more reliable information, this rule should be adhered to ...". The rule lumps together the presumed effect of critical void ratio on the SPT blowcount, and the recommendations for allowable bearing pressure for shallow footings based on settlement considerations: from the shallow footing context one must assume that the SPT = 15 should correspond to depths of about 1 to 3 m. It is obviously convenient to separate the two problems, one analytic, the other synthetic (which will be discussed later, in connection with estimation of footing settlements): on the analytic side the correction is directly associated with the undrained strength of cohesionless soils of low permeability, as, "If the void ratio of the soils is higher than the critical void ratio, the resistance against penetration of the sampling spoon is smaller than that of a more permeable soil of equal relative density. If the void ratio is below the critical value, the reverse is true. The value of SPT corresponding to the critical void ratio seems to be about 15".

Conceptually it does seem strange that in the face of the many very significant first-order interferences ("errors") that SPT values may suffer in different soils and conditions, the latter single condition should have been selected for "correction": moreover, conceptually it may also appear strange that even for the unique case selected only the correction of reducing high SPT values (conservative) should be recommended, and not also the contrary correction of increasing the low SPT values vitiated by liquefaction. The writer feels that it was conceptually wrong to single out a correction, leaving the impression of it being the only one, when many other factors of equal moment were left to oblivion or to personal (hopefully judicious) interpretation: moreover, as will be seen easily, the selection of a single blowcount SPT = 15 (irrespective of depth, overburden pressure, precompression state of stress, etc.) to represent the boundary for application of a discontinuous correction for all possible situations represents another unfruitful orientation. Finally, it need hardly be mentioned that the critical void ratio of sands depends on many factors, at least one of which, the applied pressures, which has been recognized since the earliest studies, varies directly with depth of overburden, thus necessarily implying an eventual correction variable with depth.

There is really not enough experimental evidence to give any orientation on this count. The Gibbs and Holtz (86) investigation in the tank showed very much lower SPT for the saturated fine sand than for the air-dry sand, with zero blowcounts up to 60% RD, and no cases of increased SPT up to the maximum densities tested (≃ 86% RD): moreover, even the coarse sand showed a reduction, though of the order of 25%; but the test conditions in the tank can be considered quite special on several counts. Meanwhile in the field test of the fine sand the SPT were somewhat higher below water level than indicated by the average RD (increased gradually from 73 to 86%) and the air-dry or moist-sand correlations. Schultze and Melzer 1965 (263) have been cited in the same connection, with their conclusion for a medium and coarse sand, that "under the groundwater level the sounding resistances decreased": however, their tests were also in a tank, using dynamic cones, and employing penetrations of 20 or 30 cm only. It must be recalled that Seed and Lee, 1967 (268) have well pointed out "the importance, in testing saturated sands, of correct laboratory representation, not only for the initial effective stress condition on a soil element, but also of the initial pore water pressure in soil elements which may be subjected to undrained loading conditions".

Bazaraa, 1967 (5) investigated borings from 11 different sites and compared average SPT values 3 ft. above water table vs. 3 ft. below water table: the SPT values ranged between 5 and 20, but it is not known at what overburdens and depths, and it is assumed that relative densities are the same above and below the water level, and that the silty sand is unsaturated above and saturated below. Thereupon the conclusion is that there is a general "tendency towards increasing the SPT-value upon submergence", with an average ratio of 1.7, and an effect "more pronounced for loose than for dense sands". However, it is rightly concluded that "this effect cannot be reasonably represented by any simple formula".

It is of interest to study the results reported by Drodz, 1965 (66) both for saturated sands, and for unsaturated sands (showing a peak of SPT at about 83% saturation): would not the fine sands possess considerable capillary moisture up to 3 ft. above, and some trapped air up to 3 ft. below water level, especially if there are seasonal fluctuations?

Apparently Terzaghi and Peck presently espouse the same concept formulated by the writer, because in the Second Edition, 1967, the correction suggested in the first edition is dropped. Reference is made to the fact that Hayashi, 1966 (107) and Kishida, 1966 (145) who use the SPT to evaluate the sand strata that liquefied or not under the Niigata earthquake concluded that the SPT_{cr} was of the order of 15 to 20 at a depth of about 11-12 m ($\sigma_{oe} \approx 10$ t/m^2), and that the accepted criterion thus became SPT < 15: it must be noted, however, in the same boring profiles (145) that other values of SPT_{cr} are recorded as 32 and 33 at depths of 17 to 18 m ($\sigma_{oe} \approx 16$ t/m^2). Malcev, 1969 (169) indicates the need of the correction $SPT_c = 15 + 0.5$ (SPT - 15) for "clays and silts rather than in sands", but no evidence is offered. Seed and Idriss, 1967 (267) also discuss liquefaction in terms of an SPT value: irrespective of the numerical value, it seems to the writer that the $\Delta SPT/\Delta z$ trend should be accepted as less subject to discussion, because a very steep, near-vertical SPT vs. depth diagram can only be indicative of a very low, near zero (< 20^o) nominal ϕ value under the dynamic penetration of the spoon.

Theoretically it should be reasoned that SPT really involves a measure of the pseudo-dynamic undrained shear strength of soils, at one extreme fairly well represented by a partially remoulded c value of plastic clay, and at the other end represented by a judiciously estimated [cf. Seed and Lee, 1967 (268)] undrained-drainage ϕ value depending on the estimated pore volume change tendencies and on the pore-air compression possibilities [cf. Bishop and Eldin, 1954 (23)] and the pore-water drainage possibilities. It has long been known that critical void ratios, dilatancy, pore-air compression, etc., are factors that depend on so many first-order parameters, that it is quite unacceptable to attempt formulation of a "correction". If there is any hint of an unusually $\Delta SPT/\Delta z$, $\Delta\sigma$ trend (liquefaction of loose sands, low ϕ) or an unusually flat $\Delta SPT/\Delta z$, $\Delta\sigma$ trend (dilatant very dense, high ϕ) other field tests such as the static cone penetrometer should be used to clear the doubt.

4. COMPARISONS OF SPT WITH R_p OF THE STATIC CONE PENETROMETER.

4.1. SATURATED CLAYS

It has been generally established [cf. de Mello, 1969 (184)] that the static cone penetrometer constitutes a good field test for determining the in-situ undrained shear strength of saturated clays, irrespective of their sensitivities: it appeared that $R_p = 10$ c would represent a satisfactory correlation, although at first sight it would seem that experimental results without corrections lead to a range $10\ c \lesssim R_p \lesssim 30\ c$. The more recent information on the same problem include an investigation by Kerisel and Adam, 1969 (142a), the discussion by Schmertmann and Kerisel (184), and an investigation by Ladanyi and Eden, 1969 (156) extending earlier work on the case of sensitive clays. The latter investigation concluded that N_{cp} in the relation $R_p = c\ N_{cp}$ decreases with increased sensitivity, and may reach 5.50 to 8.00; moreover, the rate of penetration effect is of the order of 7.5% per 10-fold increase of rate, and therefore should be taken into account [cf. Thomas, 1965 (301)]. Meanwhile Schmertmann demonstrates that N_{cp} should depend on the ratio E/c in accordance with the cavity expansion theory, and suggests that since two unknowns are at play, it is necessary to resort to a second concomitant observation, for which he admits applying the local friction measurement by the Begemann, 1965 (17) jacket cone, at least in insensitive clays.

Since it has hitherto been assumed that the SPT correlations should also be of the type SPT = nc, it could well be anticipated that a reference survey would only furnish correlations of the type R_p = J (SPT). Indeed Fig. 20, summarizes most of the data hitherto quoted, whereby 1.5 SPT $\lesssim R_p \lesssim$ 7.5 SPT, it being implied in one instance [Malcev, 1964 (169)] that the higher ratios R_p = (4-6) SPT correspond to CL clays of low plasticity, while the lower ratios R_p = (2-4) SPT correspond to CH - OH clays of medium to high plasticity.

There is no reason for the R_p value to suffer from extraneous depth effects in clays (besides the usual effect of depth, or consolidation, on c), since the constant point resistance value is reached within much less than one diameter of penetration of the cone, and tests are usually conducted with penetration of 10 cm for the 3.5 cm diam. cone. Incidentally, a good way of investigating the nature of a clay stratum (homogeneity, consolidation, etc.) is by regressions of R_p = f (c, z): one need only be guarded against the very frequent case encountered in commercially conducted investigations wherein the c values determined suffer from a depth effect because of sampling disturbances that tend to increase with depth. Meanwhile, it has been shown that the SPT suffers from a pronounced effect of sensitivity, as well as an effect of depth. It is obvious, therefore, that the correlations illustrated on Fig. 20, and currently quoted, are questionable and incomplete. Moreover there seems to be no reason to expect an effect of plasticity, and if one does exist, possibly through frequent connection of higher sensitivities with more plastic clays, it should be preferably associated with the sensitivity as the real intervening parameter.

There are very few case histories involving deep deposits of clays which may permit confirming the above postulated trends. One published case involves the Mexico city clay [Correa Rachó et al. 1964 (532)] which is doubtless an exaggerated case. The data on Santos clays is also revealing, although they suffer from imprecisions because of the ruggedness of the SPT for sensitive clays of 1 to 4 blows.

4.2. PURE SANDS.

The use of a correlation between SPT and R_p in pure sands was first suggested by Meyerhof, 1956 (190) and because of the advancement of the European school in connection with uses of the static cone penetrometers has since found greater and greater interest. Once again, however, the correlations offered have systematically admitted a linear function passing through the origin, R_p = J (SPT), and have induced a reasoning whereby the value of J should increase gradually from plastic to nonplastic materials, and within the latter, increase with grainsize. A tabulation such as Sutherland's, 1963 (289) more or less reproduced in many other references (e.g. 169) summarizes the present beliefs, Fig. 21.

After all the qualifications that have transpired within the present state-of-the-art report, it should be unnecessary to reemphasize the need to scrutinize the sources of information somewhat more carefully,

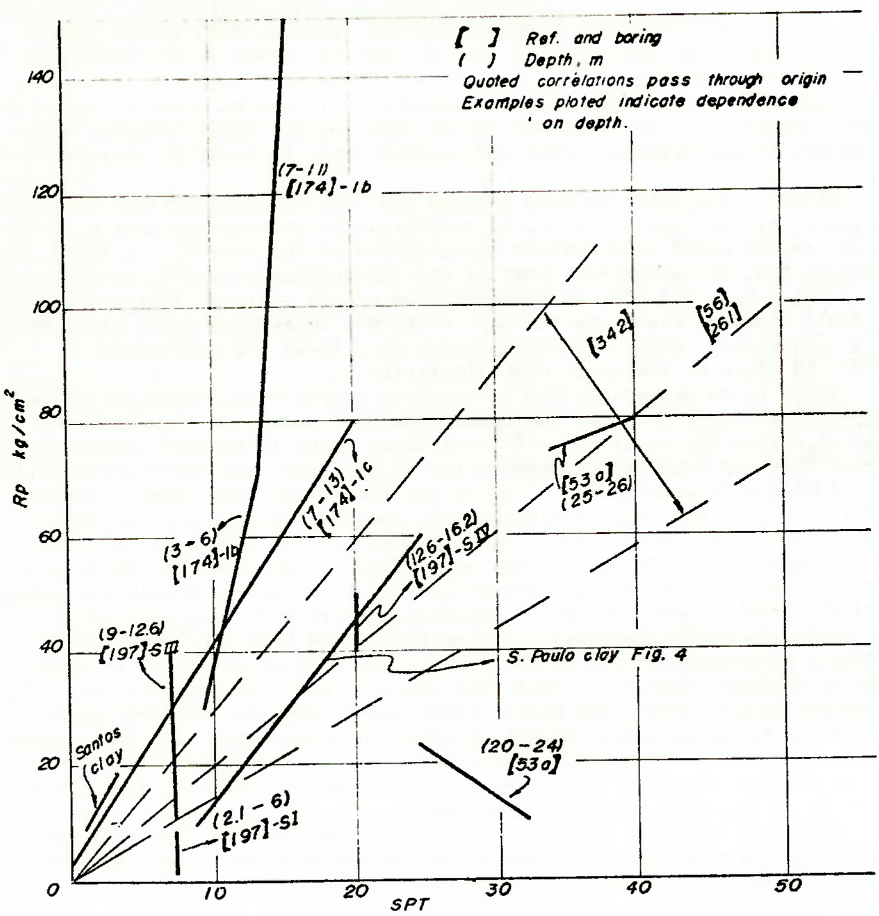

Fig. 20 Correlations R_p = f(SPT) for saturated clays.

within a minimum pseudo-theoretical context. Even without introducing the indispensable context of depth effects etc., the following qualifications on the tabulation may be noted as an illustration. The first Glasgow case refers to a submerged sandy silt with only two SPT values below 15 and all the others incorporating the SPT correction discussed under item 3.7.

The Meyerhof (190) data include three values from São Paulo that were on the IPT sampler, not yet correlated to SPT, but probably about one-half of the latter; also, two London points that are much higher may hint at the stress-release effect of the large-diameter holes (item 2.6). Both the Rodin and the Meigh and Nixon cases have already been mentioned as comprising the United Kingdom lower-SPT effect due to large diameter holes. The Costa Nunes (56) data are for rods of 3 kg/m and it can be

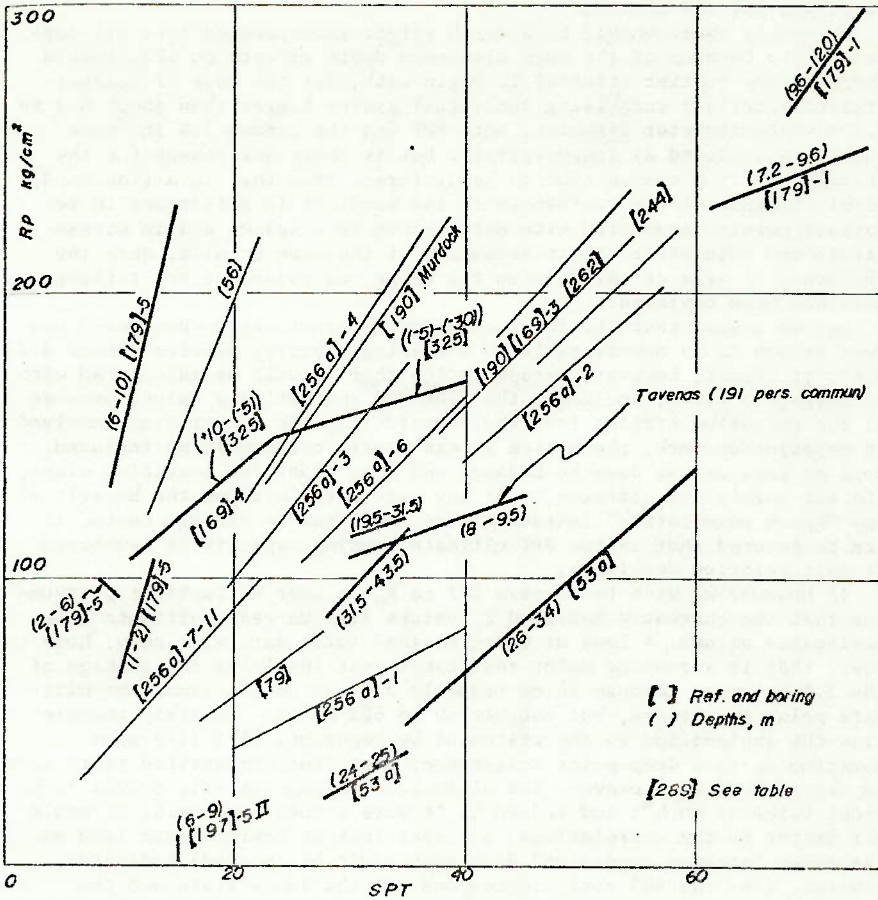

Fig. 21 Correlations $R_p = f(SPT)$ for pure sands.

Apud Sutherland (289)

Sources	Description of soil	J
Glasgow	Sandy silt	2.5
Meyerhof (190)	Sand and gravelly sand	3.6
Schultze & Knau-senberger (261)	Non-cohesive	3.6
Meigh & Nixon (179)	Fine sand and silty fine sand	4
Rodin, Site 2 (244)	Fine to medium sand	4.8
Rodin, Site 3 (244)	Sand with some gravel	8
Glasgow	Fine to medium sand	10
Costa Nunes (56)	Sand	10
Meigh & Nixon (179)	Gravelly sand	8 -18
Meigh & Nixon (179)	Sandy gravel	12 -16

seen that the quoted J = 10 is not the average for the available points on sands, but the maximum.

Obviously there should be a depth effect incorporated into all correlations, because of the much discussed depth effects on SPT. Should there be any further effects? To begin with, for the case of coarser-grained materials comprising individual grains bigger than about 0.2 to 0.1 the penetrometer diameter, both SPT and the common 1.4 in. cone should be excluded as inappropriate: but is there any reason for the correlation in a coarse sand to be different from that in a fine sand, merely because of the coarseness of the sand? Or is difference in behaviour merely associated with differences in ϕ values and in stress-strain and volumetric strain behaviour of the sand or silt, once the influence of size of particle on the shape and volume of the failing mass has been obviated?

Let us assume that the SPT spoon-sampler penetration phenomenon has been proven to be associatable to a bearing capacity problem (items 3.2, 3.6): it cannot, however, escape notice that it will be associated with an average ϕ value, including the peak and the ultimate value, because of the excessive strains involved. Considering the integration involved in penetration work, the entire stress-strain curve must be included, more or less as was done by Ladanyi and Eden (156) for sensitive clays, and not merely the ultimate ϕ. At any rate, even without the benefit of any "spoon penetration" investigations analogous to sinkage tests, it can be assumed that in the SPT ultimate bearing capacity is developed at most relative densities.

If however we wish to compare SPT to R_p we must be implicitly assuming that the currently measured R_p values also do reach ultimate base resistance values. A look at Vesic's, 1967 (325) data will show, however, that in a routine point resistance test involving the sinkage of the 3.5 cm point through 10 cm (roughly 3 D) we do not reach the ultimate point resistance, but roughly 40 to 60% of it. Possibly therein lies the explanation to the statement by Begemann, 1965 (17) upon equating R_p to a deep-point resistance, that "For comparative tests made in the laboratory, however, the ultimate bearing capacity proves to be about twice as much": and indeed if it were a constant ratio, it would not matter to the correlations. A closer look at Vesic's data (and at the other "sinkage condition" data that could be located) indicates, however, that the 40% ratio corresponds to the loose state and the 60-65% ratio to the dense state: therefore, within a given sand the R_p should rise more rapidly with increased density and ϕ than the use of Begemann's average correction factor of 2 would indicate. Thus, even for a given depth, the correlation $R_p = f$ (SPT) for different RDs should not pass through the origin except by sheer coincidence: the tendency might be for R_p to yield somewhat more rapid rates of change (with depth) of average values than the SPT. Incidentally, on first appearance Schmertmann's 1970 (256a) data concerning both depth and denseness effects on the ratio R_p/SPT appear to be contrary to the trends postulated in the present report, but this impression would require closer scrutiny: the important first step is to recognize that there must be a significant interference of both these parameters.

It must be emphasized herein that de Beer's, 1963 (10) technique of evaluating ϕ on the basis of rates of change d R_p/dz does not suffer from the qualification raised, on the contrary, probably seeded the thought: neither does any interpretation of R_p values in connection with

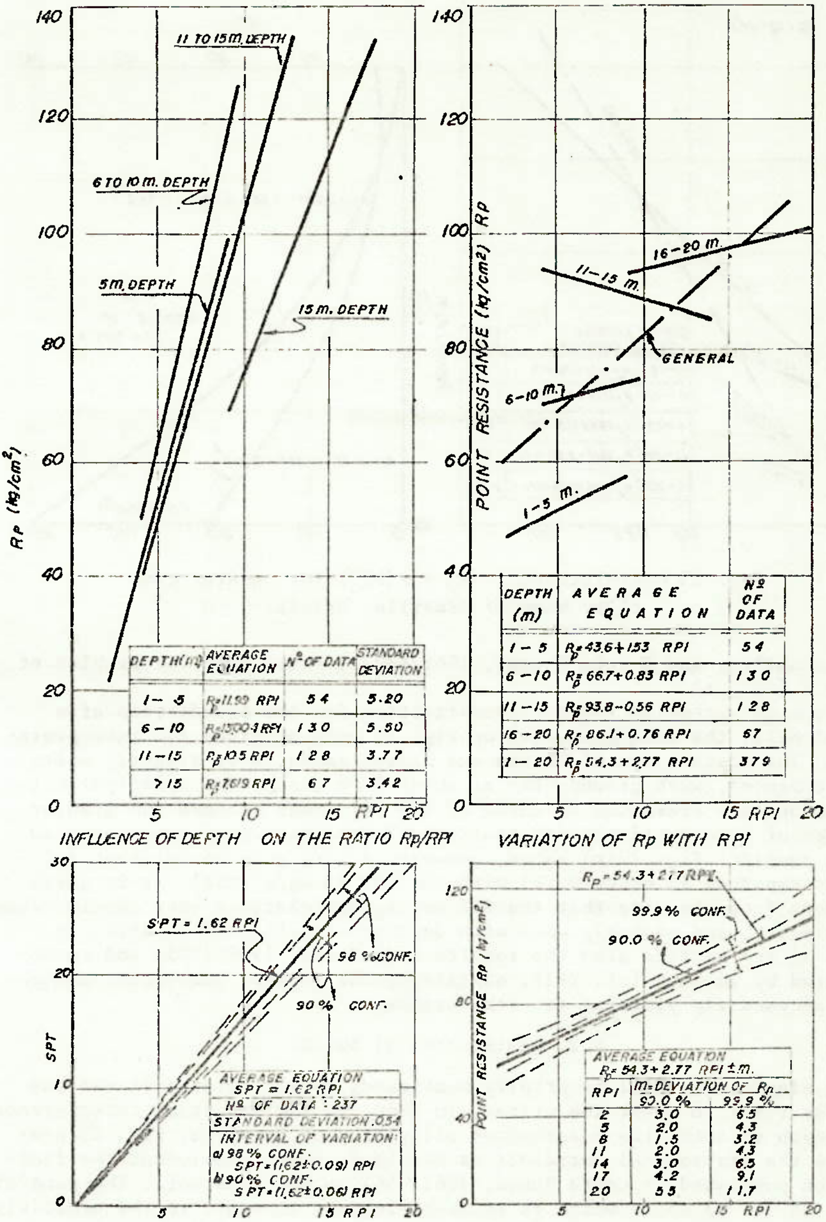

Fig. 22 Correlations on R_p = f(SPT) for sands, Usiminas, Bras.

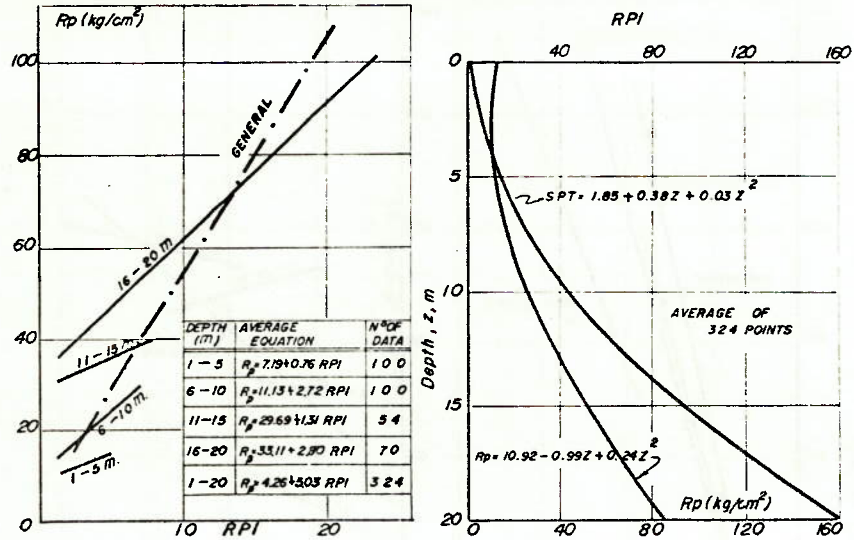

Fig. 23 Correlations on R_p = f(SPT) for unsaturated silty clay of Brasilia, Brasil.

deformability and E [cf. Thomas, 1968 (302)] enter into the problem of R_p = f (SPT, z).

From an extensive routine investigation for the foundations of a steel mill, the data presented on Fig. 22 were obtained for interpretation. The deposit was of coarse and medium sands, very slightly silty and cemented, with groundwater at about 15 m depth. The principal correlations are presented in terms of the RPI index because the greater number of data available for statistical analyses were connected with that sampler: Fig. 22(c) shows, however, a very good correlation (already reported in other work) with the SPT (3 kg/m rods). It is quite evident from the data that the SPT vs. R_p correlations vary considerably with depth, and probably also with deneness (reflected in SPT). In Fig. 21 are plotted also the results from Vesic, 1967 (325) and communicated by Tavenas (cf. 291), sustaining the general contention though not necessarily yielding the same trends.

4.3. GENERAL (c, ϕ) SOILS.

A single case will be briefly mentioned, merely because it was one of the first to alert the writer, in 1960, to the important interference of depth effects: the unsaturated silty clay of Brasilia. Fig. 23 presents the statistical correlations obtained, which complement the indication published by Costa Nunes, 1961 (56) on the same soil. The general case of (c, ϕ) soils which is of most frequent interest to the practising engineer has never been dealt with: it is hoped that the necessary investigations on correlations will bear in mind the need for more than two simultaneous equations to determine the minimum number of intervening parameters of first-order significance.

It may be noted that the phenomenon of spoon penetration in such soils is also continuous, permitting the visualization of pseudotheoretical analyses of the parameters in analogy to the cases already roughly suggested. Using data supplied by Rodio on scores of borings through a residual unsaturated sandy clay and clayey sand (weathered gneiss) in which the SPT penetration ($3 < SPT < 144$) were recorded with blowcounts at every 5 of the 45 cm, it was found that the penetration curves were entirely similar to those of Fig. 5: moreover, the statistical regression for the factor A defining the form of the curve yielded the result $A = 0.6875 - 0.997 \times 10^{-4}(SPT)$, very similar to Eq. 11, perhaps in part by coincidence.

5. COMPRESSIBILITY AS INDICATED BY SPT.

The SPT has not yet been suggested, to the best of the writer's knowledge, as a field test for evaluating compressibility of soils, but the static penetrometer R_p is widely quoted as correlated to "compressibility characteristics of soils" [e.g. 87, 180, 251, 256a, 302, among the most recent references], and it cannot escape notice that the correlations of $R_p = f$ (SPT) are supposed to be applicable in all soils: therefore, by analogy, the SPT should indeed be available for the desired evaluation.

In the case of clays the general impression is that the SPT has no connection with compressibility parameters, and, particularly, agreeing with Fletcher (79), that the "most important limitation of the SPT in cohesive soils is that it does not reflect preconsolidation". Quite on the contrary, however, if the SPT can give something regarding clay compressibility, it should be an indication of the preconsolidation, since it does give an indication of the cohesion of the clay: of course, in evaluating the preconsolidation pressure in different clays the plasticity of the clay should interfere, because of the correlations, to first-order approximation, of the type $c/p \approx 0.115 + 0.00343\ I_p$ (33). Even in as sensitive a clay as the Santos clay, the unusual, as yet geologically unexplained occurrence of a highly preconsolidated lens of the same clay within the area [cf. Teixeira and Geotecnica, 1959 (293)] was initially revealed by RPI blowcounts of the order of 6 to 9 vs. the usual consistent values of roughly 2 at the same depths. Recently in the interpretative studies of the insufficient laboratory data on undisturbed samples of the São Paulo clays, with a view towards interpolation and extrapolation for the design studies on the Subway line, very suitable statistical regressions were worked out for undrained Mohr envelopes, as shown in Fig. 24, taking into account the collateral information on preconsolidation pressures: the approximate appropriate consolidated-undrained angles for the two "straight lines" were established from triaxial tests. The unquestionably normally consolidated cases permitted establishing the "virgin lines" of CU $(\sigma_1 - \sigma_3)$ vs. SPT: next for points apparently overconsolidated, the overconsolidation ratio OCR was estimated and linear regressions were "forced" to be parallel to the flatter strength envelope, confirming approximately the reasonable preconsolidation pressures. Thus, in short, insofar as SPT may be connected with undrained shear strength of clays, and the latter are connected with consolidation pressures, doubtless one of the important inferences that may be extracted from SPT values, with some account taken of the probable $c/p = f\ (I_p)$, is on the state of preconsolidation.

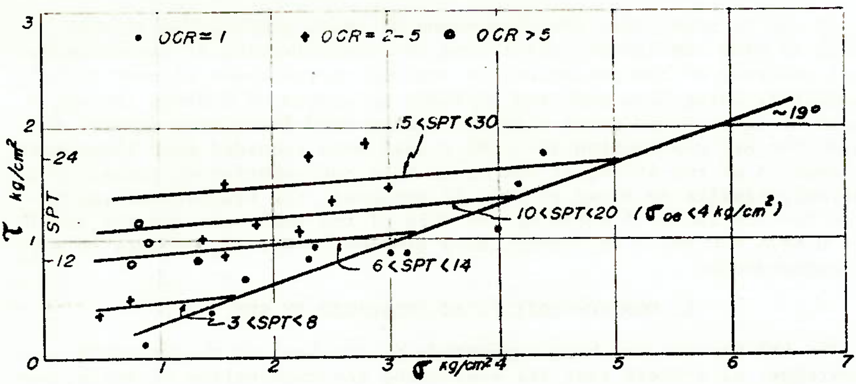

Fig. 24 Illustration of attempt at establishing undrained strength envelopes, S. Paulo clays, from (SPT, σ_{oe}) data. PROMON'S FILES - SUBWAY.

Regarding compressibility of soils in a general way it is necessary to consider first the case of sands, for which the Buisman approach employing the static cone penetrometer R_p has been in long and relatively successful use, as very meticulously and extensively discussed in recent papers [e.g. Thomas, 1968 (302), Schmertmann, 1970 (256a)]. For direct use of SPT on fine sands Chaplin, 1963 (47) recommends the correlation SPT = $1/44\ Es^{4/3}$ for the Es = 1/mv modulus of compressibility. More generally one employs R_p, the use of which for sands arose on the basis of elastic theory (even if somewhat extrapolated from its normal use), with the concomitant assurance that time effects would be negligible in distinguishing between "immediate (elastic) compressions" and slow-drainage volume compressibility: in the case of clay, however, serious questions must be raised if the conception presumes to lump together the immediate settlements (probably without exaggerated first-order scatter because of Skempton's demonstration of a reasonably constant E/c ratio for clays), and primary and secondary compressions that should be related to clay plasticity and should introduce considerable scatter depending on the proportions of porewater dissipation that could participate in the cone penetration phenomenon.

Thus, on the one hand it must be reported that in very recent developments on the use of cone penetrometers, Gielly et al., 1969 (87), and Meigh and Corbett, 1969 (180), have extended the Buisman analysis of the modulus of compressibility $m_v = 1/\alpha R_p$ as connected to R_p values, asserting the applicability of the same form of correlation for plastic clays, both normally consolidated and preconsolidated: moreover, it is anticipated that because of the SPT vs. R_p correlations, the approach will be extended into the bin and bane of SPT uses. The writer considers the very use of R_p subject to questions in such cases, and, despite anticipation of overall "correlations" [cf. Nishida, 1956 (216)] warns against eventual attempts at correlating SPT with clay compressibility: since the SPT retrieves a sample it is unnecessary to relinquish, for normally consolidated clays, the probably closer correlations available for Cc = f (W_L), and so on.

6. SYNTHETIC CORRELATIONS FOR APPLICATION TO ENGINEERING PROBLEMS.

The part of this state-of-the-art report which was the principal source of enthusiasm for tackling it (the universalization of design recommendations), has been patently nipped in the bud because of the analytic treatments disclosing that not merely do all synthetic correlations thus far published err in forcing linearized correlations between complex lumped parameters (cf. item 1), but also the SPT lumped parameter embodies very significant consistent errors besides the well-acknowledged random errors that could be abstracted by statistical averages. Nevertheless, considering the fundamental purpose of the Standard Penetration Test, as a tool for preliminary foundation design indications, the following discussion is indispensable.

6.1. SHALLOW FOUNDATIONS.

6.1.1. CLAYS.

As was discussed in de Mello, 1969 (184), the Terzaghi and Peck, 1948 (300) and most contemporary recommendations on allowable bearing pressures q_a for shallow footings on clays are based on the premise that bearing capacity is the controlling factor: therefore it must seem reasonable to establish q_a as an essentially direct function of c, and thus of SPT. However, obviously two important factors interfere directly in such a prescription, so that the settlement (and differential settlement) criterion should often be the controlling one: one is the increasing size of footings, for higher and heavier buildings, since settlements in clays are essentially proportional to the diameter of the footing [Osterberg 1947, suggests the linear relation of log p vs. log ρ/D]; the other is the varying plasticity of clays, whereby the c/p ratio should vary and correspondingly the connection between SPT and preconsolidation, while the concomitant variation of compressibility (or, more appropriately, recompression) characteristics of the clay vary with plasticity in somewhat different manners. Indeed, as Terzaghi and Peck, 1967 (300) state "The allowable soil pressures q_a corresponding to a factor of safety of 3 against a bearing-capacity failure are almost always less than the preconsolidation pressure", because if we accept the approximate c/p ratio of Eq. (33) and the bearing capacity as equivalent to about 6c, only clays with $I_p \gtrsim 112$ would be excluded from the statement: however, the recompression settlements on a very plastic clay may be so much larger than those on a clay of low plasticity that the subsequent statement "As a consequence, the differential settlements of footing foundations on such clays seldom exceed those of adequately designed footing foundations on sand" presents an unexplainable generalization.

The fact is that there should be some interest in evaluating the "direct" settlements under individual footings on clays, which should include both "immediate, elastic" settlements and, hopefully, consolidation settlements, if the load tests are sufficiently long-duration: in other words, the coefficient of subgrade reactions ks = q/ρ (cf. 298) from drained load tests on preconsolidated highly plastic clays should be of interest to foundation design. A general correlation will obviously be found, whereby stiffer clays yield higher SPT values and higher k_s values: but what is the confidence-band in a direct single-parameter correlation between an undrained shear strength phenomenon in clays and a drained distortion-compressibility phenomenon?

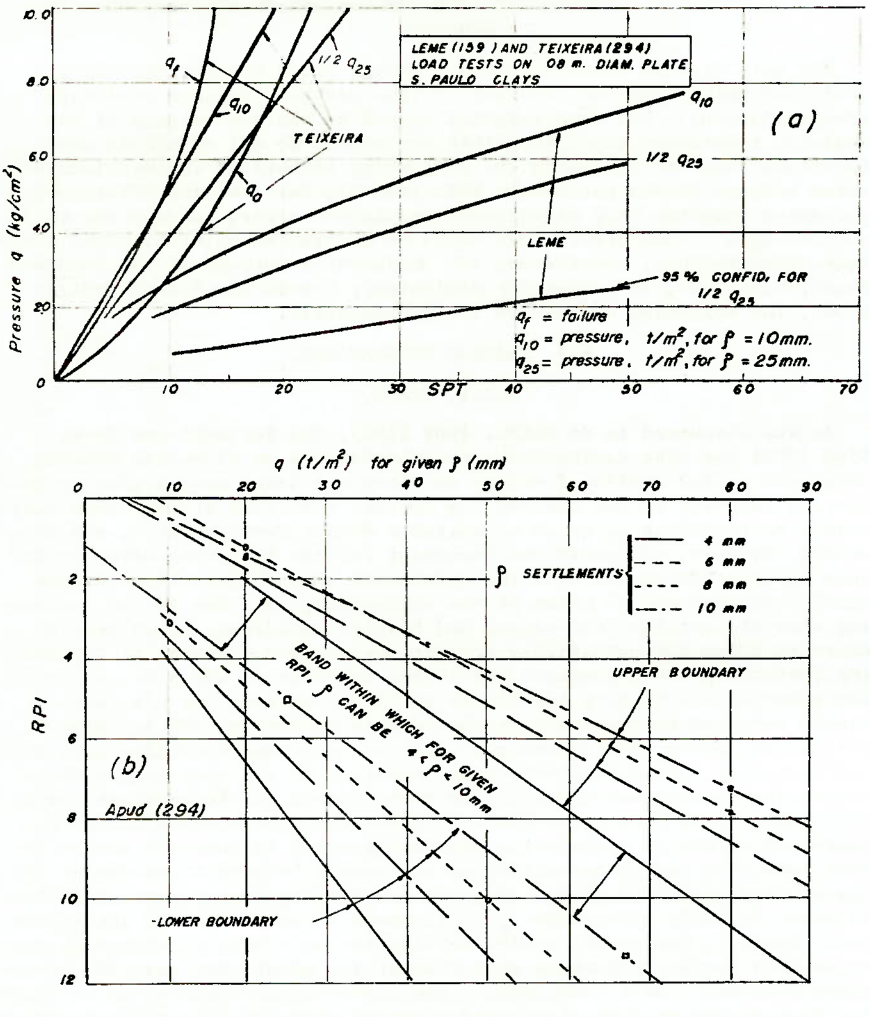

Fig. 25 Correlations of plate load test settlements vs. spoon penetration resistances, S. Paulo clays.

Fig. 25 presents the results of two statistical analyses of 80 cm diam. plate load tests on S. Paulo clays [cf. Leme, 1953 (162), Teixeira, 1966 (294)]. The discrepancies in average trends, Fig. 25 (a), as well as the wide margin of error in consequent settlement estimates, Fig. 25 (b) are quite apparent. Surely the correlations can be considerably improved by judicious introduction of additional first-order parameters, but it is the writer's contention that to begin with the "statistics at random" should be reoriented with regard to the trends of theoretically anticipated behavior.

6.1.2. SANDS.

As is widely stated, "settlement, rather than bearing capacity (stability) criteria, usually exert the design control when the least width of a foundation over sand exceeds 3 ft. to 4 ft." [Schmertmann, 1970 (2562)]: thus, one of the most important uses to which the SPT has been put, particularly in North American foundation design practice, is for orienting with respect to allowable bearing pressures q_a on sands. The Terzaghi and Peck 1948 (300) design recommendations for a q_a for a settlement not greater than 1 in., traditionally presented in the form of design charts, can be transformed into the equation

$$(q_a)_B = \frac{SPT - 3}{10} \left(\frac{40}{21}\right)^2 \left(\frac{B + 1}{2B}\right)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (34)$$

where q_a is in tsf, and the width of the footing B is in ft., for $B \gtrsim 4$ ft.: this equation embodies four important distinct steps into which it must conceptually be subdivided.

(a) The first is the evaluation of the coefficient of subgrade reaction k_s for a standard-size plate load test as a function of denseness, and thus presumably of SPT: the second comprises the evaluation of settlements of footings of different dimensions by extrapolation of the above indication, and assuming homogeneous material; the third comprises evaluation of probable differential settlements that might occur in a given foundation on footings, due to all likely causes (differences of loading and heterogeneity of subsoil); and finally the fourth incorporates the decision as to what are boundaries of tolerance of the total and differential settlements.

Firstly one would conclude that $k_s \approx 4.4$ (SPT - 3) tonsft^{-3} $\approx$ 1.4 (SPT - 3) t/m^2 per cm for B = 1' .... (35)

Such a presumed correlation could be established directly, from 1 ft. x 1 ft. plate load tests and from closely correlated SPT blowcounts for the sands at the level of the pressure bulb under the plate. If the essentially "elastic" coefficient of subgrade reaction k_s can be assumed to be directly relatable to SPT, there is no need to seek any more than a direct statistical correlation. Bazaraa, 1967 (5) analysed information from Peck's and Ireland's files, together with published information: he applied some corrections to the SPT values (overburden and critical void ratio) and plotted the values of average k_s vs. SPT, concluding, in a nutshell, that linear relationships such as k_s = SPT/36 and SPT/24 (tonsft^{-3}) represent "lower limiting conditions" but that such "equations cannot be used to give any reasonable correlation between the results of standard penetration tests and the results of standard plate-loading tests" because predicted settlements can be as high as 10 to 20 times the actual settlement.

The same data without "corrections", are herein analysed by linear statistical regression, with the results shown on Fig. 26. The regression had to be worked out in the form log (q/ρ) = a + b log SPT in order to improve the statistical analysis of dispersions, which in the form q/ρ = a + b (SPT) for obvious physical reasons fanned out from the origin (0,0). As anticipated in the face of Terzaghi-Peck's intention of being conservative, and in the light of published discussions (e.g. 192, 57, etc.) it can be seen that the Terzaghi-Peck prescriptions lie close to the lower 95% confidence limits, being particularly conservative for loose sands (only about 5% of <u>individual cases</u> might fall below the estimate, whereas in medium dense and dense sands the values lie close to the lower 95% confidence limit of the average).

The main problem lies, moreover in the very wide bands that represent roughly the 95% confidence limits, both of the average, and, particularly, of the individual data that are of direct concern to an engineer in the face of a specific problem. It must be emphasized in all fairness that although one is conscious of a number of factors that should be introduced to "correct" both the SPT values quoted, and the corresponding plate load test initial linear coefficients of subgrade reaction, the data were used directly, without any such corrections, in large part because most often the papers do not submit sufficient information to permit applying valid and defensible corrections.

The above discussion may be more conveniently followed through the table:

SPT		95% conf. of indiv. case	95% of aver.	Aver.	95% of aver.	95% conf. of indiv. case	Terzaghi-Peck (Eq. 35)
10	k_s values in t/m^2 per cm	12.1	35.2	42.6	51.4	165	10.0
		Zolkov-Wiseman (353) for 4 < SPT < 14			50	100	
		S. Paulo clayey sands, plate diam. 0.8 m for 6.5 ≳ SPT ≲ 13	29.4	50.6	72.5		
30		18.9	45.7	66.4	96.4	233	38.2
50	—	23.0	62.7	81.7	107	291	66.5

At any rate, it is the writer's wish to emphasize herein that one strictly independent step comprises the eventual correlation of the k_s value for sands, with the corresponding SPT blowcounts. What are the theoretical and practical perspectives with regard to such a correlation?

On the one hand it has already been pointed out that the driving-energy factors should make the SPT at very shallow depth a particularly rough test subject to very significant variations. If the latter thesis is sustained and confirmed, the intervening factors being reasonably understood, there should be no reason why a special corrective procedure could not be developed and recommended for the first few meters of routine foundation exploration. It seems to the writer that one of the principal purposes of a state-of-the-art effort might be to suggest the fields in which one might profitably stop flogging dead horses. The statistical regression (Eq. 25) showed that even for controlled laboratory tests the margin of error on blowcounts was very wide. The same conclusion may be reached by an impartial examination of the data offered by Zolkov and Wiseman [e.g. 1965 (353)] to the effect that Israel dune sands bear out the U.S.B.R. data: the average curve of SPT vs. dry density (0 psi) really permits a band of almost ± 80% error of the SPT values. That is, indeed, too high an error for a homogeneous material and condition.

Theoretically, how close could one expect a dynamic large-strain failure phenomenon, connected with penetration energy and therefore with the area under the stress-strain curve, to be related to an elastic strain phenomenon at stresses far below the failure condition?

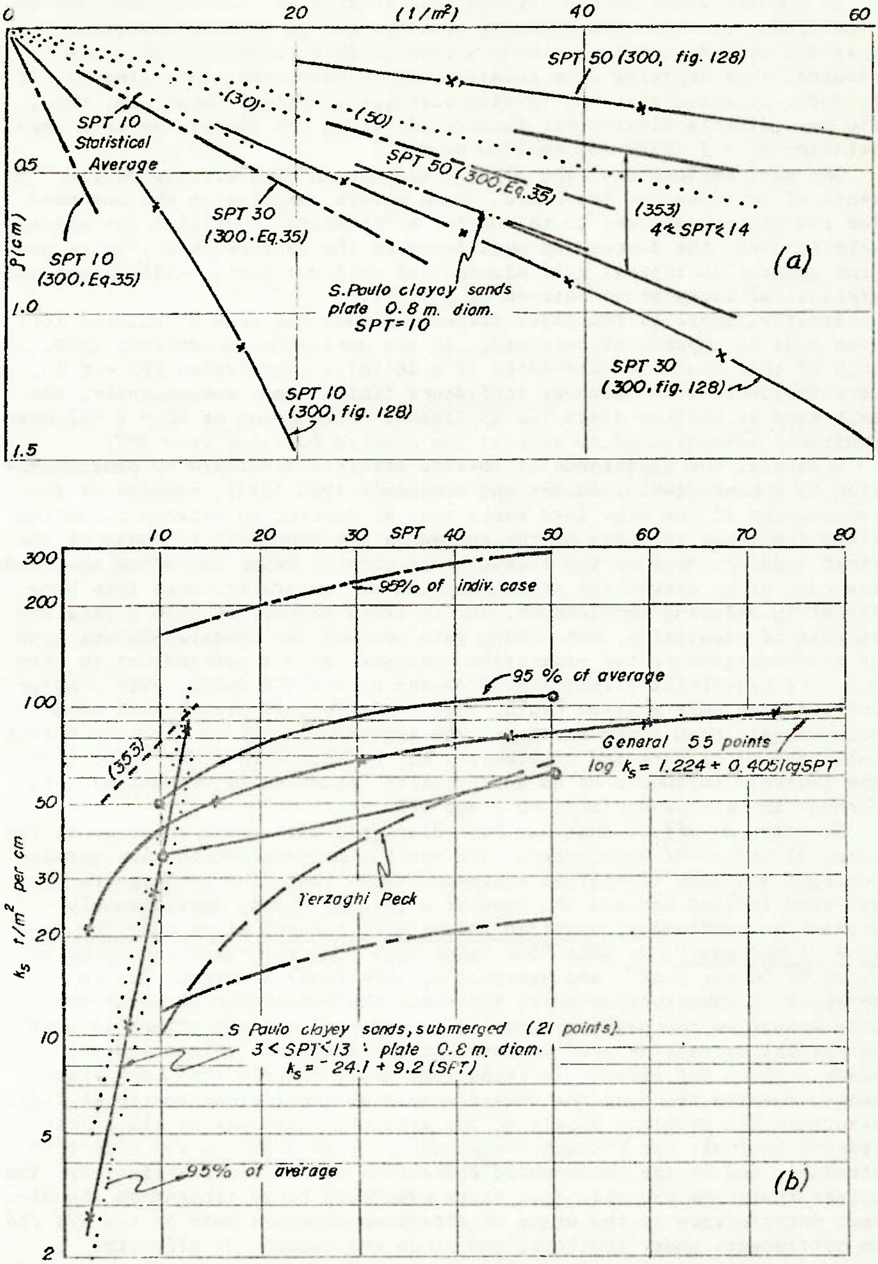

Fig. 26 Plate load test linear settlements in sands.

It appears accepted and logical that footing settlements on sands are principally related with Relative Density: and it is well understood that the strains involved embody a considerable proportion of shear strains, thus implying some relationship of phenomena: and, finally, it is today accepted that the in-situ stresses interfere. What are, then, the recognizable first-order factors affecting the dispersion of a correlation $k_s = f$ (SPT) for shallow depth?

One must exclude from the present discussion such effects as the depth of embedment of the plate, which merely comprise an obvious need for the plate load test to be run for a "standard" condition (or adjustable to one): the discussion must focus on the factors which, lying beyond control in typical well-planned and well-run tests, will affect any statistical correlation between k_s and SPT.

Firstly, there is the major dispersion that has been postulated (cf. item 3.3) as capable of occurring, in the estimation of RD from (STP, σ). Much of this could be eliminated if a definite correlation SPT = f (σ, ϕ) were improved, with narrower confidence limits, and, subsequently, for each sand at shallow depth the applicable correlation of RD = f (ϕ) were routinely investigated to extract the desired RD value from SPT.

Secondly, the importance of in-situ stresses developed by precompression is uncontestable. Zolkov and Wiseman's 1965 (353), results of recompression of the very load tests are, of course, an extreme condition since the shear stresses of the reloading are identical to those of the first loading, most of the irreversible strains being therefore absorbed: the case of an overburden precompression is, naturally, much less beneficial in reducing settlements, but it still exists, as also exists the benefit of excavation, but taking into account the Leonards-Baladi type of Boussinesq-corrected excavation analysis. As was pointed out in item 3.6, the beneficial effect should depend on the OCR value, with a major influence at very shallow depth, that is, affecting plate tests more pronouncedly than large footings. The important point is that the direct influence on strains and settlements may be much more significant than the indirect influence on RD and porosity (practically unchanged) and, through the stress history, on ϕ and SPT.

Thirdly, an effect that has been discussed with great interest is the eventual effect of submergence. The various pronouncements vary between Terzaghi and Peck's original statement (300) that "the submergence of the sand located beneath the base of a footing should approximately double the settlement, provided the base is located at or near the surface of the sand", so that "the value (q_a) from the chart should be reduced by 50 per cent", and Meyerhof's, 1956 (190) statement "since submergence of cohesionless soils increases the settlement by about the same amount as the bearing capacity is reduced" the net effect is that no special correction need be introduced because SPT values would already reflect the effect. It transpires quite clearly that both statements, despite reaching the widest spread of conclusions (ratio of 1:2), associate the problems purely to the effective stresses at play: which appears logical; but Terzaghi-Peck had not linked SPT to the effective stresses, and so the recommended correction would result obligatory. The writer therefore proposes that since everybody today recognizes the direct interference of the state of effective stresses both on the SPT and on settlements under footings, and since the changes of effective stresses under footings (affecting both concomitantly) rarely occur under such simplified conditions (e.g. the proviso underlined above) as to permit establishing a simple general ratio of "correction", that any correction be applied only in connection with changes of effective stresses, be they from submergence or from any other cause.

It cannot escape notice, because of all the foregoing discussions, that it appear much more logical to the writer [in agreement with Thomas, 1968 (302) and Schmertmann, 1970 (265a)] to employ for settlement estimates the Buisman-type approach and the static cone penetrometer R_p results, which cover a range of stress-strain behaviour much more akin to that of a coefficient of subgrade reaction of plates than the SPT large-strain penetration. Conceptually the SPT is being used somewhat outside of its appropriate field, as regards settlements of footings. At any rate, if use of SPT is to be made, upon accepting the R_p to be really more appropriate, it transpires that careful account must be taken of the problems raised regarding the relation $R_p = f$ (SPT) in item 4.

b) The second independent problem implicit in Terzaghi-Peck's design prescriptions for q_a (which were indeed not intended as a method for estimating the actual settlement of a footing) concerns the effect of dimension of footing on settlements, for which the relation was proposed

$\rho/\rho_1 = (2B/B + 1)^2$ for $B \geqslant 4$ ft., B in feet .... (36)

Much work has been done since [cf. Bond, 1961 (29), Bjerrum and Eggestad, 1963 (25), et al.] and it seems strange that the single simplified relation is still retained, irrespective of denseness of the sand, etc.. By reporting to the Buisman-type approach based on R_p one sees immediately that in a purely homogeneous sand the extrapolation of settlements from one size to another depends not merely on the R_p value under the small plate, but also on the $\Delta R_p/\Delta z$, trend, which may vary considerably: perhaps it had been assumed that R_p and $\Delta R_p/\Delta z$ values in homogeneous sands vary simultaneously, but the examination of numerous published profiles shows that there is a sufficient margin of independent variation, to justify some of the "errors" attributed to the "SPT methods of estimating settlements". The problem is in no way connected with the SPT. The more advanced comprehension of the effect of size on settlements should be incorporated automatically into revising this part of the Terzaghi-Peck prescription.

c) The third independent part of the Terzaghi-Peck prescription comprises the estimate that a differential settlement of 0.75 in. between adjacent columns will not be exceeded "if the pressure is selected such that the largest footing would settle 1 in. even if it rested on the most compressible part of the sand deposit" (300): that is, implicitly, the decision to choose q_a as "equal to the pressure that will cause the largest footing to settle 1 in." requiring, moreover, that "if the tests in different drill holes furnish different values of SPT, the lowest value should be used for estimating the allowable soil pressure". Regarding differential settlements of buildings on sands, Skempton and MacDonald, 1956, and Bjerrum 1963, have concluded that "the differential settlements of buildings on sand are of the same order of magnitude as the maximum settlements", where Terzaghi-Peck had admitted 75% as the maximum and "normally it is much smaller". However, in discussing the matter no qualifications were made with regard to the heterogeneities implicit in a sand's geology, nothing is mentioned with respect to the effects of adjacent footings in mutually altering each other's "independent" settlements, nor is credit given for any effect of structural rigidity in redistributing loads prior to the threshold of cracking [cf. de Mello, 1969 (184)].

Since the prescription is for the largest "independent" footing resting on the lowest SPT profile, it cannot help but be conservative for differential settlements: but that is a design decision that is in no way intrinsically connected with the use of SPT in sands.

d) Finally, the fourth really independent part of the Terzaghi-Peck prescription comprises the assumption that a differential settlement of 0.75 in. between adjacent columns of "typical buildings" constitutes the limit of tolerable differential settlements, because of the threshold of damage. On this count also, there have been some theoretical and practical developments that could suggest revisions of the early prescription. At any rate, it constitutes a vast and important separate field of discussion.

In short, of the four components of the Terzaghi-Peck prescription, the only one that is truly connected with SPT is the evaluation of k_s: two are connected with extrapolation from model to realistic prototype: and one is connected with a design decision. A number of studies very important to the practice of the profession [Meyerhof, 1956 (190) and 1965 (192), Bazaraa, 1967 (5) and D'Appolonia et al., 1968 (57)] have compared "Terzaghi-Peck predicted settlements" with observed settlements, concluding in general that the overall, lumped, prescription is conservative. The writer would suggest as much more profitable to the fostering of judicious application of SPT to restrict the discussion to the possible correlation of k_s vs. SPT. Thereupon the techniques of forecasting the behaviours of different footings (and their consequent differential behaviour) may, in special cases, be considerably developed, in the same way as is done with R_p values (e.g. 15). And, finally, the design decision on maximum tolerable differential settlements is, for individual, special cases (e.g. 57) a matter for judicious decision by the designer. The overall prescriptions should probably be applicable only to routine cases that would rarely merit publication.

6.2. PREDETERMINING PILE LENGTHS.

Intuitively one would expect that one of the principal uses of SPT profiles should be connected with the important design decisions on pile foundations. And, indeed, it is known that the practice of the profession is full of rule-of-thumb indications, based on SPT values, that are used by design offices and contracting firms. Surprisingly little is to be found on the subject, however, in published form.

The principal design decisions are generally two: firstly, how deep can a given pile be driven against the dynamic resistance to penetration; secondly, how deep must a pile be driven so that it will guarantee the desired static allowable load (due consideration of bearing capacity, settlements, group action, etc. having already been incorporated into the selection of this allowable load).

On the first problem the only reference encountered was by Holtz, 1961 (352 discussion) who claims "the correlation of spoon penetration resistance and pile driving resistance is based on the fact that both the spoon and the pile are driven by impact methods". For timber piles "P", the approximate bearing resistance in pounds" is estimated by the empirical formula $P = K_1\ (d_1\ SPT_1) + K_2\ (d_2\ SPT_2) + K_n\ (d_n\ SPT_n)$ ...(37) where SPT_1, SPT_2 are the blowcounts in corresponding increments of depth d_1, d_2 etc. and K_1, K_2 etc. are factors in pounds per linear foot of pile depending on the type of soil. Examples of K values are: Plastic clay = 100; low plasticity saturated clays to silts = 50 - 65; loess of low water content = 200; loess of high water content = 100.

Fletcher 1965 (79) states "it is in connection with pile foundations in granular soils that the SPT probably finds its widest application", yet "with so many types of piles and an array of pile hammers, no simple way can be found". "For the wide range of cast-in-place pile foundations in which piles are to be driven to reach a bearing stratum of granular soils, experience suggests that for 30 ton piles, the blows recorded by the SPT in the bearing stratum should range from 20 to 30; for 50 ton piles, 60 to 70; and for 75 ton piles, 90 or more". Other local rules-of-thumb, considering a summation of SPT blows at every meter of "compatible soil strata" to account for lateral friction, plus a separate weighted interference of the SPT blowcount near the point, have been in use by the writer for precast concrete piles (20 to 70 tons) and for Franki piles, and appear to be quite successful. Yet, Mohr, 1962 (201) emphatically states that "he can predetermine pile lengths neither within a reasonable degree of accuracy, nor with consistency", "and he strongly questions whether anyone else can do so".

The fact that nothing has been published on the all-important problem really signifies that nobody is yet satisfied that his "solution" can be pseudo-theoretically backed. The writer feels that one of the principal reasons for the pseudotheoretical analyses attempted under items 2.8, 3.2 and 3.6, will be the opening of doors for pseudotheoretical solutions to the problems of piles.

Regarding the forecasting of the static behaviour of the pile, Meyerhof, 1956 (190) formulated recommendations, basing much of the deduction on the assumed general correlations $R_p = 4$ SPT. Indeed, profiting of the fact that solutions to the problem on the basis of point (R_p) and skin friction measurements by static cone penetrometers have been much more developed, the general trend seems to be to relate SPT to R_p and thereupon to resort to applications of the cone-penetrometer interpretations. Correlations between SPT and local friction [cf. Begemann, 1969 (18)] will be necessary. For the case of driven displacement piles, Meyerhof 1959 (191) gives indications on the compaction effects in sands, so that suitable corrections may be applied.

6.3. PIERS AND CAISSONS.

No special mention is encountered with respect to design recommendations for piers and caissons in clays and in sands, on the basis of SPT values. Presumably intrinsic shear strength and deformability parameters of the soils must be evaluated, for subsequent use in theoretically-oriented design formulae; or an alternate approach once again comprises resorting to comparison with R_p values. Once again the basic need is to improve the "analytic correlations" on SPT.

6.4. MISCELLANEOUS USES.

Principal use of the SPT, besides evaluation of density [e.g. Clevenger, 1956, (50)], has been for the determination of the liquefaction potential of sands under seismic effects. As regards evaluation of density or compaction achieved by various means such as explosives, compaction piles, vibroflotation, etc. the value of the SPT test seems proven, since average values are compared: once again the writer would recommend greater use of the $\Delta SPT/\Delta z$ trends. As regards liquefaction potential, although successful application has been reported (107, 145, 223, 267), it is felt that the problem is subject to the wide dispersions of $\phi = f$ (RD) relations, plus all the heterogeneities connected with liquefaction conditions of sands, and, separately, all the errors connected with the

SPT = f (σ, z) relation: moreover, it appears that the case is not such as will permit application of the narrower confidence bands of average values. The writer therefore does not expect that the successful application hitherto reported will be sustained upon wider applications or more meticulous study.

7. CONCLUSIONS

7.1. The SPT seems to be most profitably interpreted as related to in-situ undrained strength parameters of the soil.

7.2. The depth effects apparently interfere more than has been presumed.

7.3. For many cogent reasons it appears necessary to employ more than one spoon concomitantly, the dimensions of spoons and the specific test conditions requiring judicious selection, after better study of the energy-penetration phenomenon, in order to foster deduction of simultaneous equations most appropriate for computation of separate contributing parameters.

7.4. For determination of RD of sands it appears necessary to work from ϕ = f (SPT, σ) to RD = f (ϕ).

7.5. Apparently the SPT is least appropriate for shallow depths, and possibly in particular for sands at shallow depths.

7.6. For evaluation of settlements of footings it is apparently recommendable to resort to correlations of R_p = f (SPT), not passing through the origin, and not independent of depth, and thereupon to employ computation techniques based on the Buisman approach.

Acknowledgements.
The number of colleagues from abroad who contributed with data, and with personal opinions on the problem, comprise too long a list to be inserted herein, but the writer wishes to convey herewith his special thanks for the kind response. For closer help in all the statistical and computational work grateful appreciation is due to Professors Evelyna B. S. Silveira and A. Silveira, and to Luiz Guilherme de Mello and Luiz Augusto B. Barreto.

APPENDIX I

1. Demonstration of significance of parameter A on value of. SPT (item 2.8, Eq. 11).

no. of data = 246

Source of variation	Sum of squares	Degrees of freedom	Variance
Due to regression	31.66	1	$S_r^2 = 31.66$
On the regression	113.29	244	$S^2 = 0.464$
Total	144.95	245	

$S_r^2/S^2 = 68.23 \gg 3.84 = F_p$ for 95% confidence, from tables, Fisher and Yates $\therefore$ A is v. signif. dependent on SPT.

2. Demonstration of significance of dependence of regressions SPT = f (q_u, z) for S. Paulo clays (Fig. 6) and SPT = f (c_{vane}, z) for Santos clays (item 2.9.6), on z.

(a) S. Paulo clay. No. of data = 14.

Source of variation	Sum of squares	Degrees of freedom	Variance
Due to linear regression	368.45	1	
Improvement on depth	81.04	1	81.04
Due to depth	449.49	2	
On the regression on q_u and z	34.81	9	3.86

$\frac{81.04}{3.86} = 20.9 > 4.60 = F_p$ for 95% confidence .'. dependence on z is significant.

(b) Santos clay No. of data = 92

Due to linear regression	12.104	1	
Improvement on depth	6.151	1	6.151
Due to depth	18.255	2	
On the regression of c_{vane} and z	59.57	87	0.685

$\frac{6.151}{0.685} = 8.979 > 3.84 = F_p$ for 95% confidence .'. dependence on z is significant.

3. Summary of statistical analyses for coarse and fine sand [U.S.B.R. (86)], and for both jointly.

Sand	No. of data	Values determined B	D	c	F	Standard Deviation	Significance
Fine	24	-4.0	0.022	6.8	5.7	3.4	Significant
Coarse	16	3.6	0.012	6.6	8.9	5.3	Significant
Both	40	4.0	0.015	2.4	8.7	5.8	Significant

Analyses of variance, for determination of significance of the regressions.

Fine Sand:

Source of variation	Sum of squares	Degrees of freedom	Variance
Due to regression	2644	3	$S_r^2 = 888$
On the regression	259	20	$S^2 = 12.9$
Total	2923	23	

$S_r^2/S^2 = 68.8 >> 3.10$.'. regression is highly significant (for 95% confidence, Fisher and Yates)

Coarse sand:

Due to regression	3568	3	1189
On the regression	337	12	28.08
Total	3905	15	

$S_r^2/S^2 = 42.3 >> 3.49$.'. regression is highly significant

Both sands jointly:

Due to regression	5582	3	1860
On the regression	1248	36	34.66
Total	6830	39	

$S_r^2/S^2 = 23.02 >> 2.60$.'. regression is highly significant

APPENDIX II - REFERENCES

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