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VICTOR F. B. DE MELLO

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RAPPORTS SUR L'ETAT ACTUEL DES CONAISSANCES

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FONDATIONS DE STRUCTURES SUR ARGILES

by Victor F. B. de Mello D. Sc. Prof.
University of São Paulo, Brasil.

1. INTRODUCTION

It is the express intent of any state-of-the-art report, to review current knowledge on the subject, and to attempt to prognosticate and recommend the most necessary and productive lines of endeavour in order to solve existing problems, and to foster effective development within the field, in the immediate future.

In the present case, however, the very nature of the subject proposed, very vast on the one hand, and very practical on the other, posed an initial problem of the necessity of choosing an approach and scope. The very concept of what knowledge is current, and what are the problems faced, is obviously most relative, depending on place, time and person. It has seemed that the most frequent key-note in similar state-of-the-art reports has been to stress the latest developments and what appear to be the next problems challenging the research ability of academic circles. The vastness of the subject, and its eminently practical facet have hinted at the possible interest in dedicating the present endeavour to quite a different key-note, which may be thus summarized: within the broad area of routine practice of the profession, and its consecrated faiths, what is really known, and how should recent advances affect such practices ?

Thus, as a preliminary, it must be emphasized that many important contributions may not be mentioned, partly through inevitable ignorance, partly due to a subjective estimate of the probability of such contributions entering effectively into the main stream of the practise of foundation engineering, in the near future.

The following thoughts may better elucidate the intents along which the present report was conceived, and for which the contributions are universal, whether acknowledged or not, the report itself being but an inevitably insufficient vehicle.

1.1 Aim

The principal aim of the treatment is to discuss the sequence and the levels of confidence in the more typical design decisions.

In so doing it is felt that the practising engineer will be accompanied through typical dilemmas and doubts that gradually shape that "judgment" that is the butt of well-meaning criticisms of the eager young theoretician.

Moreover, a fundamental aim is to discuss a present working synthesis, not merely as a basis for practical design, but also as a basis for constructive criticism and development, notwithstanding its temporary acceptance.

Engineering embodies an act of decision under uncertain premises, and not an expression of certainty. Theory, insofar as it is tied to the facts from which it evolved, belongs to the past: facts under observation constitute the present, which is never any better than our means of observing it: finally, the abstractions of analysis and synthesis, and retheorizing, are our hopeful projection into the future; and so the cycle proceeds.

Since the main tasks of Civil Engineering, and of Foundation Engineering, continue essentially the same, we need not look for new tasks, but merely look at the tasks newly.

1.2 Scope

The greatest difficulty is to choose a meaningful but limited scope.

It has been felt that one must firstly recognize that in examining clayey soils one really faces the truly general (c, ϕ) case.

Renegating the standard refuge of the dualism clay vs. sand, the first dilemma is where to fit the general case which is neither of these pure idealized cases. This is an inescapable starting point, and, without labouring under any delusion of seeking the unified field, it was concluded that the treatment should start from the general case.

Therefore, since to embrace too vast a task was clearly an unattainable proposition, the scope has been restricted by excluding, in compensation, the discussion of items that may be considered "particular cases" or interesting sideline issues.

1.3 Concept

Throughout the study of pertinent literature one of the problems that kept assailing the preparation of the report is the fact, inevitable and comprehensible, that every publication is destined to prove something right. On the other hand, the theoretical and testing advances within this field, qualifying and complementing earlier pronouncements, are so continuous and important, that the designer may easily fall into bewildered apathy. How can so many things be right at the same time, being nonetheless different ? One solution, unfortunately not uncommon, is to ignore all but "one" right solution, through an act of faith or subconscious emotions of sympathy. But this is clearly a short-lived illusion in a world of accelerated communications and development, within which the speed at which the levels of precision of observation of experimental facts and intervening factors have continuously improved, has been such as to expose any postulated laws to criticism and revision quicker than they penetrate into acceptance.

The answer is that many thoughts apparently contradictory, can be "right" simultaneously, within the degrees of precision and dispersion. Such a concept, that has been increasingly recognised during the past decade, constitutes a stabilizing thought within the fertile cauldron of discussion. The concept, and its lesson, should a fortiori be extended towards reexamination of older "established facts" which under calmer atmospheres were able to assume an aura of consecration under unsuspected and unchallenged conditionings, pertaining to the then current margin of errors, conscious or random.

1.4 Method

The greatest number of contributions have individually been concerned with the observation or establishment of single-parameter correlations, which appear, successively, as yet another factor to be accounted for. It is felt that a diametrically opposite approach may be less frustrating. One recognises that obviously any parameter of behaviour X is simultaneously a function of an infinite number of independent or interrelated parameters, a, b, c, ... n, i.e. X = f (a, b, c, ... n). If we adequately represent the field of interest of observation of parameter X, and collect sufficient data to represent it statistically, the average behaviour will be satisfactorily established irrespective of discoveries of temporarily second- and third-order effects. In a first stage, when only single parameter correlations are considered, the range of dispersion is very wide and confidence levels very low. But as hitherto second-order effects progressively come within the range of correlatable effects of a more refined level of observation, these bands will be narrowed. And so on successively.

Thus the task of digesting the vast produce of analytical research efforts must be faced with respect to decisions on the level of precision of our present-day capacity to observe, and capacity to apply the observed information within the design theories. And the practicing engineer's task is not one of discussing what additional factor may interfere, but rather, one of sifting out the factors that, at a given stage, must be relegated into the classification of second-order effects, buried under the protection of the Factor of Safety.

What seems incomprehensible is that twenty years of research and development should pass, with little or no suggestion of a reduction of F values earlier recommended.

An eminently statistical reasoning seems indicated. In order not to delve deeper into this method than our present knowledge justifies, it will be assumed that the simple statistics of averages, under normal distributions, may be applied. There are obvious cases, however, in soil mechanics, in which it would appear more appropriate to reason under the statistics of extremes.

1.5 Difficulties

Ironically, the frontiers of knowledge are really pushed forward by failures, disasters, and the testing of the thresholds of acceptance by the feared occasional exceeding thereof. Thus the field of foundations is inevitably retarded in its opportunities for progress by the fact that the practicing engineers try to stay as far from such threshold limits as economic factors will permit. Whereas Nature is not conscious of a reputation at risk and does not squander Factors of Safety on slopes, and therefore slope stability analyses may frequently be checked against failures, the same may not be claimed with respect to foundations. Thus, the academic efforts can be extended very far along purely theoretical lines, possibly aided by model tests, long before any de-facto failure in design will be available, at the misfortune of some well-meaning colleague, to push forward some of the thresholds limiting present design practices.

1.6 Recommended generalized approach

Within the report, the sequence followed has been as closely as possible connected with the sequence of decisions facing the foundation engineer, both chronological and of relative responsibility of risks involved. Thus, for instance, bearing capacity problems are evidently more important than settlement problems, and the levels of confidence of determination of applicable shear strength parameters precedes an appraisal of differences in bearing capacity formulae.

For the future progress in the field two basic recommendations are offered. Firstly, in any observation or test, one must keep complete records of all possible collateral data, whether its interference is suspected or not. Unsuspected factors may suddenly appear as intervening parameters under later concepts, or shift in scale from second-order to first-order factors. Too much investigation effort is rendered useless for future reanalysis merely because of a lack of somewhat more complete registry of facts. Those that are presumed of small relevance may merely be kept as constant as possible, and recorded, rather than being completely omitted from thought, observation or record.

Secondly, with the new tools furnished by statistical analysis and computers, one should dedicate much more effort to observe facts in prototype-scale, letting them speak for themselves, analyzing their trends through suitable statistical regressions, and finally fitting such behaviour into computorized simulation for the theorization. The approach of first visualizing a theory and then straight-jacketing the facts into the desired fit has hitherto occupied a large proportion of efforts, and may be said to have funnelled itself into a temporary condition of diminishing returns.

2. THE ULTIMATE BEARING CAPACITY OF FOUNDATIONS

2.1 Safety against failure: separate and prerequisite analysis

Indubitably the first step in the analysis of the adequacy of a foundation is to ensure that the maximum foundation loading foreseen will not cause a shear failure of the underlying soil, tantamount to the punching of the foundation into the soil, characterized by settlements of very large and unpredictable magnitudes, of a progressively deteriorating nature.

Indeed, as is well known, as soon as loads are applied to a foundation, settlements occur. Thus, a typical loading test result, or footing settlement observation, corresponds to a load or pressure vs. settlement curve which is generally continuous, with a slope increasingly steepening until the "ultimate", "critical", or "failure" loading or pressure is reached, frequently assumed to correspond to the abscissa of the point "at which the settlement curve becomes steep and straight" (Terzaghi & Peck, 1967).

We must at this stage defer for later consideration the quite distinct methods employed for running loading tests, and particularly, for interpreting the resulting pressure settlement curves.

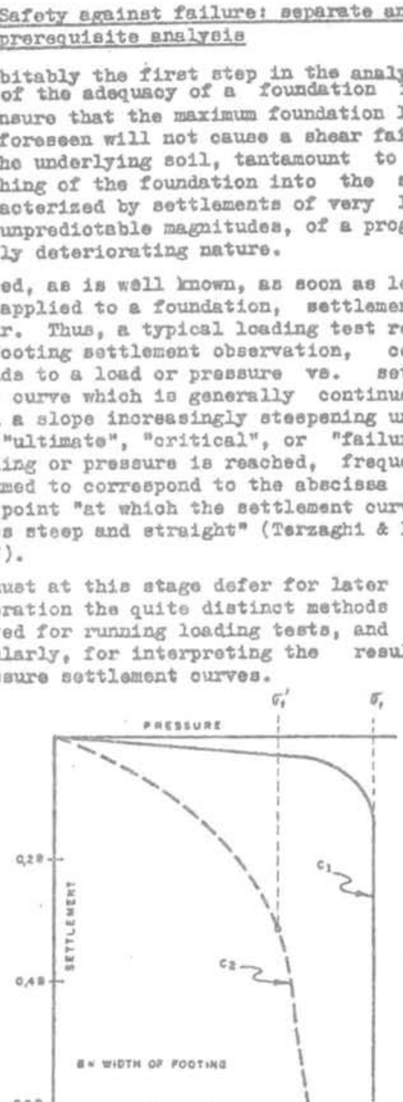


fig. 1

Pressure-settlement curves of load tests on C_1 dense or stiff and C_2 loose or soft soil

It is increasingly likely that considerable and laudable efforts will continue to be made in Soil Mechanics to unify all phases of the load-settlement-time behaviour of loaded plates and "piles", as part of a single complex process with progressively different proportions of participation of the theoretically differentiated components (e.g. Whitman & Hoeg 1965). Such endeavours will doubtless reach important conclusions based on model tests and computer solutions simulating idealized behaviour patterns: but the effective contribution of such information to foundation design practice is still not close in the offing. Moreover, the important point derives from the basic premise of engineering practice, whereby, in the face of any problem, ever inescapably fraught with unknowns and uncertainties, the solution must be formulated for that set of working hypotheses which would ensure the necessary conservatism or factor of safety F. Thereupon one must recognize that with particular reference to foundations on clays the critical conditions for maximizing potential development of shear failure and for maximizing compressibility settlements and their deleterious consequences, generally imply different loading hypotheses. In short, for instance, on a saturated clay a quick, undrained, loading condition most likely to cause failure is distinct from the slow long-duration loading that accentuates settlements.

Thus it seems that it will continue to be most appropriate to treat the two conditions (of decelerating and stabilizing compressibility settlements, versus potentially accelerating "failure" deformations) separately. Failure problems are obviously preeminently important. Thus, every foundation must first be investigated with regard to a minimum satisfactory F value against the ultimate bearing capacity failure: and, within such "safe stressing" the deformations and settlements must be computed, separately, since in many an instance the behaviour under the service loading and conditions may prove to be the controlling issue in comparison with the foundation failure criterion (Skempton, 1951).

Notwithstanding the great importance of the determination of the ultimate bearing capacity of a foundation, it is evident that the theoretical solutions to the problem are still subject to discussion, both in comparisons between them, and in comparisons with controlled tests designed to check their validity. However it will be shown that some degree of the edifying discussion does not significantly affect design practice. Thus, in order not to embark directly and discouragingly into the areas of controversy, however stimulating, it appears worthwhile beginning with a cursory presentation of the solutions that for some reason or another appear to have penetrated into wider acceptance and usage. There is no intent, herein, to establish relative merits of solutions: the reasons for the greater relative penetrations of some solutions are many, among which as important a part is played by the simplicity of use of formulae, graphs or tables of results, as by the authority of the author and the measure of divulgation of the work.

2.2 Bearing capacity solutions most commonly applied

2.2.1 Terzaghi's (1943) solutions and Krizek's (1965) empirical simplification

Terzaghi's (1943) bearing capacity theory represents one of the first efforts to adapt to Soil Mechanics the results of theorizing on the Mechanics of Continuous Media (Prandtl, Reissner) and, being sufficiently general, achieved for multiple reasons the position of a practical solution, most authoritatively backed, and of total penetration. Moreover, most formulae are generally transcribed into forms analogous to Terzaghi's, so that the most convenient manner of comparing the various theories is to compare the dimensionless factors called Bearing Capacity Factors, originally defined therein. It probably may be claimed that despite much complementary work in the intervening twenty-five years, Terzaghi's original solutions are still widely used.

With but slight adaptations to conform to subsequent more common versions, Fig 2 and the accompanying equations, together with the numerical values transcribed, indicate the basis and the results of Terzaghi's derivation for a strip foundation of width B at a "shallow" depth D, ($D \leq B$), below the surface of the surrounding soil, the shear strength of which is expressed by the Mohr-Coulomb equation $s = c + \sigma \tan \phi$.
The effect of depth is conservatively simplified into merely a flexible surcharge $q = \gamma D$, that is, neglecting the extension of the failure surface and its resisting shear stresses above the level of the base

of the foundation.

BEARING CAPACITY FORMULAE, SHALLOW FOOTINGS

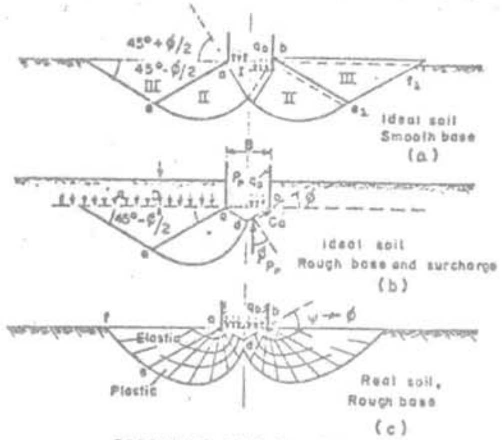
STRIP FOOTING: $\sigma_f = c N_c + q N_q + \frac{1}{2} \gamma B \; N_\gamma$

GENERAL FORMULA WITH SHAPE FACTORS

$$\sigma_f = S_c \, c N_c + S_q \, q N_q + S_\gamma \frac{1}{2} \gamma B \; N_\gamma$$

SQUARE FOOTING: $\sigma_f = 1{,}3\, c N_c + q N_q + 0{,}4\, \gamma B \; N_\gamma$

CIRCULAR FOOTING: $\sigma_f = 1{,}3\, c N_c + q N_q + 0{,}6\, \gamma R \; N_\gamma$



FORMULAE FOR Nc, Nq

Case	Nq	Nc
PRANDTL-REISSNER $\psi = 45 + \frac{\phi}{2}$	$N_\phi \, e^{\pi \tan\phi}$	$(N_q - 1)\cot\phi$ where $N_\phi = \tan^2(45 + \phi/2)$
TERZAGHI, FOR $\phi < \psi < 45 + \frac{\phi}{2}$	$\frac{\cos(\psi-\phi)}{\cos\psi} a^2 \tan(45+\frac{\phi}{2})$ where $a = e^{(3/4\pi - \phi/2)\tan\phi}$	$\tan\psi + m\left[a^2(1+\sin\phi-1\right]$; $m = \frac{\cos(\psi-\phi)}{\sin\phi\cos\psi}$
TERZ. RECOMMENDED "ROUGH BASE" $\psi = \phi$	$\frac{a^2}{2\cos^2(45+\phi/2)}$ where $a = e^{(3/4\pi - \phi/2)\tan\phi}$	$(N_q - 1)\cot\phi$

ϕ°		0	10	20	30	40	45
PRANDTL	Nq	1	2,47	6,40	18,4	64,2	134,9
REISSNER	Nc	5,14	8,34	14,8	30,1	75,3	133,9
TERZAGHI	Nq	1	2,6	7,3	22,0	77,5	170,3
$\psi = \phi$	Nc	5,7	9,1	17,3	36,4	91,2	169,3

FOR $\phi < \psi < 45 + \frac{\phi}{2}$, INTERMEDIATE VALUES

EX. $\phi = 30°$, $\psi = 45°$ $N_q = 18{,}5$; $N_c = 31{,}5$

Fig 2
Basic elements of Terzaghi (1943) bearing capacity solution, shallow strip loading

The general formula derived for the strip footing was further adapted by empirical shape factors, as shown, for square and circular footings. Thus the shape factors were estimated to be $S_c = 1{,}3$ and $S_q = 1$ for both square and circular footings, and $S_\gamma = 0{,}6$ for the circular and $S_\gamma = 0{,}8$ for the square footing.

The solution which Terzaghi proposed as more realistic is that wherein "on account of the roughness of the base of the footing and the adhesion between the base and the soil, the contact faces" of the underlying wedge of soil that descends vertically together with the footing "rise at an angle ϕ to the horizontal" (Terzaghi 1943 p. 123). However, formulae are given also for the limiting case (Prandtl) for which no shear stresses develop at the base of the strip load, originally stated to be the "perfectly smooth case", as well as for cases of "intermediate roughness" wherein the angle lies within the postulated limits $\phi < \psi < 45° + \phi/2$.

Considering the importance of Terzaghi's original solution on most of the subsequent work it is deemed worthwhile repeating the above fundamental information notwithstanding its appearance in most publications.

Since the Terzaghi solution for shallow footings is probably still the one most used, it is possible that engineers may already have adopted with interest the Krizek (1965) empirical simplification on the bearing capacity formula, dispensing with the cumbersome use of the graphs for the bearing capacity factors, since such graphs tend seldom to be easily read in the standard publications:

$$\sigma_f = \frac{(228 + 4.3\phi)c + (40 + 5.0)\gamma D + (3\phi)\gamma B}{40 - \phi}$$

2.2.2 Meyerhof's (1951, 1955) general solution and simplified recommendations (1963)

An important place within the area of general bearing capacity theory applied to foundation design practices is next assumed by Meyerhof's work (1951), with some earlier, and many subsequent contributions (e.g. 1955, 1963). The basic developments comprise (a) the extension of the previous analysis for a "surface footing" to shallow and deep foundations without the conservative simplification of the depth effect as a uniform surcharge at surrounding foundation level, (b) a more detailed examination of probable conditions of roughness of footing base, their consequences on the geometry of the failure surface, and consequent revisions of the bearing capacity factors.

According to this theory the zones of plastic equilibrium increase with foundation depth to a maximum for a "deep" foundation (Fig. 3), for each depth the size of the zones varying with the roughness of the foundation and also with the shape of the foundation. To simplify the analysis

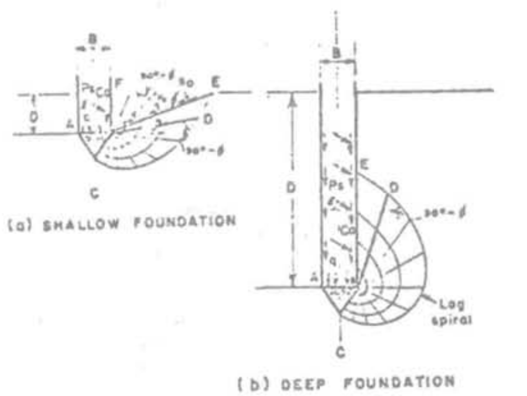


Fig. 3

Plastic zones near rough strip foundation (apud Meyerhof 1951)

Meyerhof postulates an "equivalent free surface" (the inclination β of which increases depth) on which normal and tangential equivalent stresses p_o and s_o are established on the basis of the equilibrium of the wedge BEF: for the derivation it is necessary to postulate a value m ($0 \leq m \leq 1$) of the degree of mobilization of the shearing stresses, up to the limit of the shearing strength, on the equivalent free surface.

The results of Meyerhof's derivation (1951) are, once again, furnished in the form employing Bearing Capacity Factors, but these are now not merely functions of ϕ, but also of the angle β connected with the depth of the foundation, and of the value m. Fortunately, however, regarding the latter value, which it would be most difficult to estimate, it is seen that its effect on the computed results is of second-order. The difficulties in the practical application of the Meyerhof solution therefore pertain principally to the assumption of reasonable values for β for intermediate cases between the $\beta = 0^o$ of shallow and the $\beta = 90^o$ of really "deep" foundations, wherein the depth is greater than 6 to 8 times the width. It must further be emphasised that the Meyerhof general solution

$$\sigma_f = c\,N_c + p_o\,N_q + \frac{1}{2}\,\gamma\,B\,N_\gamma$$

is expressed in terms of p_o which would have to be estimated and checked, although for the idealized cases of purely cohesive and purely cohesionless materials further developments of the solution have eliminated this inconvenience.

In a further meticulous discussion of the influence of roughness of base, Meyerhof (1955) concludes that the minimum values of N_c, N_q, N_γ correspond to values of ψ of the order of $1.2\,\phi$: the respective values are furnished graphically. Whereas it lies beyond the scope of the present report to enter into the details of such comparative analyses, reference to this work is inserted to emphasize the fact that not only are the hypotheses on these issues subject to discussion, but also the resulting theoretical and practical implications have been reasoned distinctly by various authors. For instance, the fact that the postulated Rankine state of the wedge under the footing develops the $(45 + \phi/2)^o$ base angles is tantamount to the admission of the strip loading as a principal stress, but does not necessarily imply for that contact the quality of "smoothness" corresponding to an inability to mobilize shear stresses.

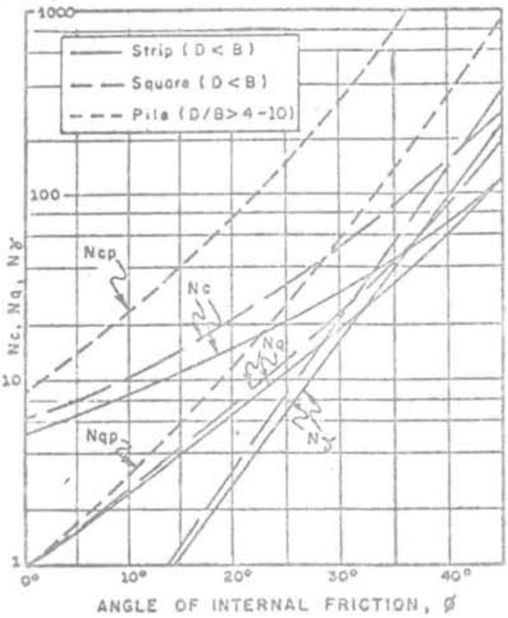
Since the aim of this section is to summarize the solutions that appear to be in more practical use, it may be noted that Meyerhof's many contributions have been distilled into the following simplified recommendations (1963) which are conveniently subdivided into the following four areas.

(a) Revised bearing capacity factors for shallow footings.

For shallow horizontal strip foundations the Prandtl (1920) theory is retained for the bearing capacity factors N_c and N_q ; using the customary $N_\phi = \tan^2(45 + \frac{\phi}{2})$, $N_q = N_\phi\, e^{\pi \tan\phi}$ and $N_c = (N_q - 1)\cot\phi$, and the N_γ factor is approximately established as

$$N_\gamma = (Nq - 1) \tan (1.4 \phi)$$

These bearing capacity factors are reproduced in Fig. 4 together with Terzaghi's and Brinch Hansen's values, for convenience of comparison.



COMPARATIVE VALUES BY TERZAGHI (1943) AND BRINCH HANSEN (1961) FOR SHALLOW STRIP LOAD

Ø		0°	10°	20°	30°	40°	45°
Nc	TERZAGHI	5,7	9,1	17,3	36,4	91,2	169
	B. HANSEN	5,1	8,3	14,8	30,1	75,3	134
Nq	TERZAGHI	1,0	2,6	7,3	22,0	77,5	170
	B. HANSEN	1,0	2,5	6,4	18,4	64,2	135
Nγ	TERZAGHI	0	1,2	4,7	21,0	130	330
	B. HANSEN	0	0,5	3,5	18,1	95,4	241

Fig. 4

Meyerhof (1963) general bearing capacity factors

(b) Formulation of "shape factors"

Shape factors of square and rectangular shallow footings are semi-empirically recommended as

$$S_c = 1 \quad 0,2 \; N_\phi \; B/L$$

$$S_q = S_\gamma = 1 \quad \text{for } \phi = 0^\circ$$

$$S_q = S_\gamma = 1 + 0,1 \; N_\phi \frac{B}{L} \quad \text{for } \phi > 10^\circ$$

(c) Formulation of "depth factors"

Semi-empirical depth factors are suggested for depths less than the width of the foundation

$$d_c = 1 + 0,2 \sqrt{N_\phi} \; D/B$$

$$d_q = d_\gamma = 1 \quad \text{for } \phi = 0^\circ$$

$$d_q = d_\gamma = 1 + 0,1 \sqrt{N_\phi} \; D/B \quad \text{for } \phi > 10^\circ$$

As the depth of the foundation increases further, the depth factors are found to increase at a decreasing rate approaching maximum values corresponding to pile foundations which are separately discussed.

(d) Pile bearing capacity factors

The bearing capacity of a single pile is taken as the sum of point resistance and skin friction, both of which depend on the properties of the soil and the method of installing the pile.

As regards the point resistance the basic bearing capacity equation can be suitably adapted, because of relative dimensions, into the form

$$\sigma_f = c \; N_{cp} + \gamma D \; N_{qp}$$

and the pile bearing capacity factors N_{cp} and N_{qp} are semi-empirically established to account for the limiting values of depth factors, the influence of the shape of the base (Meyerhof 1961) and the compressibility of the soil.

The values of N_{cp} and N_{qp} are represented in Fig. 4. They are stated to apply "only if the pile base is embedded in the load-bearing stratum near the base at least to a depth, which is, approximately

$$D = 4 \sqrt{N_\phi} \; B$$

If the soil properties vary near the base, the use of average values is suggested in accordance with the probable failure zone, of the order of one pile diameter above to four below the base.

2.2.3 Skempton's (1951) bearing capacity of clays

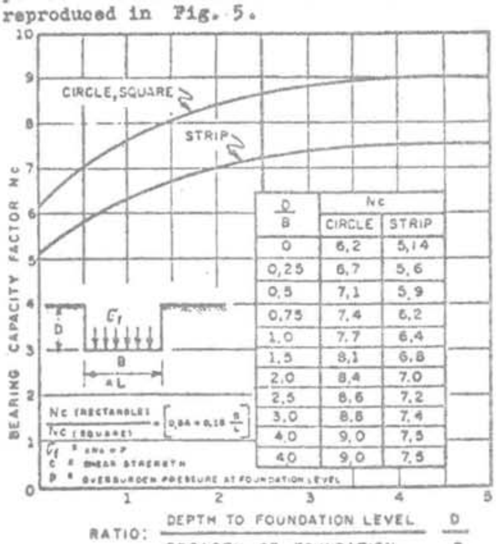
Specifically for the subject of bearing capacity of saturated clays for which it is stated that "the assumption that $\phi = 0$ forms the basis of all normal calculations of ultimate bearing capacity" (some special cases excepted) the basic reference in use in the profession is Skempton's paper "The Bearing Capacity of Clays" (1951).

Skempton rewrites the Terzaghi general formula to differentiate between the total stress, p, applicable at the base of the footing as a replacement of the excavation stress release, and the effective overburden pressure at the foundation level as the surrounding surcharge affecting the nett bearing capacity. However, for $\phi = 0$ the general equation reduces to

$$\sigma_{all} = \frac{1}{F}\, c\, N_c + p$$

and $\sigma_f = c\, N_c + p$

A thorough discussion of the derivations of Nc values is summarized, and, thereupon, as a conclusion the simplified rules were put forward as herewith transcribed and reproduced in Fig. 5.



D/B	Nc CIRCLE	Nc STRIP
0	6,2	5,14
0,25	6,7	5,6
0,5	7,1	5,9
0,75	7,4	6,2
1,0	7,7	6,4
1,5	8,1	6,8
2,0	8,4	7,0
2,5	8,6	7,2
3,0	8,8	7,4
4,0	9,0	7,5
40	9,0	7,5

Fig 5

Bearing capacity factors for foundations in clay ($\phi = 0$), (apud Skempton 1951)

(a) Surface footings (D = 0)

$N_c \simeq 5$ for strip footing;

$N_c \simeq 6$ for square or circular footings

(b) At depths D

Depth factor $d_c = (1 + 0{,}2\, \frac{D}{B})$

for $D/B < 2.5$

Depth factor $d_c = 1.5$

for $D/B > 2.5$

(c) At any depth, for rectangular footings of B x L

Shape factor $S_c = (1 + 0.2\, B/L)$

The confirmatory evidence for these recommendations will be pooled together with other data in discussions further on in this report. It may be noted with interest, however, that the Nc values of 6 (or 6.2, based on the more rigorous Prandtl 5.14 for strip footings) for square and circular surface plates, and of 9 for deep plates, has so gained acceptance that in any attempts at field test correlations the tendency during the past 15 years has been to question the shear strength tests and c values, through admission of the above Nc values as definite.

2.2.4 Brinch Hansen's (1961, 1966) simplified recommendations

Among the solutions in more general use mention must finally be made of Brinch Hansen's "General Formula for Bearing Capacity" (1961) with the subsequent partial revision (1966).

In the notations above employed, and excluding for the present the "inclination factors" for inclined loading, the ultimate bearing capacity of footings of any shape and depth may be written as

$$\sigma_f = (c + q \tan\phi) N_c S_c d_c + \gamma B (\tfrac{1}{2} N_\gamma) S_\gamma d_\gamma + q$$

wherein the interrelation between the Nc and Nq factors

$$N_c S_c d_c = (N_q S_q d_q - 1) \cot\phi$$

has been employed to eliminate the Nq term.

For the Nc factor Brinch Hansen retains the basic Prandtl solution whereby

$$Nc = (Nq - 1) \cot \phi$$

and

$$Nq = N_{\phi} e^{\pi \tan \phi}$$

For the N_{γ} factor, for which no "rigorous" solution was available from both static and kinematic considerations, the author adjusted an empirical relation

$$N_{\gamma} \doteq 1.8 \ (Nq - 1) \ \tan \phi$$

as lying intermediate between the solutions representing what he recognizes as being upper (Meyerhof 1951) and lower (Lundgren and Mortensen, 1953) limiting values.

For shape factors the empirical equations suggested have been simply reduced to(1966)

$$Sc \doteq Sq = 1 + 0.2 \ \frac{B}{L}$$

and $$S_{\gamma} = 1 - 0.4 \frac{B}{L}$$

For depth factors it is first concluded that $d_{\gamma} = 1$. Further, an empirical formula for shallow foundations $(D < B)$ is given for $d_c = 1 + 0.35 \frac{D}{B}$. It may be noted that because of the fórm of the general equation adopted, a separate depth factor d_q is excluded.

Finally, as regards depth factors for great depths, it is stated that "it must be admitted that at present too little is known", so that the empirical formulae and graphs earlièr proffered (1961) were revoked (1966).

2.3 Shear strength values applied to $\phi = 0$ analyses of bearing capacity

Since practically all the confirmatory evidence on ultimate bearing capacities of clays is connected with saturated clays and the $\phi = 0$ analyses associated with rapid loading, it behoves us to examine somewhat the methods employed, over the past thirty years, to define the c values that enter into the correlation $\sigma_f = Nc \ c$. Obviously, besides the "random dispersions" in the measurements of the parameters σ_f and c, which necessarily affect the level of confidence of any determination of Nc by a statistical regression, important changes may be introduced in the very statistical universe examined, and therefore in the average Nc value determinable, at every significant change introduced in the methods of measuring either of the separate parameters σ_f and c.

Fundamental to the application of any calculation of the ultimate bearing capacity in a soil is the appropriate definition of the shear strength values to be employed. Moreover, since the very subject of definition of the shear strength of clayey soils has been at the core of the most intense programs of research and revision during the entire period within which bearing capacity formulae have been concomittantly reexamined, it is necessary to examine the problem jointly, lest one-sided developments within one sector, even when in the right direction, inadvertently invalidate the mutual relationship implicit.

It may be that for empirical confirmation of the bearing capacity factor Nc, some of the early theoretical formulation relied upon data from model tests on remoulded clays (Golder 1941), and that most frequently the concomitant undrained strength determination was by the unconfined compression (UC) test wherein $c = \frac{q_u}{2}$ (unconfined compression strength). Such formulations of the correlation must be accepted as valid within the degrees of precision which prevailed in the measurements of σ_f and of c for the particular experimental set-up, and to the extent to which the establishment of the failure pressure was not unduly influenced by accentuated deformations and by loading techniques and rates. The roughness of the base of the plate is being considered as a random factor within this Report.

Thus, whatever may be the correlating factor Nc thence derived, which could be considered rigorous if the above-mentioned margins of errors were small, it would primarily be associated with UC tests. Next the same reliance on UC tests was transplanted into field problems. Thereupon, all testing techniques tending to underestimate in-situ c values would lead to conservative design practice, and, with the same Nc value maintained, any developments tending to revise in-situ c values towards an upper limit truly representative of virgin in-situ undrained strength behaviour of soil elements along the presumed eventual failure surface will merely raise the design pressure. The same will also happen, independently of any revision of the concept and the number reflecting the desired factor of safety F, if merely a smaller range of dispersion is achieved around the statistical correlation established.

The subject subdivides into two main avenues, that of improved undisturbed sampling and representative laboratory testing, coupled with interpretative attempts at visualizing the behaviour as excluded from inevitable though minimized sampling and testing effects: and that of in-situ shear testing on the other.

2.3.1 Sampling and laboratory testing

Along the line of sampling and laboratory testing, in order to represent adequately the behaviour of in-situ soil elements under the proposed foundation, so much outstanding work has been done during the past twenty years that to attempt to digest the published data on the subject would constitute another important state-of-the-art paper in itself. The interested reader is referred to some selected papers as a key to the vast field, lest the present report convey any fallacious or over-simplified impression (e.g. Skempton and Sowa 1963, Ladd and Lambe 1963, Noorany and Seed 1965, Ladd 1965). Some of the principal steps that are herewith mentioned are intended to convey the writer's impression of the contributions that have distilled into more significant positions within the practises of foundation engineering.

(a) Sampling technique

Whereas under many routine jobs of lesser responsibility twenty years ago some preliminary testing was performed (on soils of low sensitivity S_t) with samples retrieved within a liner inside the thick-walled driven spoon sampler of the Standard Penetration Test (Terzaghi & Peck 1948), the publications by A. Casagrande (1932) and Hvorslev (1949) rapidly fostered the recognition of the pushed 2" thinwalled - Shelby tube sampler as a minimum. In undisturbed sampling of soft to medium clays the widely recognized major developments are the Osterberg stationary-piston hydraulically pushed sampler (Earth Manual, USBR, p. 369) and the Swedish foil sampler (Kjellman, 1950). The acceptance of undisturbed block samples, hand-extracted from pits, as the best possible sample continues in vogue, even though it may in special cases (Ward et al, 1965) be subject to difficult problems associated with stress releases accentuated by the more delayed and wider excavation. As regards stiff to hard clays, usually of very low sensitivity S_t, the thicker driven sampler continues in use as unavoidable and as explainably acceptable (Moretto 1967) and, as an alternative, one must resort to special core samplers such as the Denison double-tube core barrel (USBR 1963, p. 358)

(b) Laboratory testing

As regards laboratory testing with the undisturbed samples retrieved, early improvements attempted (Taylor, et al.) comprised the substitution of unconsolidated-undrained (UU) and consolidated-undrained (CU) triaxial tests for the unconfined compression test, (UC), which frequently indicates a c value significantly lower than the intercept of the $\phi = 0^{o}$ straight line of the series of UU tests (Skempton & Henkel 1953, Farrent 1960, et al). The topic has been recently receiving much attention whereas earlier the preferences had been more or less tacit, without the possibility of adequate experimental backing. It appears quite obvious from the publications reviewed that the conclusions on the applicability of the above laboratory tests as representative of in-situ shear strengths will vary considerably from one clay deposit to another, and even within a given clay deposit varying with the depth and the sample's degree of preconsolidation, depending principally on what may be described under the general term of "structure", including both the "microstructure" of sensitive clays, and the "macrostructure" of fissured clays.

Mention has not been made, herein, of the Swedish fallcone test (Hansbo 1957), frequently used in Scandinavia in combination with UC tests, and employed as a basis of confirmation of some stability analyses and plate load tests (Odenstad 1948). The later study indicates that the method may be suitably correlated with the undrained shear strength of the clay (for which the field vane test result is accepted as the standard of reference), and, in that respect gives better results than the UC tests in the sensitive clays. However, the author points to the importance of relating the fall-cone test, to the type of sampler used and to other details of design, procedure, and interpretation, concluding that the values of the field undrained shear - strengths of a clay "given by the new interpretation of the fall-cone test differ from those obtaining hitherto, and a re-appraisal of the safety factors to be-applied may be necessary".

2.3.2 Field vane testing

Principally because of the problems associated with undrained in-situ shear strength determination of soft very sensitive and "quick" clays, the field vane testing device and techniques (Cadling and Odenstad 1950), with subsequent developments, have gained wide acceptance as about the best method presently available. However, the opinion is not absolutely generalized, since there are references wherein the sampling and laboratory testing concept continues to merit preferences, even setting aside the cases wherein practical considerations would impose it.

At any rate, since some basic standard of reference must be admitted, it shall herein be accepted as true that the in situ vane values, in saturated clays of $\phi = 0$ conditions, are representative of in situ shear strengths for the current stability analyses.

Truly one must concede that at the present moment the study appears founded on somewhat arbitrary premises, since the methods of stability and bearing capacity analysis are under inevitable revision, and so are the basic strength parameters to be inserted in them. As will be concluded, the situation points to a need for additional especially programmed tests, and a reappraisal of earlier observational data, since in the final analysis all of these studies are merely means to an end, and the end, which is the design of the engineering structure, is not being served at all if a new set of design recommendations does not result therefrom. But con concomitantly it must be conceded that some developments are to be set aside, temporarily, as belonging to the field of "random factors", not merely because not enough is known about them, but also because available methods of design analysis are not able to incorporate them. In such a category lies for the present, as an instance, the consideration of anisotropic in situ strengths as revealed by different vanes (Aas, 1965; Lo and Milligan, 1967).

Without any illusion or hope of attempting an adequate coverage on the subject, which depends on multiple factors (e. g. Jakobson, 1954, et al.) the accompanying Table I and Fig. 6 have been prepared to illustrate the diversity of results and conclusions. Despite the tremendous dispersion of results, Cadling and Odenstad 1950 had shown for clays of strengths between 1 and 4 t/m² at depths of 0 to 20 m, that the ratio of the vane strength τ_f to the UC τ_f value can increase almost linearly by about 100 % for every 10 meters. Most of the other investigators have similarly found that acceptable results of UC tests may be obtained up to depths of about 15 m, but at greater depths the vane values continue to register the regular increase of strength with depth while UC values tend to remain constant. Much depends on sensitivity. It is important to note that there are cases of medium to stiff clays for which the UU values were as low as one-half the corresponding vane results: such variations naturally strip of any engineering significance the discussions of Nc values in the order of ± 20 %.

The dispersion in both types of approaches are major, and point to the need of statistical treatments. Lumb (1967) investigates statistically the minimum number of tests required to estimate the strength of a clay to within 10 %, within 95 % of confidence level, assuming a purely cohesive material of strength independent of depth, and a normal distribution of the test results. The conclusion is that 30 tests would be required so that the mean strength be determined to within 10 %.

It must be recognized that such reasoning constitutes an unavoidable step in any testing and design program. Depending on the level of confidence, and, consequently, presumably on the F value to be adopted, a minimum number of tests must be established (Mello et al, 1959). Much depends on the nature of the distribution of the random factors around the mean. Lumb (1967) concluded that the distribution was normal for a marine clay, and postulates that the same would hold for other soft normally consolidated marine clays. It is feared that in stiff fissured clays the distribution of laboratory test results may not be normal, which may be one of the reasons for the difficulties of interpretation of the behavior of London Clay in the light of routine techniques and theories.

Special reference must finally be made to the multistage triaxial test, initially divulgated by Taylor (1950) and today very widely accepted. By employing a single specimen for several triaxial tests it avoids some of the dispersions, especially

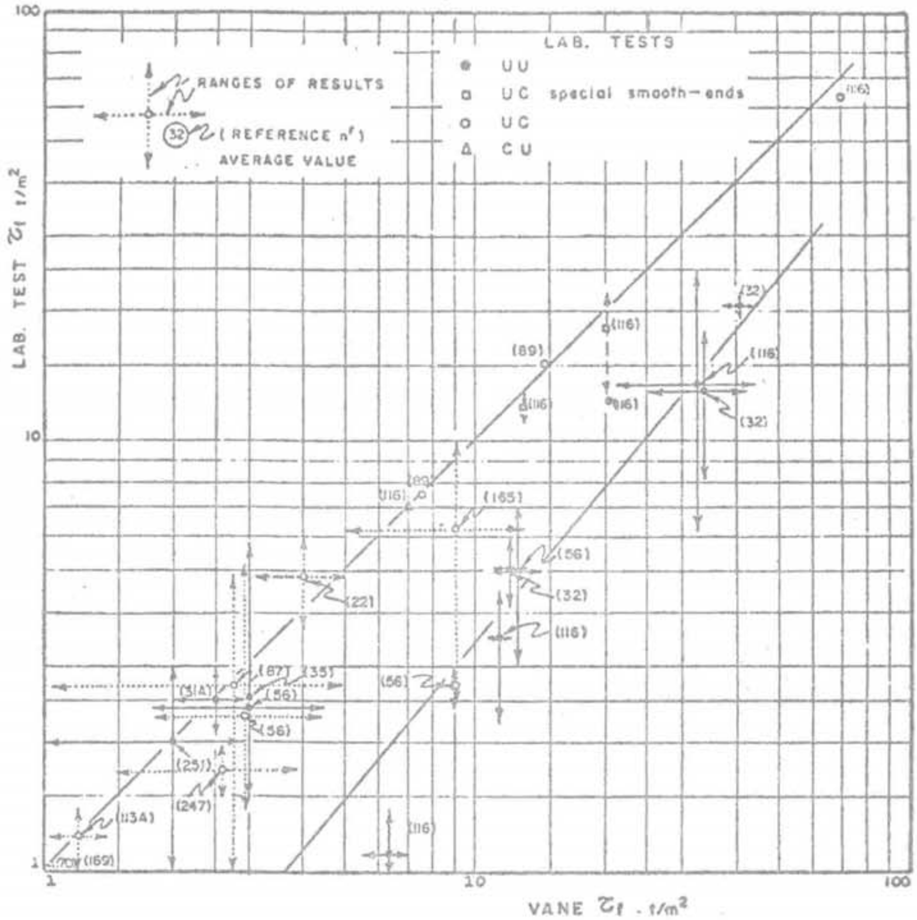


Fig. 6
Range of dispersions in undrained τ_f from
UC, UU, CU, and special CU tests vs. in-situ vane results

in the determination of values of ϕ as defined by the ratio $d\tau_f/d\sigma$.

2.3.3 Foundation failures and shallow plate tests

Considering the above dispersions, and differences in method and result, in defining the in-situ undrained strength of saturated clays under plate load tests and sundry foundation failures, it appears that the acceptance of an average No value of the order of 5 to 6 for shallow foundations is not subject to errors of more than about ± 20 %.

As will be emphasized below, an equally significant influence is played also by the method employed to run the load test and to interpret its ultimate value, especially in the case of soils subject to large deformations.

TABLE I

Illustration of comparative values of undrained strengths (t/m^2) determined by different methods

Ref. nr.	Vane / no. τ_f /(tests)	τ_f UU / UC * / no. CU **/ (tests)
116	21-32.6-44 (9) 5.5-6.3-7 (3) 11 -11.3-12 (3) 20.4 (1)	6.1-13.4-24.5 (9) 0.8-1.1-1.2 (3) 2.2-3.5-3.8 (3) 12-12.4-12.7 (4)
32	11 - 13 37 - 44 25 - 42	4 - 6 19 - 22 8 - 18
251	1 - 3 (~20)	1-3(~20:4" samples)
170	0.3 - 1.3 (~ 60)	0.3 - 1.3 (~60:Swedish foil)
56	1.5-3-5.6 (24)	1.8-2.4-4.4 (23) 1.4-2.3-5.3 (33) *
39A	2 - 3 (22)	2 - 3 (5)
56	11 - 14 (~20)	3 - 7 (3)
13A	1 - 1.4 (many.)	1 - 1.4 (many)
116	7 (1)	7 (1) **
116	13 (1) 20 (1) 70 (1)	11.2-11.8-12.3(4)a)* 12.2-18.1-22 (8)a) * 72 (2) a) *
169	0.25 - 1.25	0.25 - 1.25 (85:Swedish foil)*
89	7.5 14.5	7.5 ** 15 **
22	3 - 5 (14)	3.5 - 6 (34) **
87	0.5-5 (~300)	0.5-5 (~900)
56	9 (1)	2.7 (3) in situ **
165	5 - 13 (many)	2.5 - 10 (many) **
35	2 - 4 (~20)	1 - 4 (~20) **
13A	1 - 1.4 (many)	1 - 1.4 (many) **
247	1.5 - 4 (8)	1.5 - 1.8 (8) **

a) Special test, smooth ends

For further illustrative references of different cases and consequent opinions, the reader may consult, among others:

In favour of in-situ tests as against sampling and testing:-

Marsland and Butler 1967, Oslo, Proceedigns of the Geotechnical Conference, I p. 139; Crawford and Eden, 1965, Montreal 6th International Conf. on Soil Mech., I p. 39, for Leda Clay; Hanna, T.H., 1967, discussion ASCE Proceedings SM 3, p. 183 ; Milligan, Soderman and Rutka, 1962, ASCE Proc. SM 4, p. 31, for Canadian varved clays; Newland and Allely, 1952, 1st Australia-New Zealand Conf. on Soil Mech., p. 123; Gray, H., 1957, ASCE Transactions 122 p. 844; Weber and Kleinman, 1959, ASTM Spec..Tech. Publ. 254, p. 284.

In favour or in acceptance of laboratory testing:-

Marsland and Butler 1967, Oslo, Proc. of the Geotechnical Conf., I p. 139 for 3" to 5" specimens; Crawford and Eden, 1965, Montreal, 6th International Conf. on Soil Mech., I p. 39, for a preconsolidated clay; Blight, G.E., 1967, Cape Town, Proc. 4th Regional Conf. for Africa on Soil Mech., I p. 229; Noorany and Seed, 1965, ASCE Proc. SM 2, p. 49; Wu, T.H., 1958, ASCE Proc. SM 3 paper 1732; Skempton and Sowa, 1963, Geotechnique, XIII p. 269, for a remolded reconsolidated clay; Lo and Milligan, 1967, ASCE Proc. SM 1, p. 1, for anisotropic stratified clays; Weber and Kleiman, 1959, ASTM Spec. Tech. Publ. 254, p. 284, up to depths of about 30 m; Meyerhof, G.G., 1950, discussion, II. p. 135.

As regards the ultimate bearing capacity of shallow footings on $\phi = 0$ clays the main source of error and dispersion therefore continues to be the definition of the applicable c value. And since this problem still lies within the realm in which behavior, results, and interpretations, may vary very significantly from clay to clay (and from place to place with respect to sampling and testing procedures and precisions) it appears that any attempt to improve present design criteria must still resort either to locally proven practices, or to an awaited collection of a vast amount of more truly representative data for statistical analyses.

In Table II, an attempt is made to up-date earlier similar tabulations in support of Nc values and simple $\phi = 0$ foundation failure analyses. The coverage unfortunately cannot presume to be complete or of any more than first-order accuracy. Wherever possible the original publication was

TABLE II ILLUSTRATION OF SHALLOW FOUNDATION LOAD TESTS AND FAILURES ESTABLISHING N_c VALUES.

REF N' 1st	REF N' ADDITIONAL	REF N' USED	CASE and DIM. B x L x D m	CLAY	AVER. c t/m² AND TEST	N_c	OBSERVATIONS L.T. = LOAD TEST S.C = SWEDISH CONE V = VANE; d = DIAM.; F = FAILURE
(98) GOLDER 1941	(250) (248)	(250)	Models 0,08 x 0,08 / 0,08 x 0,25	Rem. London		≈ 5,2 / ≈ 6,7	strip / square
(219) ODENSTAD 1948 HAGALUND	(44) (219) (250) (17)	(44)	9 L.T. 0,4 x 2,0 pits 2,2 x 1,1	Und.	UC ≈ 0,68 / V ≈ 0,81 / SC ≈ 1,05	≈ 5,2 / ≈ 5,8	strip : vane
(216) SKEMPTON PIPPEN 1942	(250) (17)	(250)	Footing F. 2,5 x 2,8 x 1,7	Und.	UC 1,6	6,0 − 7,2	
(214) NIXON 1949 Shellhaven Tank A	(287) (250) (242) (17)	(214) (250) (242)	Tank F. 7,5 d x 0,0	Und.	UC, UU 1,5 / 1,35	7,4 / 6,2	
SHELLHAVEN TANK B	(250) (17)	(250)	Tank F. 16 d x 0,0	Und.	UC, UU 1,40	5,9	
MEIGH + YASSIN 1951	(250)	(250)	Models penetr from surface 0,006 to 0,025	Rem. + Und. London		6,2	σ_f interp. differently from penet. curve
(20) BISHOP & LITTLE 1967		(20)	4 - L.T. 0,3 x 0,3 x 0 / 0,3 d x 0	Und. London	UU 3−7 block, tube, and 0,6 x 0,6 in-situ direct shear	5,6 / 10, 3	cf. in-situ shear / cf. tube UU σ_f
(84) ERTEL 1967		(84)	L.T. 0,5 d x 0,5 in pit, possibly narrow	Frankfurt Und.	UC = 11	6,5	σ_f at ρ = 0,1 d
(116) JACOBSEN 1967		(116)	14 L.T. (0,05−0,15) d x a / 6 L.T. / 2 L.T. / 6 L.T.	Danish preconsol. boulder clay	V 6 ± 0,6 / V 22 / V 70	min. av. max. / 4,9 - 5,8 - 6,4 / 5,4 / 4,7 - 5,5 - 6,6	
(22) BJERRUM & OVERLAND 1957	(242) (17)	(22)	Tank F. 25 d x 0	Und Marine	V 3−5 / UC 3−5	5,6	Assumed local F for part of loaded area, 10m wide
(113) HOSHINO 1953		(113)	≈ 50 L.T. 0,2; 0,3; 0,5 d	Und Marine	UC 2−5	6	
(65) DASTIDAR 1967		(127)	6 L.T. 2 of 0,3 x 0,3 / 2 of 0,6 x 0,6 / 1 of 1,5 x 1,5	Slickensided Haircracked	UC tube 5−13 / UC block 2−3	2,6 / 10	M.L. test
(47A) CARLSON & FRICANO 1961		(47A)	Tank F. 50 d	Und silty	UC 2	≈ 6,2	
(224) PECK & BRYANT 1953	(17)	(224)	Silo F. (Transcona) 20 x 60 x 3,7	Und., silty ± slickensides Stiffer at top	UC 5−3,5	5,1 − 7,2	Based on Skempton (1951)
(287) TSCHEBOTARIOF 1951	(17)	(287)	Silo F. 15 x 69 x 0,9	Varved glacial, stiffer at top. St increase with D, 2−10	UC 10−5	5,2 − 5,9	cf Skempton (1951) averaging τ_f progres. effects
(21) BJERRUM 1954	(17)	(21)	4 L.T. 0,36 d at D / 0; 0,36; 0,36; 0,72	Quick S_t = 80	UC 6	D/B = 0 : 6,5 / D/B = 1 : 7,8 / D/B = 2 : 8,9	cf Skempton (1951)
(79) EDEN & BOZOZUK 1962		(79)	Silo F. 6,7 d x 1,2 / load 550 ± 50 tons	Varved S_t = 15	UU 1,2 / V 1,6	—— / 6,6	cf Skempton (1951) Meyerhof (1951)
(56) CHRISTENSEN & BENT HANSEN 1959		(56)	21 L.T. 0,05 − 0,3 d	Skive Septarion, high precons.	UU, UC 3 − 5 / V 9 − 14	6,0	N_c assumed checks with UU, UC

examined: moreover, the reappearance of the same data reinterpreted in other publications is recorded as far as possible.

The obviously striking fact is that the greatest proportion of published data concern soft clays and attempts to improve the rigour of determinations of what may still be classed as second-order factors.

An important development in the analysis of foundation failures of the larger loaded areas is furnished by Raymond (1967), wherein the increase of the in situ strength with depth is taken into account. Earlier computations had been based on Skempton's (1951) recommendation that "if the shear strength within a depth of approximately 2/3 B beneath foundation level does not vary by more than about ± 50 % of the average strength in that depth, then this average value of c may be used ...", and thereupon computations were based on constant c values subjectively averaged. Such a simplification could obviously lead to noticeable errors in cases of slip circles that develop to greater depths (Kantey 1953, et al). The important point, moreover, is to distinguish between random variations about the average value, which could possibly be even greater than ± 50 %, and the definite trend of increase of strength with depth, irrespective of such dispersions.

2.4 Bearing capacity theory and formulae

2.4.1 Summary of present status of plate bearing capacity

It was indicated above that within the prerequisite preoccupation of every foundation design with regard to an adequate factor of safety F against a general failure, it appears that the crux of the problem continues to be connected with the appropriate selection of the applicable shear strength parameters, and that, as a result, there has not been a noticeable practical interest in the collateral developments taking place within the area of theoretical formulation of improved bearing capacity equations. On the other hand, it must be reminded that the very applicability of a given set of strength parameters to a bearing capacity formula represents a simplification of a stress-strain concept and must necessarily depend on the similarity between the mathematically hypothesized behavior, and the real behavior of the soil as revealed by indirect and direct tests. Thus inescapably it becomes necessary to examine, at least cursorily, the principal problems and solutions of bearing capacity theory.

Usually it is much easier to design for the limit load than for the service load, using such resources as the Theory of Elasticity for the latter, and, as a principal recourse, the Theory of Plasticity for the former. Thus, despite the many difficulties of its application in soil mechanics, the solutions based on the latter theory constitute the most common avenue. This theory can only be used if: (a) the problem in question is a failure problem; (b) the materials have plastic properties at failure, i.e., are able to sustain relatively large deformations under constant failure stresses, permitting redistribution of stresses; (c) the pertinent constants can be determined with reasonable accuracy; (d) it is possible to find a reasonably correct solution to the failure problem.

It appears that all the mathematical theories of bearing capacity in soils, developed through Plasticity Theory, had as a starting point Prandtl's solution of the problem of indentation of a rigid-plastic incompressible and weightless, continuous, semi-infinite, homogeneous and isotropic medium by a rigid solid of finite width and infinite length causing plane deformation conditions. Apparently the single exception of a different approach is that employed by Bishop-Hill-and Mott in metals, and adapted for clays by Gibson through Swainger's large-strain theory, simulating the failure of a deep foundation in clay as the expansion of a spherical cavity (Skempton 1951).

Within the main direction set by Prandtl, two principal avenues of reasoning and endeavour have been followed: one being the more purely mathematical, attempting to solve more "rigorously" the equations, boundary conditions, and kinematic conditions that the mathematical hypotheses would impose; the other being partly experimental, attempting to observe more carefully the nature of movements of the soil under model tests, in order to simulate as truly as possible the geometry of the failure surface, the movements of the masses of soil, and the boundary conditions of the problem (Bergfelt 1956, de Beer and Ladanyi 1961, Biarez et al 1961, Jumikis 1961, Spencer 1964, et al.).

Against any disproportionate zeal towards the first objective one might merely level the criticism that the rigour sought would, when consummated, merely achieve a true representaion of a hypothetical rheological model and its mathematical behavior. The statistical dispersion within which a real soil would be represented by such a rheological model still awaits investigation. With respect to the second line of investigation the problems raised concern the measure within which the small-scale models do represent the desired prototype, and permit observations of a precision capable of distinguishing between different geometrical formulations of the surfaces and kinematics involved.

Fortunately, in simplifying the present aim at presenting a very brief summary of the status of bearing capacity theory, it is possible to refer the reader to a series of very recent and thorough publications (de Beer 1965, Kerisel 1965, Vesic 1965, Martins 1965, and Brinch Hansen 1966).

As a first step the analysis is concerned with a strip load on the surface of a weightless material. The problem was treated by Rankine (1857), Prandtl (1920), Reissner (1924), Buisman (1940), Terzaghi (1943), Meyerhof (1948, 1951), Mizuno (1948), Caquot and Kerisel (1956) Sokolovski (1960) and Biarez (1961) among others. The Prandtl-Reissner solution for this weightless material, under a strip vertical major principal stress surrounded by a surcharge q, is considered a "rigorous" solution, giving the respective classical N_c and N_q equations. The subsequent solutions incorporate different attempts to introduce "realistic" friction conditions at the base of the footing, and failure masses, surfaces, and movements more in accord with theoretical impositions.

With the interest still restricted to surface footings, the next step generally comprises the introduction of the N_γ term, in attempts to account for non-weightless material ($\gamma \neq 0$), for which case no rigorous analytical solutions are yet available. As may well be seen, the controversy around the values of N_γ is considerable (de Boer 1965). It is of interest to note that within the area of theoretical derivations two sources of error and discrepancy arise thenceforth: the first concerns the choice of method of stability analysis to be employed, with its explicit failure surface assumption; and the second concerns the fact that upon completion of such a stability analysis, with consequent minimization of the factor of safety along some surface distinct from that which prevailed for the weightless case (for N_c and N_q), the general bearing capacity formula is obtained by adding the two theoretical non-coincident conditions. In this respect, Brinch Hansen (1966) gives a very elucidative picture of how the N_q value would simultaneously change, depending on the method of stability analysis. Comparing several methods of stability analysis for the simple case of a weightless soil of $\phi = 30°$, for which the exact Prandtl solution is known, he shows that Rendulic's "extreme method" and his own "equilibrium method" of analysis come closest to reproducing the known value (an error of about 22 % on the unsafe side). It is of special interest to note that the values range from 30 % to 169 % of the "exact" value of Prandtl's postulates, and that within a major portion of the assumed failure surfaces, their near coincidence would make it rather difficult to distinguish, in small-scale model tests, as to which seemed more realistic.

The very revealing fact also is that recent developments in the subject indicate the futility of theoretical developments unaccompanied by satisfactory test results (de Beer 1965). In order to obtain a better fit of theoretical curves to "suitable" test results, not only are corrections introduced (Meyerhof 1963) in the value of ϕ because of the roughly 10 % larger value found in plane strain tests in comparison with triaxial tests; but also, the curvature of the Mohr envelope, presumed as $\tau_f = \sigma \tan \phi$, is introduced in view of the crushing of grains, as a function of the modulus of elasticity of the material constituting the sand particles. Thus, indirectly, the compressibility of the non rigid-plastic material begins to interfere.

The problems in this area have been treated by Buisman (1940) referred by de Beer and Wallays (1948), Terzaghi

(1943), Mizuno (1948), Caquot-Kerisel (1948, 1956), Meyerhof (1948, 1951, 1955), Lundgren-Mortensen (1953), Kopadzi (1961), Gorbunov-Possadov (1965), among others. It is generally recognized that the satisfactory solution of the general case must be obtained either by rigorous numerical methods as suggested by Lundgren-Mortensen (1953) and Sokolovski (1960) (the latter better adapted to computer solutions), or by "rigorous" graphical methods such as Josselin de Jong's (1959).

As regards deep foundations in clays, treated as $\phi = 0$ materials, no theoretical treatments have been proposed (Kerisel 1965) beyond those already mentioned under the reference to Skempton (1951) under item 2.2.3. Recent experimental evidence, principally with respect to London Clays and to the use of the Static Penetrometer will be discussed separately. The latter has, in cases, suggested Ncp values as high a 15 to 35 instead of the theoretically proposed Ncp = 9: but hitherto it is the penetrometer that has been subject to cross-examination under the assumption of the Ncp = 9 as well-established, and not the contrary. On the other hand, the experimental evidence from large--bored pile tests in London has pointed to the major relevance of the interference of deformation phenomena, and of the necessary definition of "realistic" shear strength parameters: once again, the tendency has been to accept Ncp = 9 and to adjust field and laboratory tested strengths to the "appropriate value". It is important to emphasize that the interference of deformation phenomena, through elasto-plastic analysis of a strain-softening material to represent more correctly the typical stress-strain behavior of undisturbed sensitive clays, has led Ladanyi (1967) to conclude that the Ncp value of 9, valid for non-sensitive clays, should decrease to about 6 to 7 for clays of moderate sensitivity, and to about 4 for extremely sensitive clays.

Finally, as regards deep foundations in sands, one observes that a series of prototype-scale tests very carefully conducted and observed has forced a very major revision of earlier theoretical concepts (Vesic 1965). To begin with, in the case of deep foundations in sands ($\phi > 0^o$), the controversial N_γ factor had been set aside because of the insignificant dimension of the point in comparison with the depth. However, with the formula reduced to the form σ_f = Sc c Ncp + Sq q Nqp, and with the acceptance of the first term as satisfactorily established, the differences of method of formulation of the problem, as summarized in Fig. 7 had already led to very major discrepancies in Nqp values proffered by various authors (Vesic 1965).

Further, the new experimental facts (Kerisel 1961, et al.) revealed, once again, the very major influence that deformation problems, not considered until then, play in the determination of failure conditions, appropriate values, and consequent Nqp values. Fig. 8 herein reproduced from Vesic 1965 serves to illustrate the proportions of the effect of compressibility considered.

Although the subject has been treated solely in connection with sands, its inclusion in the present report is considered necessary on three counts: (a) to reemphasize the need for prototype-scale observation of facts, possibly under improved techniques and renewed perspectives; (b) to reemphasize the influence of compressibility and deformability phenomena, which would analogously affect the general case of (c, ϕ) clayey soils, (c) to illustrate, within the general panorama of bearing capacity problems how little has really been effectively established for the general soil case.

2.4.2 Analysis of the significances of different theories

In appraising the routine foundation design procedure, one must recognize the fact that engineers are immediately faced with the problem of selection of a formula to use. Unless such a selection is facilitated by considerations of tradition, authority or sympathy, one cannot ignore the fact that the Nc, Nq and N_γ factors recommended by different authors can appear discouragingly different.

Therefore, as a next step in the present report it was decided to investigate

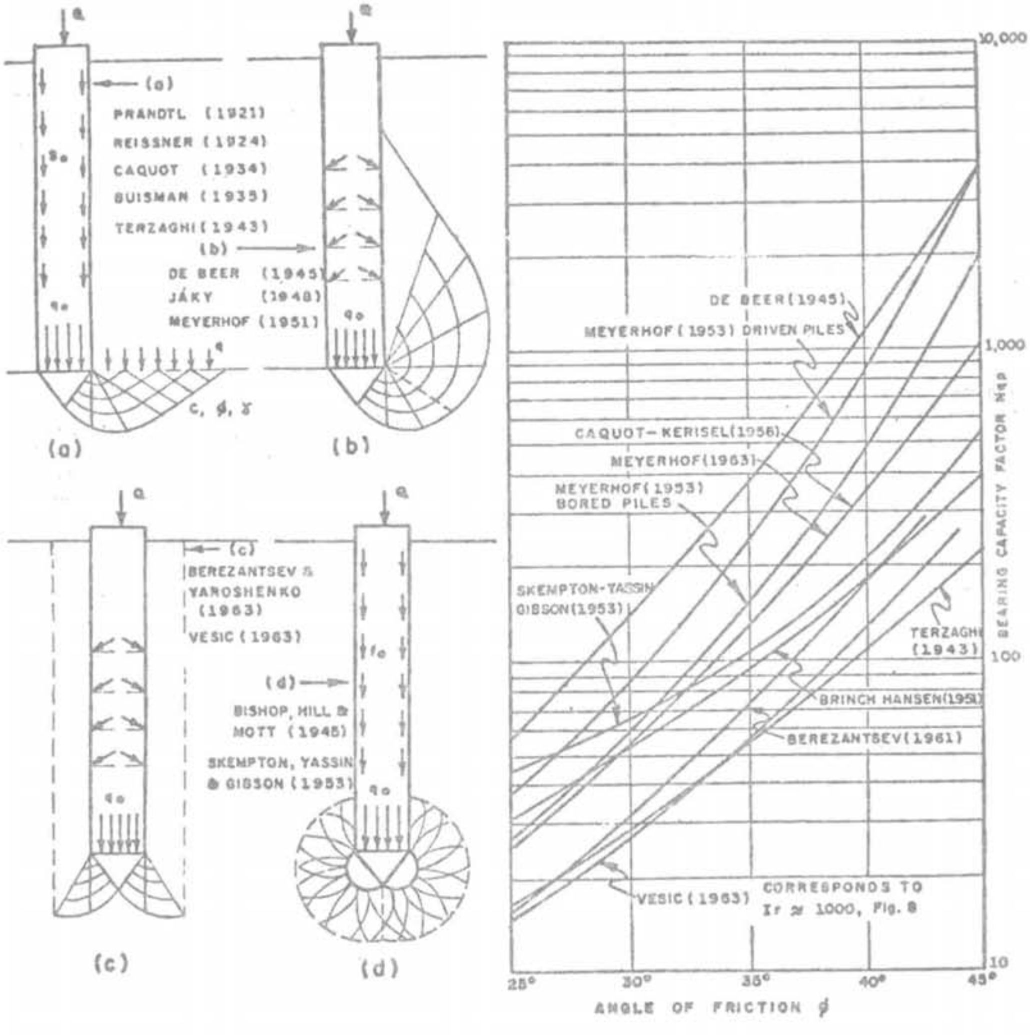


Fig. 7

Different failure patterns and bearing capacity factors for circular deep foundations (Apud Vesic 1965)

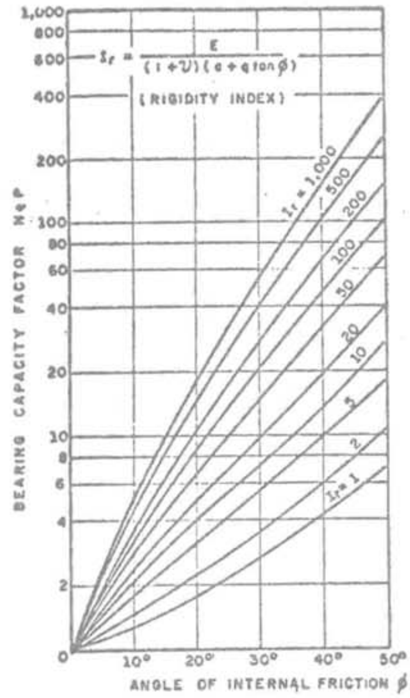
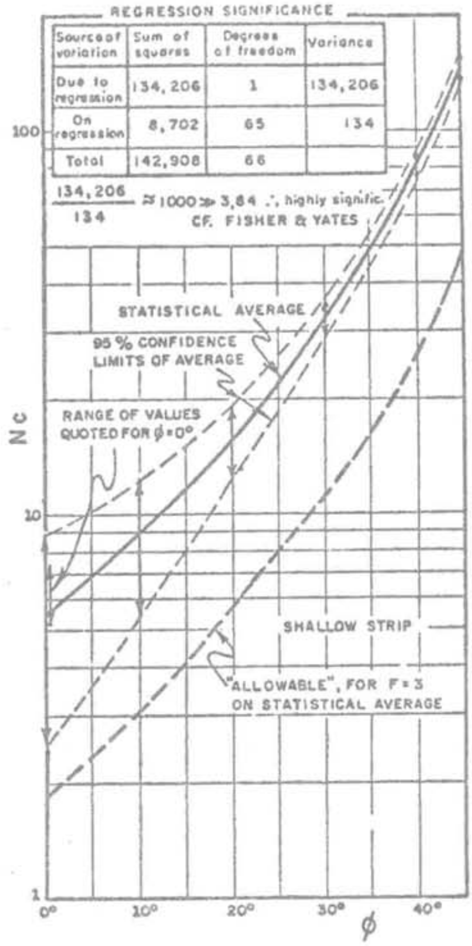


Fig. 8

Bearing capacity factor N_{qp} for a deep foundation in compressible solid (Apud Vesic 1965)

through rudimentary statistics, the levels of confidence implicit in any such choice.

It is certainly unwarranted to reproduce herein the tabulations and computations involved. The procedure followed was: firstly, through as complete as possible a reference study, a tabulation was prepared of all recommended numerical N values for ϕ values of 0°, 10°, 20°, 30°, 40° and 45°: next a statistical linear regression was computed for each separate N factor in comparison with the "reference equations" discussed below; finally, the 95 % confidence levels of the average value were computed. The results of these computations are summarized in Fig. 9, 10 and 11, for the simpler shallow foundation case.



RESULTS

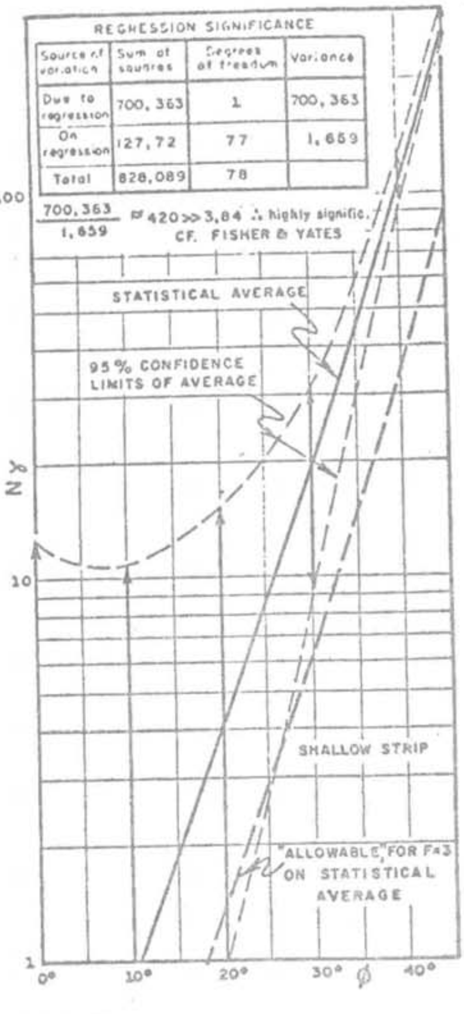
AVERAGE $N_{cA} = -0,0981 + (1,0896)\, N_c$

WHERE $N_c = (N_q - 1)\cot\phi \quad N_q = N_\phi\, e^{\pi\tan\phi}$

ϕ		0°	10°	20°	30°	40°	45°
N_c		5,14	8,34	14,8	30,1	75,3	134
N_{cA}		5,5	9,0	16,1	32,7	81,9	146
95 % conf. limit	max.	9,0	12,5	19,4	35,6	85,9	153
	min.	2,0	5,5	12,8	29,8	77,9	139

Fig. 9

Statistical average relation for Nc, in the form of Prandtl-Reissner et al.: shallow strip foundation



Source of variation	Sum of squares	Degrees of freedom	Variance
Due to regression	700,363	1	700,363
On regression	127,72	77	1,659
Total	828,089	78	

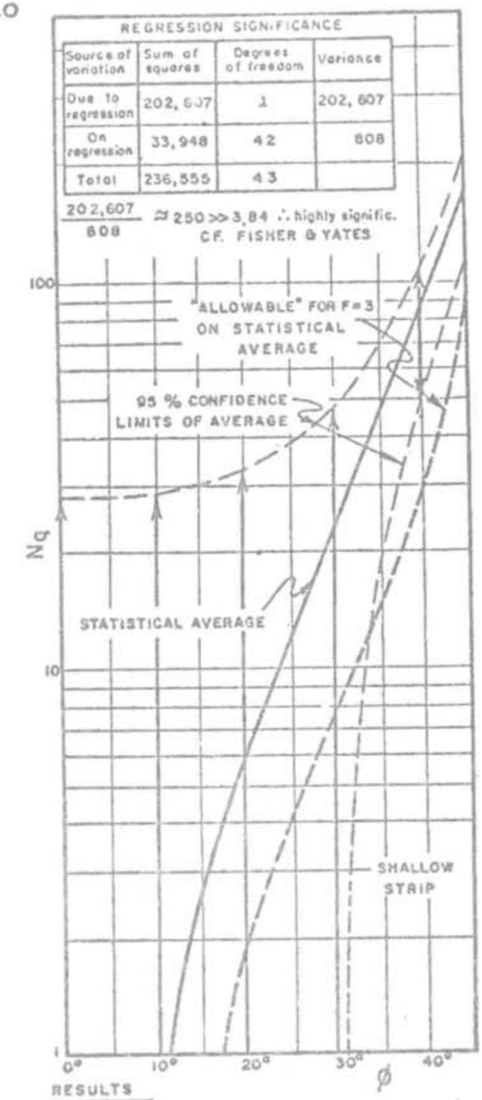
RESULTS

AVERAGE $N\gamma_A = -1,47 + 2,06\,(Nq - 1)\tan\phi$

WHERE $Nq = N_\phi\, e^{\pi \tan\phi}$: B.H. = BRINCH HANSEN 1961

ϕ	0°	10°	20°	30°	40°	45°
$N\gamma$ B.H.	0	0,47	3,54	18,1	95,4	241
$N\gamma_A$	-1,5	-0,9	4,1	19,3	107,8	278
95 % conf. level max.	12,6	10,5	15,1	29,7	118,3	297
95 % conf. level min.	-15,6	-12,3	-6,9	8,9	97,3	254

Fig. 10
Statistical average relation for N_γ, in the form of Brinch Hansen's (1961): shallow strip footing



Source of variation	Sum of squares	Degrees of freedom	Variance
Due to regression	202,607	1	202,607
On regression	33,948	42	808
Total	236,555	43	

RESULTS

AVERAGE $Nq\,A = -3,17 + 1,43\ Nq$

WHERE $Nq = N\phi\, e^{\pi \tan\phi}$

ϕ	0°	10°	20°	30°	40°	45°
Nq	1,0	2,47	6,40	18,4	64,2	135
Nq A	-1,7	0,4	6,0	23,3	89,0	190
95 % conf. level max.	27,9	29,4	33,4	47,0	122	284
95 % conf. level min.	-31,3	-23,6	-21,4	-0,4	56	117

Fig. 11 – Statistical aver. relation for N_q, in the form of Prandtl -Reissner

As regards the selection of the "reference equations" it was reasoned that it would be acceptable to employ the Prandtl - -Reissner basic equations for Nc and Nq (shallow), as has been done in both the more recent simplified recommendations (Brinch Hansen 1961, Meyerhof 1963); and that for the N_γ factor, by analogy, it might be acceptable to use the Brinch Hansen 1961 equation adapted to $N_\gamma = \alpha(N_q - 1) \tan \phi$. In part this reasoning was based on the impression that all the formulae constituted variations on the original Prandtl theme, with insufficient evidence to support the preferences on the modifying factors separately introduced.

The results were sufficiently revealing, and of a high enough level of statistical significance, to permit restricting the analysis to this rudimentary degree. Second-order regressions may suggest themselves as a next move, if desired or justified.

The analysis of the results reveals quite clearly the areas within which the Prandtl theory model has led to wider ranges of disagreement. It must be noted that the Nq regression would become meaningless if "deep foundation" and "shallow foundation" formulae were considered together: the result presented refers to shallow foundation formulae alone, and, as may be seen, even with this limitation the levels of confidence are much wider than for the N_γ and Nc cases.

Some simple practical conclusions that derive from the figures and statistical analyses may be summarized as: (a) the efforts spent with respect to better definition of values of Nc have basically resulted in an average increase in its value by 9 % in comparison with the Prandtl-Reissner solution; (b) the average result for N_γ solutions proposed would indicate $N_\gamma = 2.06\ (N_q - 1) \tan \phi$, which represents but a 14 % increase with respect to Brinch Hansen's (1961) recommendation; (c) a noticeable discrepancy occurs with respect to the Nq values, since on an average the solutions proposed tend to indicate a 43 % increase in comparison with the Prandtl-Reissner solution, an indication that merits questioning not merely by itself, but also in connection with the postulated basic interrelation between the Nc and Nq factors in the Prandtl-Reissner solution; (d) the levels of significance of the regressions assumed decrease noticeably from Nc to N_γ to Nq, and in the latter two cases the range of dispersion is considerably wider.

As an indication of practical interest, in each of the Figures 9, 10, 11 curves have also been plotted of $\frac{1}{3}$ the respective statistical average N value. Since frequently a factor of safety F = 3 is employed in foundation design, it would be of interest to represent how far the lack of "correct" N values might affect the safety of foundations (assuming that the average applicable shear strength parameters are not in question at the moment). At least as far as formulae are concerned, the critical areas of decision arise for Nq values for $\phi < 30°$, and for N_γ values for $\phi < 20°$. It must be reminded, however that the value of this preliminary appraisal is very limited, since it concerns only the relative comparison among different formulae.

2.4.3 Estimate of the field of practical interest and available confirmatory evidence

Recent advances in the use of statistical interpretation of data, and of computer solutions to formulate theories to fit the observed facts from a statistical universe, permit one to foresee that the Prandtl-model approaches to bearing capacity problems may shortly be superseded by entirely new formulations. There is, therefore, some interest in examining candidly, from the perspective of a stranger to the field of Soil Mechanics' earnest and hitherto fruitful endeavours, what are the probable boundaries of the statistical universe of bearing capacities that concern the foundation engineer, and what would be the most recommended delineation of a sampling program to represent it most efficiently, to desired levels of confidence.

In a very simple approach to the problem, assuming that a general bearing failure is at stake, Fig. 12 was prepared for the intended analysis. Setting aside many second-order factors, and adopting the general shallow footing bearing capacity formula with the Brinch Hansen 1961 N factors, the principal parameters at play were limited to the c and ϕ values, and the dimensions D and B. For necessary simplification single values of $\gamma = 1{,}6$ t/m^3, and D = 1 m were applied. Thus the principal independent parameters were

① ~ 12 LOAD TESTS; 1 FAILURE

② $\gtrsim$ 75 LOAD TESTS; 3 FAILURES

③ $\gtrsim$ 10 LOAD TESTS; 5 FAILURES

• LOAD TEST DATA, σ_f t/m² (MILOVIC 1965)

Fig. 12

Field of interest in bearing capacities and available confirmatory evidence

reduced to the following, with the ranges of values applied to each: $0 \leq c \leq 20$ t/m²; $0 \leq \phi \leq 40°$; $0,3 \leq B \leq 20$ m. Finally, for the further simplification of graphical presentation, the corresponding computed bearing capacity values are plotted in Fig. 12, only for B = 2, thus setting aside the somewhat smaller-order variations due to B, in drawing the curves of equal σ_f.

In order to facilitate the statistical analysis, the parameters of the two axes should preferably be chosen so as to reduce the set of σ_f curves as nearly as possible to parallel straight lines: this has not been achieved, but it was not pursued because, with the graphs as they are, the conclusions are obvious to any engineer. The concentration of field and laboratory test evidence, plotted in the Figure, merely describes the extremities of the two boundaries (c = 0, and ϕ = 0) of the field of interest. Very few data have been located that fall within the area of more general interest. As a result, it was concluded that until a number of additional points are contributed for plotting approximately in a grid pattern within the general area, at present it is not possible to subject the problem to statistical analysis for suggesting the testing program that would be of greater interest or efficiency towards revising the present theory.

When such a formulation becomes more fruitful, the program would be established with a view to raising the levels of confidence of determination of σ_f. Moreover, if once again the use of F = 3 be interpreted as a likely present lower 95 % confidence level, the programming of practical interest to foundation engineering practise would be such as would raise the respective bearing value, by raising this lower confidence level.

2.4.4 Some basic problems associated with Prandtl theory and Bearing Capacity Formulae

In any attempt to examine the perspectives of future development along a given line of thinking, it is necessary to revert in some measure to the basic premises, which are generally forgotten during more routine application of its results. For instance, whereas it has become routine to apply Plasticity Theory as if it were an absolute law, it would doubtless be edifying to examine the confidence limits of the very data which prove its validity, in order to check how far it might be worth refining conclusions based on the theory. An analogy is furnished by the meticulous demonstrations by Bishop (1966) for the case of another implicitly accepted law that "The experimental results, however, strongly support the Mohr-Coulomb criterion, and we must, I feel, accept the Mohr-Coulomb criterion as being the only simple criterion of reasonable generality. It does, however, underestimate the value of ϕ' for plane strain in dense sands by up to 4° ...".

Thus the use of the Theory of Plasticity to bearing capacity may gain from specific research on some of the basic premises, among which the following are cited as examples.

Firstly concerning the stress-strain behaviour, it must be noted that according to the methodology of the theory of Plasticity, up to the present the theoretical solutions have been worked out for rigid-plastic behavior, whereas most soils would require a more realistic treatment as elasto-plastic, with strain-hardening or strain-softening, as the minimum adequate form of representation. The use of computational models, as exemplified by Whitman and Hoeg (1965) and others, opens new vistas along this line. Indeed, mention has already been made of Ladanyi's (1967) demonstration of the effect of a strain-softening behavior of sensitive clays on the Nc value of deep foundations in clays.

Moreover, it may be noted that time effects are not considered, neither are evidences of hysteresis in stress-strain behavior. Also it is accepted from the theory that in plastic frictional materials every process of plastic deformation is accompanied by an increase in volume, which doubtless raises questions on the acceptability of the model (Drucker 1953). Also, the theory has been systematically applied to soils under the assumption of the Linear Theory of Deformation, i.e. considering that the geometry of the mass will not suffer changes during the deformation process. Finally, from the theory it is demonstrated that the medium can only undergo movements of rotation or translation as a rigid body when the sliding surface and the failure lines are straight lines or log spirals, $r = r_0 \, e^{\theta \tan \phi}$, wherein the log spiral becomes circular only in a purely cohesive material(Badillo and Ricó 1967).

A second problem that is raised concerns the choice of the applicable "flow criterion" through the Mohr-Coulomb equation $s = c + \sigma \tan \phi$. As is well-komwon, if this equation is expressed in terms of effective stresses, involving the true angle of internal friction ϕ', the bearing capacity analysis should be conducted with full cognizance of pore pressures. If, however, the pore pressures immediately under a loaded footing are not known, as is most generally the case, the total stress analysis coupled with some undrained strength parameters c_u and ϕ_u will have to be employed, and a problem arises with respect to the value applicable. Stated in general terms there may be three different values at play within a bearing capacity formula, even if we exclude from consideration the gradually curving envelopes that generally obtain in unsaturated soils, and that are traditionally substituted by straight lines according to the range of normal stresses faced by the problem at hand.

Firstly there is the well-established fact that the geometry of failure planes is determined by the true angle of internal friction: therefore, since the bearing capacity factors are determined through stability analyses of failure masses established according to such a geometry, a discrepancy may arise inasfar as by application of the ϕ_u value, the N factors will be associated with a failure surface somewhat different from that which should be the real one. This problem has been discussed by Davis 1956, under a set of necessarily simplifying assumptions, some of which pertaining to Elastic Theory constants, together with Skempton's A and B coefficients assumed to be constants at failure, and it is concluded that "In spite of the fact that c_u and ϕ_u are not the true parameters c' and ϕ', an undrained stability analysis will always give the correct answer in terms of a factor of safety or the magnitude of the load causing failure, provided all the other assumptions with regard to shape of failure plane, homogeneity and isotropy of the soil mass, absence of consolidation, etc... are reasonable, and provided the undrained strength envelope is a straight line over the range of stress of the problem. It will, however, give an incorrect pattern of slip lines or position of the failure plane" (loc. cit. p. 170). The assumptions and provisos raise some questions. The same subject is treated by Bishop and Bjerrum (1960). By analysing a simple idealized case of the stability of a vertical cut, and using Rankine active states of stress, the authors prove that the factor of safety $F = 1$ coincides for both Total and Effective stress methods of analysis, each, however, with its mathematically determined failure plane; i.e. the failure plane for the $\phi_u = 0$ method minimized to $F = 1$ results on a 45° plane, whereas the failure plane for the equivalent effective stress condition of $c' = 0$ and $\phi' \neq 0$, analogously minimized, results in a $45 + \phi'/2$ plane and the same $F = 1$. The conclusions are thus: "Firstly, both total and effective stress methods of stability analysis will agree in giving a factor of safety of 1 for a soil mass brought into limiting equilibrium by a change of stress under undrained conditions. Secondly, although the values of a factor of safety are the same, the position of the rupture surface is found to depend on the value of ϕ used in the analysis. The closer this value approximates to the true angle of internal friction, the more realistic is the position of the failure surface ..." (loc. cit. p. 46). The authors therefore caution with respect to the use of total stress methods in over-consolidated clays in which the pore pressure shows a marked drop during the latter stages of shear, because the total stress method implicitly uses a value of pore pressure related to the pore pressure at failure in the undrained test.

The second manner in which it may be visualized that additional different values of ϕ are simultaneously at play in a general bearing capacity problem concerns the interpretation of ϕ as a $d\tau_f/d\sigma$. In a general way there are two conditions of $d\sigma$ at play within a failure surface under a footing: the first concerns the $d(\gamma z)$ corresponding to overburden pressures, and the second concerns the dq of the surcharges and the very footing pressure applied, causing inevitable changes of normal stresses along the failure surface concomitant with the shearing stresses that lead to the failure hypothesized. Classically the $d\tau_f/d(\gamma z)$ has been disregarded by the use of a constant "weighted average" c value as suggested by Skempton (1951), and as above discussed under item 2.3.3. However, this ϕ value, which is that corresponding to consolidated-undrained shear tests, or as more recently preferred, the c/p ratios better defined by in-situ vane tests, comes into play whenever the size of the loaded area forces a deep failure surface, the geometry and stability of which would obviously be affected by the fact that strengths increase with depth; for instance, shallower failure surfaces should prevail. As was mentioned under item 2.3.3 this factor has received attention from Raymond (1967), more recently. Moreover, classically the $d\tau_f/dq$ has obviously been taken for saturated clays as the $\phi_u = 0$. In unsaturated clays where $\phi_u > 0$ even for a rapid loading, this value would be defined by appropriate unconsolidated-undrained UU tests: but even in saturated clays some degree of dissipation of pore pressures during construction could account for a $\phi_u > 0$. The "Influence of Construction Time on Pore Pressure Dissipation Beneath Foundations" was recently treated by Lumb (1965), but the subject remains complex.

The choice of a single ϕ value most applicable to a given problem treated by the total stress method depends on the relative participations of the several factors mentioned. For small footings on stiffer clays the applied pressures transmit stresses so much higher than the in-situ initial consolidation pressures, that the prevailing undrained ϕ value will be given by UU tests (assuming no drainage) because of a prevalence of the $d\tau_f/dq$ factor. For large areas on soft clays, with light loadings, for analyses by bearing capacity factors the prevailing undrained ϕ will more closely approximate that of the $d\tau_f/d(\gamma z)$ factor. The relative proportion of the interference of these factors may best be estimated by plotting the presumed stress paths of representative soil elements.

Finally, since some of the reasons for the development of new analyses in the past have been connected with hypotheses regarding the development of friction below the base of the footing, it may be noted that no field evidence has yet been obtained on this point, in which both adhesion and friction will be at play, the latter depending on pressure distributions which in turn depend on the structural rigidity of the footing and multiple other factors (e.g. Meyerhof 1963, Szechy 1965). The theoretical developments should aim at allowing independent c_1 and ϕ_1 parameters (or correction factors for the basic c, ϕ soil parameters) at the base of the footings because of execution effects analogous to those postulated for piers and piles in London Clay on the basis of the corresponding field evidence.

2.4.5 Bearing capacity solutions for special cases

Many special cases that the practising foundation engineer occasionally faces have merited special derivations within the bearing capacity formulae above discussed. Although within the tone set for this general report, in comparison with the dispersions already mentioned most of such cases would appear to constitute second-order or subsequent-stage effects as regards present-day ability to estimate bearing capacities, the value of such studies for comparison with the earlier simplified models certainly cannot be denied.

(a) Shape factors, for square, rectangular and circular foundations as compared with the strip foundation, have already been mentioned. Indications on the subject have been furnished by Terzaghi (1943), Meyerhof (1951, 1963), Skempton (1951), Mizuno (1953), Brinch Hansen (1961, 1966), and de Beer (1965).

(b) Inclination factors, for the case of inclined loads on shallow and deep foundations have been recommended by Meyerhof (1953), Sokolovski (1960), Brinch Hansen (1961), Shiraishi (1964), Jappelli (1967), and Jappelli and Tortorici (1967), among others.

(c) Equivalent and effective areas

If loaded areas are not of the standard shapes in plan, it is customary to transform them first into an "equivalent" rectangle of the same area and center of gravity, with the main axes coinciding, and with the same "ratio of maximum to minimum plastic section modulus (= L:B)" (Brinch Hansen 1961).

Moreover, the procedure for handling eccentric loadings has been recommended by Meyerhof (1953) and Brinch Hansen (1961), among others.

(d) Influence of adjacent footings

The subject has been investigated by model tests by Habib and Toheng (1960), with a subsequent publication also by L'Herminier et al (1961). Mandel (1963) studied the problem theoretically. It is concluded that the increase of bearing capacity is negligible in $\phi = 0$ cases, although it is very significant in sands.

(e) Bearing capacity of footings on slopes

The subject has been studied successively by Meyerhof (1953), Janbu (1957), Mizuno (1960), Kedzi (1961), Zaharescu (1961), Brinch Hansen (discussion, 1961), Jappelli (1967) and Jappelli and Tortorici (1967).

(f) Shape of the contact surface of the foundation.

Szechy (1965) investigates the influence of convex and concave flat contact surfaces of footings on the bearing capacity, and confirms the conclusions by tests on sands.

(g) Influence of layered deposits

This problem, which appears to be of more practical interest than the foregoing, has been repeatedly studied. The following sequence of contributions attests to the relevance of the topic; Button (1953), L'Herminier (1956), Milovic (1963), Kenney (1964), Yamaguchi (1967), Reddy and Srinivasan (1967), and Yokowo et al (1968), among others.

(h) Interference of an underlying shallow rock horizon

This study, concomitant with that of the influence of a limited thickness of the layer of soft clay, originally considered by Jurgenson (1934), was later treated by Mayer and Habib (1951), Habib abd Suklje (1954), Suklje (1954) and Vyalov (1967).

2.5 Bearing capacity of individual piles and piers

2.5.1 Point and skin resistance for piles

In an attempt to report first on what appear to be the most current present-day practices in foundation engineering it was mentioned, under item 2.2.2 (d) and through Fig. 4 that Meyerhof's deep foundation solution is extensively applied for the computation of point bearing capacity through the factors Ncp and Nqp as plotted. Moreover, Skempton's recommended value Ncp = 9 for point bearing capacity in clays ($\phi = 0$) was also established as of widespread acceptance.

However, under item 2.4.1 some questions were raised, although pertaining more to general bearing capacity problems. There are obvious conditions that make the bearing capacity problems of piers and piles a somewhat special class of problem: suffice it to say that there is the joint contribution of Lateral or Skin Friction, and Point Bearing, that the former frequently constitutes the principal contribution, and that even for the point bearing contribution it is a somewhat meaningless abstraction to imagine a deep plate without taking into account how it got there and what the respective consequences might be of the so-called execution effects. Thus it behoves us to examine in somewhat closer detail the present status of bearing capacity problems of piers and piles. After discussing the subject in the light of the general accumulation of data, the case of London Clay investigations will be treated separately in view of the vast amount of information and of some special problems that appear to obtain in that individual case.

(a) Point resistance

Regarding point resistance in clays it has already been mentioned that the factor Ncp = 9 is not general. Ladanyi's work (1967) with respect to the influence of the strain--softening stress-strain curves has already been mentioned as capable of reducing Ncp to as low a value as 4 for extremely sensitive clays. In an earlier study (Ladanyi 1963) based on the theory of the expansion of a cavity (both spherical and cylindrical), and using a numerical integration method to apply as closely as possible the real stress-strain curves of the clay, he had already proved that Skempton's (1951) expression for the deep circular foundation

$$N_{cp} = \frac{4}{3} \ln \left[\frac{E_s}{\tau_{f(uu)}} + 1 \right] + 1$$

based on the Secant modulus of Elasticity E_s gave values from 5 to 15 % higher than those newly derived, such values ranging from about 5 for $E_s/\tau_{f(uu)}$ = 25 to about 9.2 for $E_s/\tau_{f(uu)}$ = 550. Meyerhof (1964) in his discussion therefore summarises that since his own rigid-plastic analysis had indicated Ncp = 9.3, simplified recommendations could read that the Ncp value should be corrected by a reduction factor varying from about 0.5 for $E_s/\tau_{f(uu)}$ = 25 to 1.0 for $E_s/\tau_{f(uu)} \approx 500$.

The model test values 5 < Ncp < 8 reported by Sowers 1961 may thus have been associated with stress-strain problems. Not many field measurements are available to check the deep Ncp for clays, besides the London Clays. Kerisel (1965) states that values higher than 9 have been reported, although none higher than 20. Bjerrum (1954) reports 4 load tests on 0.25 m diameter plates at 2.5 m depth in a clay of $S_t \approx 80$ and $\tau_f \approx 1.2$ t/m^2: theoretical values were checked within 2 %, the strain at failure being remarkably small (0.025 B). Ballisager (1959) also claims agreement with the general formula, for the Aarhus Septarian Clay, using indirect calculations from pile load tests. Dinesh Mohan (1961) reports values for expansive clays wherein comparing with the vane shear strengths of 14 to 18 t/m^2 the resulting Ncp values are 8.2, 5.7, 7.8 and 7.2. Nuñez, et al (1967) report estimated point bearing capacities more in accordance with drained test parameters (c', ϕ') using the multi-stage triaxial tests and Brinch Hansen's (1961) bearing capacity factors. Regarding the point bearing capacity factor Nqp for circular deep foundations in sands, Fig. 7 transcribed from Vesic (1965) illustrates the already mentioned degree of discrepancy in opinions. Herein, however, it is felt that the adoption of suitable design recommendations cannot reasonably be molded to a concept of dispersions within a statistical universe. The new perspectives opened by the prototype-scale model tests at the Chevreuse Station (Kerisel 1961, 1964) and at the Georgia Institute of Technology (Vesic 1963, 1965) showed the important interference of unclarified scale effects.

The more apparent problem seemed to be the derivation of the suitable bearing capacity factor, appropriate adaptations being made to compare driven and buried foundations, in accordance with correction factors for the respective "execution effects". Notwithstanding Meyerhof's (1963) recommendation, and a recent indication by Broms (1966) in favour of the Berezantsev et al. (1961) factors, it is felt that at present Vesic's own values (Fig. 8) must be singled out for preference, because they permit consideration of a newly-discovered, undeniable, non-random factor as the "rigidity index" of the soil, to be separately determined or estimated.

Such a parameter would presumably have as marked an influence as was observed in sands, on the general (c, ϕ) soils, frequently more deformable than the pure sands.

However, the real problem revealed by the tests mentioned was the fact that the point resistance increased quasi-proportionally with depth only at relatively shallow depths, while at greater depths, generally exceeding about 10 to 20 foundation diameters, this resistance reached asymptotically almost constant final values. Thus, the main problem in determining the point resistance shifted to one of establishing the stress field due to overburden stresses that can be considered effective. Vesic achieves the desired compatibility between observed and theoretical bearing values by postulating that "the stress field around a deep foundation resembles that observed in a silo or, generally, in a mass of soil above a yielding horizontal support - ("arching")", as schematically shown on Fig. 13, reproduced from Vesic (1965). The critical depth, beyond which no further increase in bearing capacity obtains in a homogeneous soil is shorter in a loose material than in a dense one.

Quite apart from the discussions on applicable values of the Nqp factor, therefore, it appears important to recognise that "because the load transfer is expected near the pile, the vertical pressure must not be equal to the overburden pressure as γz " (Nishida 1963). Thus a fundamental fallacy lies in the fact that in computing

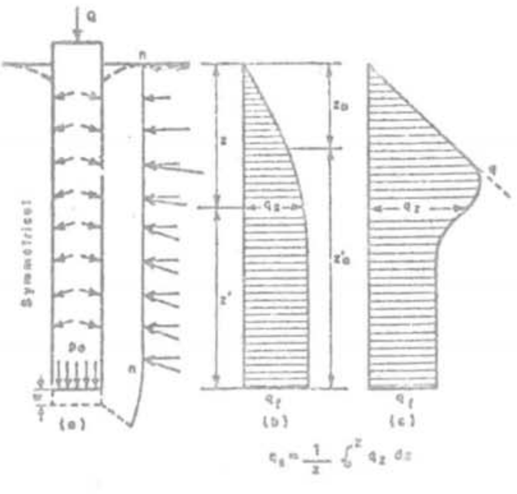


Fig. 13

Stress distribution around a deep foundation in sand (apud Vesic 1965)

total bearing capacity of piers and piles Meyerhof's(1951) recommendation "To the base or point resistance of a foundation with a rough shaft must be added the skin friction, to obtain the total bearing capacity", has been generally followed by considering the overburden stress, varying linearly with depth, as fully effective both for the point resistance and for the skin friction.

The execution effects of piles such as the enlarged-base Franki pile will obviously alter the above simplified picture very considerably.

(b) Skin resistance.

The total bearing capacity of a pile is traditionally subdivided into the components

$$Q = (c\ N_{cp} + q\ N_{qp})A_b \ + f_s A_s$$

where

A_b = area of base; A_s = surface area of shaft; f_s = unit skin friction; and other symbols as before. In homogeneous soils the skin resistance is often the principal component. In order to account for execution effects as well as for any implicit incompatibilities in the simultaneous stress-strain development of the two components, it has been customary merely to include a correction factor to compare the skin friction values f_s with the appropriate shear strength values of the contiguous soil element: thus $f_s = \alpha\tau_f$. Values of α have been determined experimentally by various means. The skin friction "can be estimated for piles driven in sands by using an earth pressure coefficient of about 0.5 to 1.0 according to the relative density of the material" (Meyerhof 1951, 1963): that is, traditionally the diagram has been assumed to be linearly increasing with depth due to overburden pressure.

Considerable theoretical and experimental evidence has come to light, however, to alter this simplified picture quite radically. For instance, the large-scale model tests above mentioned (e.g. Vesic 1965) have shown that the skin resistance in sands follows the same trend mentioned for the point resistance, i.e. reaching asymptotically almost constant final values with increase of pile length after an early quasi-proportional increase with depth up to about (10 - 20) diameters. Moreover, the displacements needed to reach the ultimate skin resistance are independent of the diameter, whereas those required to develop the ultimate base resistance are roughly proportional to size. Finally, during the progress of a typical load test not only do the proportions of overall skin resistance to point resistance gradually decrease because of the factor above mentioned, but also the distribution of skin friction along the pile length changes appreciably.

All of these facts have suggested the need to formulate better approaches to the skin friction problem, substituting for the admittedly rudimentary one earlier mentioned.

The discussion will first be concentrated on saturated clays, in which the adhesion coefficient β has been traditionally defined by the overall ratio of the ultimate lateral resistance (under rapid loading) to the product of the lateral surface A_s with the undrained strength τ_f. Naturally, in this respect, as in those formerly discussed, an important item concerns the appropriate definition of τ_f. For purposes of research on the real distribution of stresses along a pile, several distinct test conditions have been employed, besides the undisturbed value (2.3.1 and 2.3.2), including the totally remolded condition, and the condition reconsolidated to overburden pressures after total remolding. However, for purposes of design, preference is given to routine reference to the presumed in-situ undrained strengths which would be available to the designer: this decision implies, however, throwing the greater load of dispersions on the "execution effects" due to sensitivity, swelling, fissuring, etc..

With the β factor defined as above, the subject decomposes into three fundamental parts: the type of contact surface; the execution effects; and the variation of strains along the pile which earlier had fallaciously been taken as implicitly rigid. If the three factors are $\beta 1$, $\beta 2$ and β_3, the β factor will result from their product.

With regard to the nature of construction material and the consequent skin friction a Table of values is furnished by Potyondy (1961). Values range from about 0.5 for smooth steel on clay, to about 1.0 for rough concrete, along foreseeable lines. The author furnishes values for sand and clay, and also for a "cohesionless silt" and a "sand-clay mixture": the latter two cannot define any general trend, but serve to alert the profession to the fact that the general (c, ϕ) soils may yield significantly different results, that require investigation. Meyerhof (1963) refers to model tests yielding ratios of about 0.6 to 0.8 for steel, 0.7 to 0.9 for wood, and 0.8 to 1.0 for concrete.

As regards the influence of the variation of strains from top to bottom of the pile, and along the pile as the load increases, some interesting results are available from special load tests that will be discussed separately. Obviously if the skin friction develops to its peak with very small strains (e.g. Cooke and Whitaker 1961, et. al.) it cannot be expected that the peak adhesion value can simultaneously develop along the entire pile, subjected to elastic compression as part of the top settlement. Thus, for instance, by Theory of Elasticity derivations it can be shown that the average overall adhesion will not be more than about 50 % of the peak (e.g. Nishida 1953, et al.).

Estimates of β values have been published based on a subtraction of the 9c point-bearing load from the ultimate values of load tests, and a comparison with undrained strengths. Meyerhof (1951) suggested $\beta = 0.5$ is sensitive clays shortly after driving of concrete, timber or steel piles, and $\beta = 1$ for insensitive clays; the τ_f is calculated for an earth pressure coefficient $K = 1$, with an upper limit of 1 kg/cm^2 as the maximum until then observed. Rodin and Tomlinson (1952) report on 21 bored piles and 21 driven piles, in undisturbed clays of undrained strengths (UC and UU) between 5 and 11 t/m^2 and completely remolded strengths of 1.5 to 2.5 t/m^2: for bored piles β is conservatively indicated as 0.4 for soft to medium clays, decreasing to 0.25 for stiff clays; for driven piles $\beta = 1.00$ for soft clays decreasing to 0.25 for very stiff clays. Tomlinson (1957) again, analysing data from 56 steel, concrete and wood piles confirms the $\beta = 1$ for soft clays, and a general decrease, with increase of clay consistency, to $\beta = 0.2$ as an average for all piles (while $\beta = 0.4$ prevails under similar conditions for concrete piles) in stiff clays; an absolute limit of overall adhesion of 4 t/m^2 is recommended. Bergfelt (1957) summarizing the data from hundreds of piles driven in the soft Gothenburg clays confirms $\beta = 1.0$. Vey (1957) furnished data on a steel H pile driven through medium clay concluding that "frictional resistance of soil along the pile can be computed fairly accurately from UC strengths in soft to medium clay". Zeevaert (1957) reporting on results of field investigations on driven friction piles in Mexico City clay considers that the shaft bearing capacity is given by the lesser one of the two conditions: either a skin friction of perfectly remoulded and reconsolidated clay at the face of the shaft, corresponding to a $\tau_f = 0.53\ K_o\ p_v$ where $K_o = 0.75$ was recommended; or a failure at a distance of about 1.1 r from the center of the pile, with a semi-disturbed clay of $\tau_f = 0.3$ qu. Rodriguez and Rosenblueth (1959) reporting on "driven" model piles in undisturbed Mexico City clay, using pull-out tests to evaluate the adhesion, concluded that β is of the order of 0.67, with relatively small variations due to roughness of pile surface and type of pile. Nordlund (1959) reporting on driven concrete piles, subtracting point bearing as 9c concludes that $\beta = 0.3$ in stiff clays ($\tau_{f(uc)} \simeq 10\ t/m^2$) of maximum sensitivity 1.2. Mohan and Chandra (1961) reporting on cast-in-situ bored piles in expansive clays report that the adhesion was the same for compression and pull-out tests and conclude that the average $\beta = 0.5$. Woodward and Boitano (1961) interpret a series of load tests on piles through sands and clays by subtracting the skin resistance of the sand strata based on SPT blowcounts (Meyerhof 1956), and subtracting the 9c point-bearing value, obtain β

values that are roughly 20 % higher than Tomlinson's curve, for clays of τ_f (UC) between 5 and 18 t/m^2. Resendiz (1964) in a series of special load tests on long precast concrete piles driven into undisturbed Mexico City clay of τ_f (UU) $\approx$ 3 t/m^2 and St $\approx$ 2.5, reports $\beta \simeq$ 1.0 after due thixotropic strength regain, in about 60 days. Finally, Lo and Stermac (1964) report on three timber and one Franki piles in a stiff clay of $\tau_f \approx$ 7.5 t/m^2, documented with vane shear, UC and UU tests (which serve to emphasize the relative nature of some of the earlier data ... cf. item 2.3): by subtracting the point bearing in an underlying silt, and the friction in an upper layer of sand (both estimated on the basis of SPT blowcounts through Meyerhof's, 1956, simplified indications) it is concluded that β = 1 as regards the vane strengths. In the closing discussion to the paper they further report on two steel tube piles in which β = 0.35 (severe whipping during driving was observed).

Obviously the relative value of all the above information results in a fairly low confidence level of design decisions based thereon, because (a) of dispersions evident in the very results of the analyses as published (e.g Tomlinson 1957 and Woodward & Boitano 1961, et al); (b) the nature of the original data and its probable dispersion; (c) the lumped average value for the pile length, despite variations of soils traversed, and besides the variations on the component factors β_2 and β_3 already mentioned. Eide et al (1961) furnish a well documented example of the magnitude of the errors that can obtain from unsuspected factors. Notwithstanding the wide local dispersions, like all overall average indications, they will be very satisfactory while representing the same statistical universe, especially for long piles. Regarding soft clays the principal non-random factor that has not been adequately separated is the sensitivity S_t. Regarding the gap that is formed due to the whipping effects in medium and stiff clays, it may be recalled that the possibility of a self-healing of such a gap depends on the depth at which the clay occurs: any clay consistency corresponds to a certain preconsolidation pressure (roughly estimatable through the c/p ratio) and the rehealing possibilities depend roughly on the ratios of the effective overburden stresses and pile penetration stresses to such a preconsolidation pressure. A medium clay that will not heal at a few meters of depth will be subject not only to much smaller whipping gaps at greater depths but also, concomitantly, to much easier self-healing.

Summarizing these rough indications which must be subject to a considerable latitude of judgement, Kerisel's (1965) recommended curve for the long-term β value is herewith reproduced in Fig. 14.

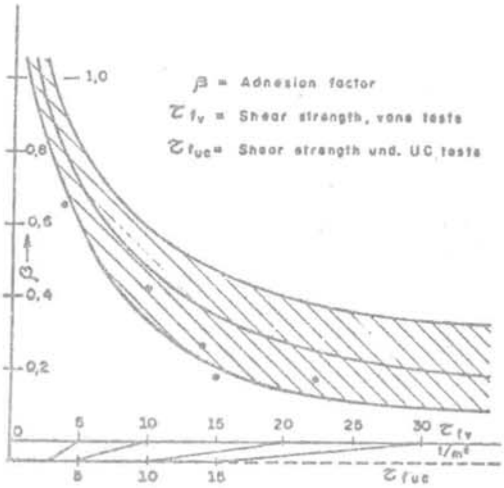


Fig. 14

Recommended adhesion factors for driven and bored piles in clays (Apud Kerisel 1965)

For a general treatment of the subject of skin friction on piles in (c, φ) soils the reader must be referred to the recent Bulletin 25 of the Danish Geotechnical Institute, by Brinch Hansen and Mazurkiewicz (1968), which constitutes a state-of-the-art appraisal on the complex subject. Some of its conclusions, confirmed hitherto principally with respect to sands, will be mentioned in connection with the negative skin friction (item 4.4) to which it devotes most attention. Within the present item suffice it to mention that it is proved that skin frictions in pushing and in pulling a pile cannot be equal, a fact that had earlier been observed (e.g. Meyerhof 1951, et al.) but which has been frequently overlooked in attempts to establish skin friction values from field tests. Moreover, some of the execution effects that interfere very significantly in this problem, will be further discussed unter item 4.3.

2.5.2 Analysis and synthesis of load-stress distribution-settlement behaviour of single piles.

It is the hope and aim of every engineer faced with the design of pile foundations, to be able to synthesize a load-settlement diagram (if possible including the ultimate failure load prediction) for the individual pile on the basis of standard geo technical testing.

Setting aside for the moment the solutions based on the Static Penetrometer (item 2.6) which approaches the conception of a model test, we may distinguish between two principal lines of endeavour; one reverts to solutions based on the Theory of Elasticity, through Mindlin's formulae for the stresses caused by a point load within the idealized semi-infinite medium; the other basically attempts to establish the behaviour of the pile through a step integration of the effects of small increments of the pile, considered to compress elastically with the point loads received and transmitted from one to the next, with due subtraction of intermediate adhesion values.

The latter procedure, postulating that "the methods of elasticity are of limited assistance in this problem because the soil is nonelastic" (Seed and Reese 1957), necessarily relied heavily on measured values of stress distributions along piles, especially fitted with strain gages, and submitted to load tests. Setting aside the analogous laboratory model tests that contributed much to the subject, it appears that the principal contributions within this area derive directly from the field tests and publications by Reese and Seed 1955, Seed and Reese 1957, Mansur and Kaufman 1958, D'Appolonia and Hribar 1963, Seed and Reese 1964, Coyle and Reese 1966, and Koizumi and Ito 1967, besides indirect information on adhesion values.

The basic procedure is summarized in Fig. 15, extracted from Seed and Reese (1957, 1964). To begin with, for cases of low point resistance, a convenient approximation is introduced, on the basis of estimates, substituting the tip resistance by an added equivalent length of shaft. Thereupon a certain tip movement is assumed, and, working upwards, the load and settlement of the pile head are computed. The problem depends fundamentally on the assumptions with respect to β_z, and the

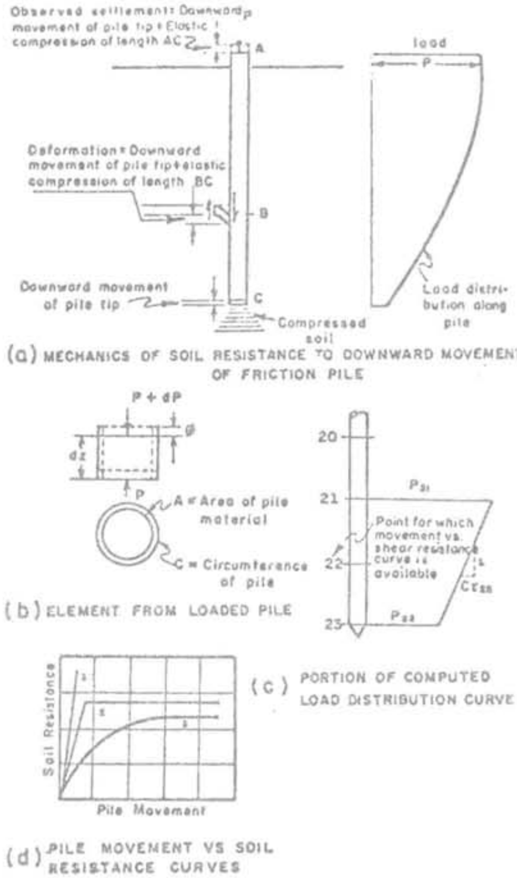


Fig. 15

Predicting the performance of friction piles in saturated clay (Seed & Reese, 1964)

stress-strain curve for its mobilization. For the value of β_z for instance they extract values from their test (1955) as shown

Depth ft	10	12	14	16	18	20
β_z	0	0.3	0.6	0.7	0.9	1.0

and, moreover, suggest "that the stress vs. deformation curves obtained from vane shear tests on the undisturbed soil be used .... The vane imposes deformations similar to those imposed by a downward movement of the pile, and the radial movement of a point on the vane extremity can be assumed to develop the same shear resistance as will an equal downward movement of a point on the pile" (1964). For improvement of the indications for practical applications it may be worthwhile establishing, from the available test data, statistical second-order regressions of how β_z varies not merely with depth but with the proportion of Q/Q_f : moreover, thereupon the step-numerical procedure may be substituted by integrations. The knowledge regarding anisotropy revealed by vane shear results may also condition the second basic recommendation.

Coyle and Reese (1966) further develop the solution, principally with respect to the applicable β_z values in order to make them compatible with "limiting" values of β observed (for instance, considering in stiff clays the limit of 5 t/m^2 of adhesion interpreted by Woodward and Boitano 1961). Another modification introduced refers to the computation of tip movements by reference to the settlement of loaded plates (Skempton 1951), which appears to constitute a definite improvement. As regards the first modification, however, notwithstanding the checks obtained of the load-settlement curve vs. observed values, it may be objected that available test data have not merely raised a wide margin of questions with respect to β values, but also at any rate serve to question any connection between the overall average β for piles at failure (item 5.2.1 b) and the β_z values at different depths and ratios of Q/Q_f .

The second main avenue of approach to the problem, through Mindlin's (1936) Theory of Elasticity solution, has received principal contributions from Nishida (1961, 1964, and some earlier), D'Appolonia et al. (1963, 1964, 1965), Salas and Belzunce (1965) and Poulos and Davis (1968).

Specifically for the problem of determination of load-stress distribution-settlement behaviour of single loaded piles it was applied initially by D'Appolonia and Romualdi (1963) with respect to two specially observed load tests of steel H piles driven through granular strata. This first paper was developed under the assumption that the tip did not move, and that "the pile is free to move within the soil" as within an elastic medium. Two subsequent papers (1965, 1964) considerably developed the solution, by recourse to computer methods. "Except for elastic-plastic deformations at the pile tip, it is assumed that soil deformations may be predicted by theories of elasciticy". The failure force at the pile tip is predicted by a deep foundation bearing capacity formula such as Berezantzev's et al (1961). The stresses at the pile-soil interface are assumed from elasticity theory, with a failure stress given by Coulomb's equation, wherein the cohesion and friction values given by Potyondy (1961) are introduced. The normal stress on the interface is taken as $\sigma_n = K\sigma_v$. Naturally, for Mindlin's equation the elastic constants must be known: assuming $\mu = 0.3$ best-fitting E values were calculated to adjust the computed load-settlement curve to load test results; the values obtained were quite reasonable for the material in question, so that the reverse process should be valid.

As stated by the authors "load transfer of friction forces from a pile to a soil can be determined for any pile load, less than ultimate" (1965) and "only four factors have a major effect on the accuracy of the computed results. These are: (1) the material properties of the soil, (2) the elastic -plastic tip movement, (3) the tip-punching force, and (4) the normal stress on the pile-soil interface." The major problem lies in the evaluation of this fourth factor. Nishida (1961) for instance furnishes derivations of "stresses around a compaction pile" underan analogous set of mathematical assumptions.

Despite the many criticisms that are inevitably made to this type of solution, it is noted that they tend to attenuate as one restricts application of the method to working loads: the great advantage of such a solution lies, in any case, in the fact that it permits simultaneous computation of stress distributions within the soil, for study of consolidation settlements, group effects, etc.. In this respect their application introduces no hypotheses beyond those in use for calculating stress distribution in soils due to applied loads (Boussinesq, Westergaard, Mindlin etc) and merely permits exclusion of the arbitrary selection of an elevation at which a pile load would, for such computations, be assumed effective (e.g. at two-thirds depth of the pile).

Salas and Belzunce (1965) also employ Mindlin's solution, considering a rigid pile, to determine the distribution of stresses along the pile. Nishida (1964) follows the same methodology to derive equations for the elastic settlement of a pile subject to a static load, as a function of the mechanical properties of the soils and surrounding soil. Nair (1967) treats an essentially similar problem of stress transmission and consequent elastic settlement caused by a friction pile.

The above solutions have been concerned exclusively with immediate elasto-plastic settlements such as are observed under short-term load tests. For the case of piles in saturated clayey soils, the stresses transmitted to the soil by the static loading in question will create pore pressures that result in an additional time-dependent consolidation settlement. Poulos and Davis (1968) investigated the settlement behaviour of a single axially loaded incompressible pile in an ideal elastic soil mass analysed by means of Mindlin's equation; both the above-mentioned contributions were considered. The subject of the time dependent settlement will be further discussed later, since execution effects and group effects enter jointly with the static load influence herein considered. Moreover, suffice it to quote for the present the authors' conclusion that "a significant result of the analyses is that the major portion of the total final settlement of a single pile in an ideal soil occurs as immediate settlement and that only a small proportion occurs as time-dependent consolidation settlement". Nishida (1963) had presented an analytical solution for the Pore Pressure in Clay Induced by Pile Friction, based "on an analogy to the theory of elasticity, with some simplified conditions from the practical point of view".

It may be mentioned that yet a third line of approach to the overall problem was suggested by Kondner (1962), using nondimensional techniques based on the methods of dimensional analysis to develop expressions for the load-deflection charactoristic of vertically loaded, vertical friction pile groups in cohesive soils. For a single pile the functional relationship, under simplified hypotheses, resulted as

$$\rho/L = f\left[\frac{Q}{\pi L d c}\right]$$

where d = diameter of the pile of length L, and ρ = settlement, Q = load, c = cohesion. By methods of curve fitting, using a set of data from both model and field tests, a two-constant hyperbolic relation was obtained, fitting the plotted data very closely. Such methods, however, have important limitations as regards extrapolation to situations for which the hypotheses and/or practical conditions bear no close analogy.

2.5.3 Some consideration on load tests on piles.

The selection of appropriate methods of execution and of interpretation of load tests on piles and piers has merited some attention, most particularly because of the need of predicting future long-term behaviour on the basis of short-term duration tests. The preoccupation has implicitly been more specific with respect to clays and to friction piles, expected to distribute pore pressures due to static loading to a larger volume of soil and, therefore, subject to slower consolidation drainage: however, none of the test and interpretation procedures published have distinguished between different types of soils or piles. Obviously, with respect to service loading it would be desirable to run the test so as to accentuate the durations and consequent settlements, to facilitate predictions by extrapolation: on the other hand, for saturated clays subject to compression in shear, to obtain a conservative estimate of the failure load a fairly rapid, undrained loading is to be preferred. Finally, because of the subdivision of the total pile load Q into its point bearing and skin friction components, there has been some collateral interest in attempting to discern these separate contributions from the very load test, although conducted under slightly different routines.

The more traditional manner of running the test is today classified as the Maintained Load, M. L., test. Each load increment is maintained constant until the rate of settlement is so small that the pile-soil interaction is implicity accepted to have reached equilibrium. Such a test usually runs for a few days. It is believed that no published data are available of measurements of pore pressures and rates of pore pressure dissipation due to such single loaded piles or piers in clays (Davis and Poulos 1968).

Principally in order to improve the evidence of failure in the load-settlement diagram (analogous to the use of constant rate of strain equipment in laboratory testing), and concomitantly to shorten the duration of tests, the Building Research Station, England, introduced the so-called Constant Rate of Penetration, C.R.P., test. Results are furnished (Whitaker and Cooke 1961) to prove that the ultimate loads defined by successive loading-unloading cycles of CRP tests establish an envelope that truly defines the load-penetration diagram of the pile into the soil, whereas the interference of settlements in the ML test curve at loads close to the ultimate does not permit such an unequivocal definition of failure. Stoll (1961) criticizes the CRP test as an unnecessary innovation towards establishing a more correct value of Q_f, and Dubose (1956) offers evidence pertaining to a desiccated medium to hard clay wherein the settlements of the two curves are vastly different beyond the service load but the CRP test gives a curved load--settlement as difficult to interpret for the Q_f as in the commonly observed ML tests.

It appears that considering the dual interests in a load test the obvious recommendation would be to run a combined ML--CRP test, for the sake of better indication on settlements, up to the design load the ML procedure is preferable, and for a rapid and appropriate indication of the usually more critical undrained Q_f, thereafter the test should be extended to failure under the CRP procedure. Sharman (1961) offers much evidence of the satisfactory coincidence of Q_f of the CRP and ML tests in soft to stiff saturated clays, independently of rate of strain employed. Mohan (1967) confirms the satisfactory correlation between ML and CRP ultimate loads in expansive clays.

As a practical method of running an adequate test quickly Mohan (1967) proposes the so-called equilibrium method in which both the settlements and the Q_f of the ML test are very closely reproduced. It consists in applying a given load increment, maintaining it for about five minutes, and then allowing it "to reduce itself due to the yielding of the ground. The next higher load is then applied and the process is repeated. For higher loads. it is desirable to maintain the initial load for a period of 10 - 15 minutes before it is allowed to diminish ...." "The load at which equilibrium is attained is always lower than the maximum and it provides a better average than that obtained in a maintained load test".

The problem of determination of the real "yield value" of a pile load test is treated by Wan-der Veen (1953), Stoll (1961), Murayama and Shibata (1960) and Yamagata (1963), among others. Van der Veen (1953) had postulated that the load settlement curve can be represented by a formula $Q = Q_f\ (1 - e^{-a\rho})$ for point bearing piles, where the loads and settlements should refer to those valid at the point bearing. He thus employs the break in the straight lines observed in a plot of ρ vs log $(1 - Q/Q_f)$ as an indication of yield. Stoll recommends Housel's method, incorporated into the Detroit Building Code. The test is run with each constant load maintained for exactly one hour, with readings of settlements at exactly every 10 minutes. Thus for each constant load the "final" increments of settlement (for the final 30 minutes) are established, permitting the plotting of the "yield value diagram" of load versus the increment of settlement in the last 30 minutes. It is claimed that this diagram separates into two distinct straight lines, the first of essentially no increment of settlement, and the second showing a steep increase of such increments of final settlement with increment of load. The intersection between the two straight lines is interpreted as the yield value.

Murayama and Shibata (1960) based on a rheological model of behaviour of friction piles in clay conclude that "the settlement of the friction pile is proportional to the logarithm of time, if the applied load is constant": moreover, they prove that linearity of the log stress-log. settlement graph results for ML tests on such piles. "Since the linearity of log τ - log ρ curve holds only for the stress below the upper yield value, the stress corresponding to the bent point from which the straight line deviates should be equal to the upper yield value". Thus they simply propose plotting the ML data on a log-log graph and defining the yield value by the intersection of the two lines. Yamagata (1963) follows the same line, accepting it also for the yield conditions of point-bearing piles. With respect to all these different techniques it may be noted that there are sufficient theoretical reasons, accompanied by evidence from the laboratory and the field, to raise doubts about the generalizations of the equivalence of the results in the different tests. On the one hand there is much evidence (see item 4.3) to prove that with some piles and soils the rates of pore pressure dissipation are so quick that any ML test would tend towards a drained test condition whereas a CRP test may yet approach a partly undrained condition. On the other hand there is the theoretical and experimental evidence that a soil allowed to consolidated partly under any stress increment, tends to develop lower values of $\Delta u/\Delta\sigma$ for the subsequent increment of pressure than it would if it were directly stressed from the start to the second stress level (Bishop 1957). The only cases in which the different load tests would produce no perceptible difference would be the cases in which the pore-pressures due to the static loading were insignificant, or of extremely slow dissipation. Highly preconsolidated saturated clays may obtain in conditions such as not to create pore pressures differentiating between undrained and drained behaviour. It is of interest to mention that in the two load-tests of longest duration known (Eide et al., 1961), 198 and 438 days respectively, the drained and undrained bearing capacities of the pile were almost identical. The highly preconsolidated condition may obtain either from the natural conditions of the clay deposit, or, for displacement piles, from high pile-driving stresses, in comparison with which the static load stresses may be insignificant. Lo and Stermac (1964) for instance observe practically no Δu due to load tests on piles in a stiff clay.

As regards the important point of estimating the long-duration settlement ρ_∞, the fundamental point concerns the knowledge or postulation of the time-settlement law. Van der Veen (1953) had already suggested a linear relation of ρ vs log t for point bearing piles, and Murayama and Shibata (1960) and Yamagata (1963) maintain the same indication for other cases. Yamagata expresses $\rho = (f + m \log t)\, Q^n$ as the basic equation, the coefficients f, m and n being determinable from the data from load tests, below the yield values. Sharman (1961) postulates a linear relation of $\log \rho$ vs $\log t$, and observing a pile during a period of 209 days obtains the empirical equation $\rho = 19.5\,[1 - (t + 1)^{-0.36}]$ being in inches and t in days. Cambefort and Chadeisson (1961) postulate as an approximate law $\rho = a + b \log (1 + m\sqrt{t})$, i.e., linearity of ρ vs. $\log (1 + m\sqrt{t})$: this approximate law is postulated to represent conveniently what are stated (presumably based on consolidation theory) to be well-known laws of $\rho = a_1 + b_1 \sqrt{t}$ for small time intervals t, and $\rho = a_2 + b_2 \log t$ for large time intervals. Finally, Tassios (1963) basing on the consolidation theory equation $U = i \log t + j$ where U = percentage consolidation and i, j are empirically fitted coefficients that he determines, derives an expression for ρ_∞ as function of i, log t and the ratio of two known settlements ρ_2 and ρ_1 at times t_2 and t_1 under the constant load.

It may be seen that the opinions on the subject are somewhat divided. The possibility or real significance of attempting to discuss the relative validity of such laws appears very small, since on the one hand it is generally agreed that most of the settlements of individual piles and piers are very rapid, and on the other hand, in any long-duration prediction, the major effects derive not from consolidation theory or from rheological behavior detected shortly after pile-driving, but from the after-effects of the execution effects, and other multiple unknown pile-soil interactions.

Finally, it may be mentioned that techniques based on repeated and progressively increasing cycles of loading and unloading have been repeatedly described both as means of bettering the interpretation of ultimate and allowable loads (Szechy 1961), and as indirect means of distinguishing between friction and point bearing contributions (van Weele, 1957, Jain and Kumar 1963, Mohan 1967). For the purpose of this separation, of frequent interest, special small load-testing devices have also been introduced for routine use in piling projects (Mori & Sone 1964, Broms 1968). Haefeli and Bucher (1961) employ a specially developed model-pile type of penetrometer which is inserted into the borehole, the toe being driven about 1 m below the bottom; by carrying out load tests on this penetrometer, measurements of skin-friction, and of the modulus of compressibility of the soil below the toe of the pile, are obtained, permitting the evaluation, through appropriate theoretical developments, of the bearing capacity and the settlement of the pile.

2.6 Complementary in-situ tests in widespread use.

Principally in connection with the design of deep foundations, the foreseable questions regarding holing, sampling, and laboratory handling for testing, early led to the development of appropriate methods of in-situ testing of soils. The two methods that have gained more widespread use appear to be the socalled deepsounding static penetrometer test, and the pressuremeter (Ménard) test.

2.6.1 The static cone penetrometer

Its early conception was partly connected with the successful use of cone tests for for the measurement of the undrained strength of clays in the laboratory (e.g Skempton and Bishop 1950, Cadling and Odenstad 1950) and partly connected with a conception of representing a model of a pile, pushed into the ground, for which both the skin friction and the point resistance could be measured, separately and jointly. Several and continuous developments of design of the equipment (cf. Sanglerat 1967) have refined the methods of separate measurement of the point resistance and the accumulated friction resistance, or the friction resistance along a small stretch of jacket right behind the conical point (e.g. Begemann 1953, et al).

Meanwhile the continuous and successive developments in its application to the design of deep foundations have closely accompanied the developments and vicissitudes already discussed with respect to the point bearing capacity factor N_{qp} of deep foundations in sands. A summary appreciation of about eighty different papers on the subject, published over the past thirty years, leads to the following simplified general conclusions and design indications, from which it will be justified to restrict further specific interest, within the present appraisal, to the case of purely cohesive ($\phi = 0$) materials.

(a) Possible rare exceptions excluded, the studies demonstrating the application of the penetrometer method have been clearly restricted either to model-prototype estimates, or to the cases of purely cohesive and purely cohesionless materials, with a vast preponderance of the latter. Thus, notwithstanding the interest in maintaining a general treatment of (c, ϕ) materials present in this discussion, it must be recognized that within the routine practical applications in foundation engineering innumerable such cases of (c, ϕ) materials must have been encountered, but without hitherto furnishing any additional light for theoretical appraisal.

(b) Regarding the design of pile foundations, similitude (with jacked precast piles), size effects, and rate of penetration effects, it has been proved to satisfaction that the penetrometer furnishes invaluable, easily applicable, data. Rate of penetration effects are insignificant. Size effects are important in connection with point resistance, as mentioned below; but there is general agreement (despite an understandable scatter of fundamented opinion) that for the skin friction the direct extrapolation in proportion to the pile to penetrometer surface areas is on the side of safety.

In special cases (e.g. Cambefort 1953) it is emphasized, as is quite comprehensible, that the direct adding of the two component resistances is unwarranted. And in special strata (e.g. Florentin et al 1961, Artikoglu, 1961) some correlation factors have been established on the basis of comparisons with a sufficient number of load tests.

(c) The ultimate pile or pier point resistance in a homogeneous medium is the same as the penetrometer point resistance (Kerisel 1961, de Beer 1963, Vesic 1965), provided, however, that the very important similar embedment ratio(depth/diameter) within the stratum is respected because of bearing capacity considerations. A certain minimum thickness (10 to 20 diameters) of uniform soil is required in order to obtain a penetration resistance which is truly representative of a particular stratum. Thus, in view of the fact that the penetrometer point resistances zig-zag much more sharply than would obtain under pile points, it is necessary to obtain for the pile a fair average value: it has been suggested that the average be obtained over a distance from one diameter below the actual pile point to 3.75 diameters above it, (van der Veen 1957).

(d) Allowable point bearing values are established through an empirical reduction factor (variously recommended between 2.5 and 10) taking into account heterogeneities of the strata, execution effects, and settlement considerations. For instance because of the heterogeneities (in sands) on the basis of statistical comparison with load tests a factor of 1.5 is recommended (van der Veen 1957); and, for the same conditions, because of settlement considerations through comparison with the stress-strain behaviour of a 42 cm diameter test pile (Plantema 1948) an additional factor of about 1.6 is recommended. For a more general discussion of such empirical reduction factors see, for instance, Menzenback(1961).

With these general indications, further attention is now directed to the use of the static penetrometer for determining the ultimate bearing capacity for deep foundations in saturated clays. If we accept the basic formula $R_p = N_{cp}c$, these investigations can variously be considered either as a check on point resistance R_p if the $N_{cp} = 9$ value is accepted and the in-situ c is considered known, or as check on N_{cp}, or finally as an additional means of determining the in-situ c.

References that permit such correlative data are relatively few and, for the present, extremely confusing.

Trow (1952) presents the results of an investigation to depths of 1.5 m comparing Rp values with UC tests, and separately comparing in-situ vane tests with the UC values. The results are somewhat difficult to appraise because of tremendous scatter and possible disturbance effects: if we assume the vane values as more nearly correct we would conclude that

$$d.5\ c < Rp < .17\ c.$$

Pellegrino (1961) furnishes thirteen pairs of data on sedimentary clays, for which one would obtain $Rp = 8.1\ c$ as compared with UU tests.

Thomas (1965) introduces the Dutch deep-sounding machine for investigation of London clays, and for clays of c values between 5 and 25 t/m^2 obtains an average $Rp \approx 18\ c$. Considering the wide dispersion, evidenced to increasingly greater degrees in the stiffer clays, the range observed was $14\ c < Rp < 30\ c$.

Begemann (1965) furnishes ten sets of results on a very soft clay of c values between 1 and 4 t/m^2: a comparison with the vane shear values would indicate approximately $Rp \approx 15\ c$.

Blight (1967) presents the results of 14 pairs of results for indurated fissured clays of vane c values varying between 3 and 55 t/m^2. With remarkably small dispersion a correlation of $Rp = 15.5\ c$ is obtained. The author adds, however: "In all of the indurated fissured clays tested so far the undisturbed vane shear strength is about twice the unconsolidated undrained triaxial strength. Hence, in relating the cone resistance to triaxial shear strength a value of $Ncp \approx 30$ would be appropriate".

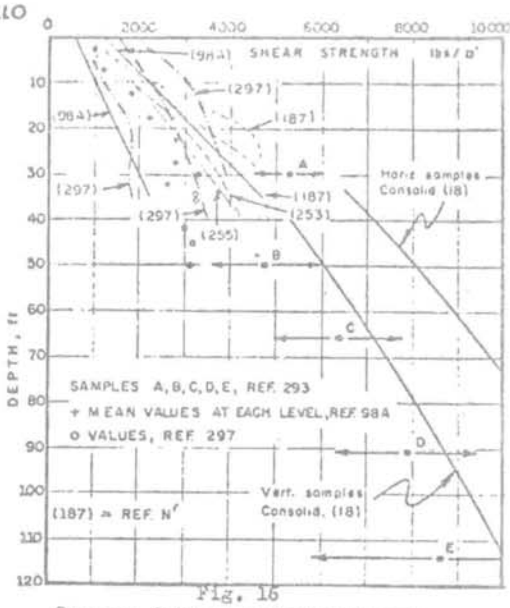


Fig. 16
Summary data on undrained shear strength vs. depth, London clay.

Finally, Ward et al (1965) reporting on the wealth of invaluable data from the Ashford Common Shaft, indicate for the stiff fissured London clay of c values between 16 and 55 t/m^2, an average $Ncp \approx 15.6$ but with variations as wide as $13\ c < Rp < 35\ c$, as compared with UU strength values on the block samples from the base of the shaft at the five different levels. These results, that indicate discrepancies much larger than would be suspected, are herewith analyzed through the accompanying figures 16, 17 and 18, through which an attempt is made to expose the interference of very pronounced effects of stress release due to the excavation and borehole perforations, as principally responsible for the strange results reported.

To begin with, Fig. 16 has been prepared summarizing some of the published data on undrained strengths in London clay as varying with depth. It will not be necessary to expatiate upon the thoroughly described and well recognized fact of the wide dispersions of strength values at a given depth, and even from specimen to specimen within a given block sample; it has been amply justified that these effects are due to the fissures that open upon release of stresses due to holing, sampling and laboratory handling. The gradual widening of the range of dispersion with depth, directly contrary to the natural tendency for a reduction of dispersions in a given material with increase of applied stress, is part of the many indications that in a clay such as the London clay, the important effects to consider lie in the holing and sampling, as against the use of "non-destructive" testing. Notwithstanding, the data clearly demonstrate that undrained strengths increase very rapidly with depth, in close agreement with laboratory test indications (Bishop et al, 1965). Simplifying this rate of increase into a linear relationship and without any attempt at the statistical regressions (e. g. Whitaker and Cooke 1966), it may be postulated that within the depths considered the minimum strength values as recorded show a $d\tau_f/dz$ of the order of 0.35 psi per foot, the average 0.52 psi per foot, and the maximum 0.70 psi per foot. These three lines are superposed on the data summarized in Fig. 16.

Next, it seemed necessary to examine a little more carefully the data tabulated on the penetrometer readings obtained at the five levels A, B, C, D, E of special sampling and testing. It is known that in a homogeneous clay the Rp reading reaches a constant value within the first 15 cm of penetration (Meyerhof 1951, et al.). But is the clay homogeneous by any means with respect to a small cone penetrating through depths of 6 to 10 ft. ? The authors recognized that "there is a general increase in the ultimate penetration resistance Rp with depth below the shaft base ..." but in the table of results "the resistances ... have been averaged at each level for a penetration greater than 1.5 ft and the average bearing capacity factor Ncp has been calculated for each level from the average vertical strength of the block sample". Under quite a different assumption, that if the penetrometer were behaving properly it should reflect an increase in Rp with depth at least as represented by Ncp times the $d\tau_f/dz$ of Fig. 16, the tabulated data have been plotted in Fig. 17. Besides the tabulated data, which furnish evident curves of increase of Rp with depth, the only other data inserted into the graph were for the probable Rp values within the first 30 cm of depth, based on the laboratory shear resistance on the block samples, as well as the c value extracted from the results of the surface plate load tests: these estimated "initial" Rp values were computed on the assumption of Rp = 10 c.

The appearances from Fig. 17 are incontestable: (a) the Rp values increase significantly with depth throughout the four to nine feet of the penetrations, and the "initial" presumed values as estimated are

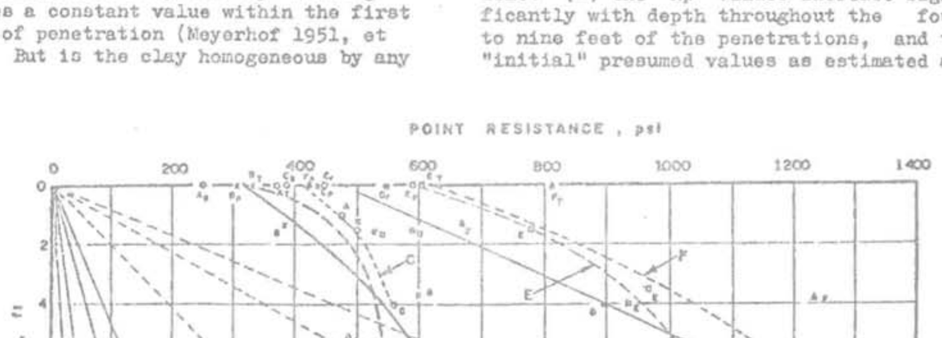


Fig. 17

Interpretation of cone penetrometer tests in London Clay, Ashford Common Shaft (293)

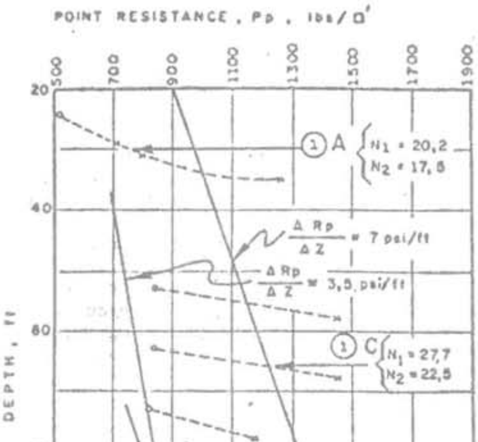


Fig. 18

Interpretation of cone penetrometer results in 6" borings, London clay (293)

not discrepant; (b) some of the curves (A, B, C) may appear to tend asymptotically to the dRp/dz lines estimated from Fig. 16; (c) the curves D, E, F indicate very much larger dRp/dz trends which require additional consideration.

Finally, to complete the preliminary analysis that the data permitted, Fig. 18 has been prepared on the basis of the tabulated results of the cone penetrometer tests, carried out from the bottom of the 6 inch diameter borehole at approximately 10 foot intervals of depth. For each level the readings tabulated correspond to the initial and the final penetration of about 5 feet. By plotting these results it is immediately apparent that for each test the overall $\Delta Rp/\Delta z$ is very much higher than what should be expected from the $d\,\tau_f/dz$ (Fig.16). Minimum, mean, and maximum values $\Delta R_p/\Delta z$ are 40, 88, and 127 psi per foot of depth, respectively. These values have been plotted on Fig. 17 for a visual comparison with the inclinations of the initial phases of curves A, B, C, and of the overall curves D, E, F.

On the other hand, a look at the series of initial readings, and the series of final readings, plotted on Fig. 18 appeared to indicate some similarity, with the dRp/dz rates anticipated on the basis of Fig. 16. In fact, despite the wide dispersions, the separate statistical regressions on the two sets of eleven results yielded the following very suggestive equations.

for "initial" readings $Rp = 600+3.25\,z$ psi
for "final" readings $Rp = 900+5.13\,z$ psi

Since the stress release effects at the bottom of a 6 inch diameter borehole should definitely not affect conditions 5 ft. lower, it seems that the in-situ undrained strength increase is well represented by the $d\,\tau f/dz \approx 0.52$ and correspondingly by the relation $Rp = 10\,c$. The stress release effects of the 25 ft. diameter shaft would, of course, be felt very much deeper than the first 4 to 8 ft (even disregarding time effects). Thus, the dRp/dz results from the bottom of the shaft reveal the stress-release effects. Finally, the coincidence of the minimum $d\,\tau_f/dz$ laboratory and plate test results with the "initial" dRp/dz, transformed into $d\,\tau_f/dz = 0.35$ through acceptance of the same $Rp = 10\,c$, is almost remarkable.

In conclusion, it appears that despite disheartening dispersions that stress the need for statistical analyses, the relation $Rp = 10\ c$ would not be far from expressing the results in London clay, if stress-release effects are better evaluated. Such clays would require the development of more "non-destructive" techniques of determining in-situ strengths. The extraction of undisturbed block samples from a deep shaft may end up being worse than a refined boring or in-situ testing technique, because, though the sample is the most undisturbed known to soil mechanics, it may merely represent a highly altered material due to the holing effects.

2.6.2 The pressuremeter (Ménard)

It will not be possible, in such a report, to expatiate upon the multiple publications introducing, during the past decade, the pressuremeter as a successful in-situ test, of entirely different conception, for the solution of a wide variety of foundation problems (e.g. Ménard 1963, Karst & Bourges 1964, et al). The intent herein is merely to include mention of the method as already having merited widespread application, and as offering promise especially for a number of cases in which the traditional methods bump against many imponderable interferences and dispersions, as above reported for the London clay, requiring, for the present, the use of unsatisfying empirical correction factors.

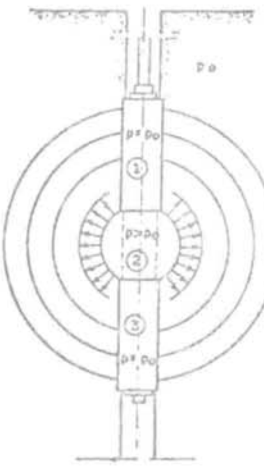


Fig. 19
Spherical pressuremeter (apud Ménard 1963)

The method basically employs as a principle the measurement of the deformability of a limited stretch of the borehole, by subjecting it to internal pressures to simulate the behaviour of elasto-plastic expansions of spherical or cylindrical cavities. Successive tests along the entire hole represent the desired E moduli measured along the borehole (bored or driven to represent the future pile) profile: these data have been successfully applied to calculate the settlements of single piles, reproducing the results of load tests (e. g. Goulet 1964). Moreover, the pressuremeter tests are carried to the point of defining as a mechanical characteristic of the soil in-situ, its limiting pressure p_l obtained for a cylindrical stress field, which corresponds to the pressure applied to the walls of a borehole causing general failure of the soil. Through appopriate formulae, the use of this single parameter permits the calculation of the bearing capacity of foundations through direct inter-relation of the two plasticity phenomena. Presumably this simplification, permitting the by-passing of the traditional approaches through (c, φ) determinations, may result in smaller dispersions in the final solutions, depending, naturally, on similitudes of conditions. Satisfactory applications have been frequently reported, and a set of design recommendations based thereon are summarized by Ménard (1963).

2.7 Deep foundations in London Clay

In any study of deep foundations in clay a special place must be reserved, at the present time, to the London Clay studies. On the one hand there is the unrivalled wealth of published information of prototype scale, with thorough coverage of observations and laboratory tests; and on the other hand, however, there is the peculiar nature of the clay, classed as a heavily preconsolidated fissured clay of $65 \leqslant W_L \leqslant 80\ \%$, which might lead to dubious generalizations in transplanting the conclusions to other clays. Thus, since there is an inevitable, ponderous tendency within the practice of foundation engineering, to follow design recommendations derived from London Clay as applicable to all insufficiently investigated clays, it is felt necessary to summarize herein some of the basic indications, and the questions that might arise.

An initial phase of investigation of bearing capacities of driven and bored piles, particularly the latter, ended with a masterly summing up by Skempton, 1959, that probably has set the bases of design practices most widely quoted. In a second phase, attention shifted to the large-bored piles, with enlarged bases, culminating recently in the Symposium on Large Bored Piles, London 1966, with its important contributions and practical summing up. Throughout these developments a very significant part was being played collaterally by the intensive research and publications on the properties and field behaviour of London Clay (e.g. Skempton and Henkel 1957, Ward et al 1959, Bishop et al 1965, and Bishop 1966).

2.7.1 Driven piles

The principal recommendations concerning the bearing capacity of driven piles (Meyerhof and Murdock 1953) may be summarized as: Point resistance to be calculated based on the undrained "natural shearing strength" using the factor $N_{cp} = 9$; Adhesion value initially admitted to be equivalent to the "fully remoulded shearing strength, which for the present clay was found to be practically identical with the undisturbed strength" (loc. cit.), and later revised to include the reduction factor β already discussed under item 2.5.1 (Tomlinson 1957, et al.)

It must be reminded however, that besides the problems generally recognized with respect to the interference of sensitivity, whipping and other execution effects, the very basis of these conclusions requires some examination. In none of the driven piles were the stresses at the tip, or along the pile, measured: the adhesion was obtained by subtracting from the total load the calculated point bearing load. Since in the London Clay the boring and sampling effects only tend to indicate poorer in-situ strengths than in reality should obtain, and the clay is insensitive, the net effect will be to underestimate the point load and to overestimate the skin friction. Consequent problems would arise in cases in which piles of significantly different lengths (those tested were 4 and 8 m long) and cross-sectional areas (a 30 x 30 cm was used) were to be considered.

The consideration of the two components as directly cumulative must also be revised, for other cases, on the basis of later information concerning the strains required for their mobilization (a small strain for the adhesion, and a strain proportional to the diameter, of about 0.085 B , for the point) and regarding the importance of the types of the stress--strain curves of the materials.

It is believed that significant revisions of present assumptions may derive from deep-sounding tests correlated with a especially instrumented pile load test. It would, moreover, be of great interest to check the application of the techniques discussed under item 5.2.2 to the better documented cases on record to check on the dispersions on the applicability of the routine UU and UC laboratory tests.

2.7.2 Bored piles

The subject received successive principal contributions from Meyerhof and Murdock (1953), Golder and Leonard (1954), sundry discussions, and finally, Skempton (1959).

A new technique, of running plate load tests at the bottom of the bored pile came into play, which has since found much application. Setting aside its use for evaluating average shear strength at the tip (through accepted Nc or Ncp factors) it has doubtless contributed to a better knowledge of ultimate tip loading applicable to bored piles in which it can simulate execution effects very appropriately.

The fundamental treatment of a direct adding of the two components of resistance continues in use for the case. The basic question under appraisal was the adhesion as affected by water-cement ratios, water migrations within the clay, etc..

Summarizing the results of 25 pile load tests on which adhesion values were determined, Skempton (1959) recommends an average factor $\beta = 0.45$, limiting the maximum adhesion, however, for prudence, to 10 t/m^2 in respect for the limits of the range of observations. The β value really observed lay between 0.3 and 0.6 as compared with the average curve of UC and UU strengths "from good commercial sampling and testing methods." An upper limit of $\beta = 0.7$ is postulated on the basis of "an increase in water content of only 0.5 %, and since some absorption of water by the clay is inevitable". The relationship between w and τ_f had been well established, as will have to be separately established in treating analogous cases of other clays, most of which will suffer different problems of water migration and consequent different changes of strength. As regards the lowest possible limit of $\beta = 0.25$ estimated, it was established based on "the lowest possible strength of the clay, measured after it has been allowed to soften fully under zero load." It appears that many clays would disintegrate under such a test, the lowest possible strength tending towards zero probably in a linear log t relation.

Thus, in short, it must be recommended that the methods of establishing β values be emulated in other clays and construction conditions, but not the respective numerical values, much cited. For instance Du Bose (1957) reports no water migration in a soil of $W_L = 35$ % and $W_p = 17$ %, compacted at 17.5 % water content. "The deformation necessary to mobilize the skin friction of a test pile was comparable to the shear displacement necessary to mobilize the shear strength of the soil in a direct shear test. This was about 0.1 in. for most of the tests", and "the end-bearing portion of the load was not developed until after the skin friction had been fully developed."

Regarding settlements Skempton concluded from six load tests on piles varying from 0.3 m x 7 m to 0.6 m x 22 m (diameter x x length) that the settlement at the ultimate load is $\rho_f = 0.085\,B$ and at 90 % of the ultimate load $\rho_{90} = 0.04\,B$, being the diameter. Thus in London Clay the settlement of individual piles at working loads is never a consideration: group effects may present a problem (see item 4.1).

2.7.3 Large bored piles.

The need for a separate investigation of the behaviour of large bored piles with enlarged bases seems to have resulted in close connection with the facts revealed by the model studies reported by Cooke and Whitaker (1961). These studies clearly showed again that whereas penetrations of the order of 10 - 15 % of the base diameter were required to develop the ultimate q_c base bearing capacity, the full shaft resistance was developed at very small penetration movements (these were expressed as 0.5 % of the shaft diameter of 19 mm, but since a single shaft diameter was used, and since it seems justified to accept that such shear strains would be independent of shaft diameter, it is recommended, that the shaft penetrations necessary for mobilization of ultimate skin friction be interpreted simply as a shear displacement). Moreover, as a general conclusion it was indicated that the overall mean shaft adhesion tended to decrease with ratio of shaft length to diameter (Fig. 20). This conclusion, which could be anticipated from the discussions of item 5.2.2, may be qualified through interpretations based on the interference of shaft length and the ratio of the E modulus of the pile material to the deformability of the clay, as well as on the basis of the sensitivity S_t of the stress-strain curve of the clay adhesion: on all counts it is interpreted that the model tests, using brass shafts in a very soft remolded clay of sensitivity about 1.2, would tend to attenuate the trend indicated in Fig. 20.

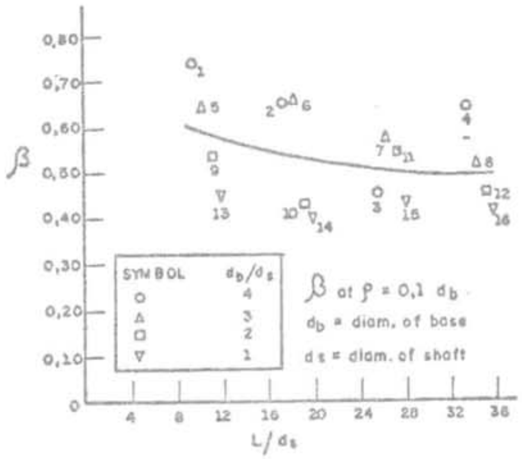


Fig. 20
Variation of mean shaft adhesion β with shaft length/shaft diameter (Apud Cooke and Whitaker 1961)

A considerable amount of field test evidence was subsequently gathered. As has already been emphasized, in the interpretation of these results, the scatter plays a very important role in selection of applicable shear strengths. As regards the use of plate bearing tests at the bottom of the shafts, it is clear that the strengths computed from such plate tests give smaller dispersions than laboratory tests, and would tend to lie nearer the lower limit of the scatter. As an example, Fig. 21 reproduces the results furnished on the

Barbican tests by Burland et al (1966). Such results are in accord with the stress-release effects discussed under item 2.6.1, and with the general indications from the field of Materials Testing, to the effect that whatever the type of failure considered, an increase in specimen size results in a decrease of scatter; furthermore, in the theory of brittle failure, average values decrease with an increase in size; in the theory of ductile failure, an increase in specimen size may result in slightly smaller, or, in cases, slightly larger, strengths; and, in the theory of deformation failure, the average strength is not affected by specimen size. Thus, Whitaker and Cooke (1966) state "When the clay beneath the base of a pile fails in shear a large volume of soil is involved, so that the shear stress at failure may be regarded as the result of testing a large soil specimen and would be expected, in accordance with the above postulation, to tend towards the lower limit of the range of strength values".

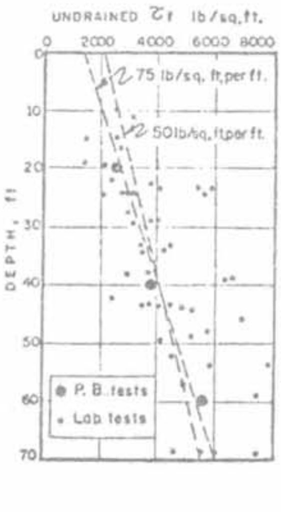


Fig. 21

Barbican shear strength/ /depth (apud Burland et al., 1966).

Fig. 24

Mean cruves stress-strain for shaft (Apud Whitaker & Cooke, 1966)

Insofar as the plate bearing test occupies the entire base (e.g. Burland et al.), irrespective of the attempt to interpret how truly it represents the in-situ strength of the clay, it doubtless represents exactly the in-situ strength and deformability of the volume of "unwittingly disturbed" clay that will inevitably comprise the support for the base of the bored pile. However, insofar as conclusions are to be extrapolated from smaller size bearing plate tests run at the level of the base, it appears that such extrapolations will be too pessimistic, considering the very rapid increase of strength (and E modulus) with depth that must prevail within the bulb of pressure releases as is believed demonstrated by the penetrometer results discussed under item 5.2.1.

By inserting a load cell at the bottom of the shaft, separate load test data on shaft and base performance were obtained, which have led to very interesting conclusions, and design recommendations (Skempton 1966) herewith summarized.

As a starting point, Skempton recommends maintaining the "theoretical" $\sigma_f = 9c$ relationship, and introducing a correction factor δ to account for the strength really applicable to the ultimate point bearing capacity, such that $c = \delta \tau_f$ where τ_f is the presumed in-situ strength. Thus the factor δ to be empirically determined would encompass the interferences of execution effects, dimension effects, etc. For instance, from the London Clay tests, Skempton states that "there was evidently an indication that up to a certain point δ tended to decrease with increasing diameter of the loaded area". Moreover the separate mobilization of the point bearing and the shaft adhesion contributions confirmed earlier evidence, (Fig. 22): overall shaft adhesion was fully mobilized at top settlements of about 0.3 to 0.5 inches when but 20 to 30 % of the base resistance would have developed, the corresponding settlement being smaller due to the elastic shortening of the shaft. Thus the suggested :

First Approximation Design Rules (loc. cit.)

a) Shaft: skin friction $\simeq 0.45$ (τ_f average over effective length of shaft)

b) Base: $\sigma_f = 9\,\delta\,\tau_f$

where $\delta = 0.8$ for diameter < 3 ft.

$\delta = 0.75$ for 3 ft $<$ diameter $<$ 6 ft. (for bells larger than 6 ft, data are not available)

Working load:

c) Plain bored pile: $\sigma_{all} = \frac{\text{Total ultimate}}{2}$

d) Belled (1) $\sigma_{all} = \frac{\text{Total}}{2.5}$ or (2) $\sigma_{all} = \frac{\text{Shaft}}{1.5} + \frac{\text{Base}}{3}$ } for diam. < 6 ft.

For diam. > 6 ft, working load to be evaluated from settlement calculations.

e) check settlements of structure.

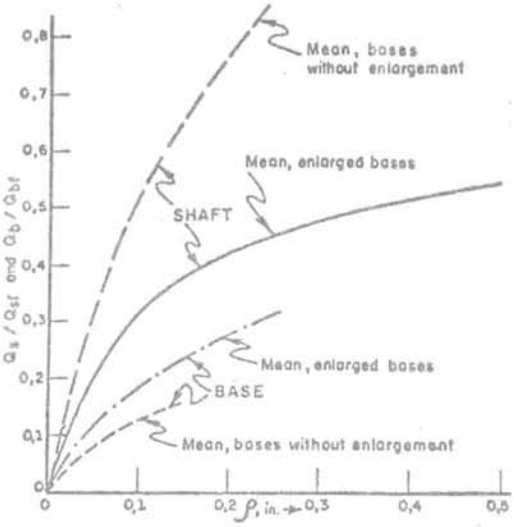


Fig. 22

Mean curves of development of ratios of shaft and base resistance (Apud Whitaker & Cooke, 1966)

With regard to the use of the results of the outstanding publications above summarized, some comments may occur to aid the design engineer's judgment. The interpreted increase of shaft adhesion with time (Whitaker and Cooke 1966, Fig. 17) may be subject to misure, despite the log. time scale, because of a lack of information on estimated limiting values. Fig. 23 herewith presented purports to incorporate estimates of the lowest limiting value of about $\beta = 0.25$ and the upper

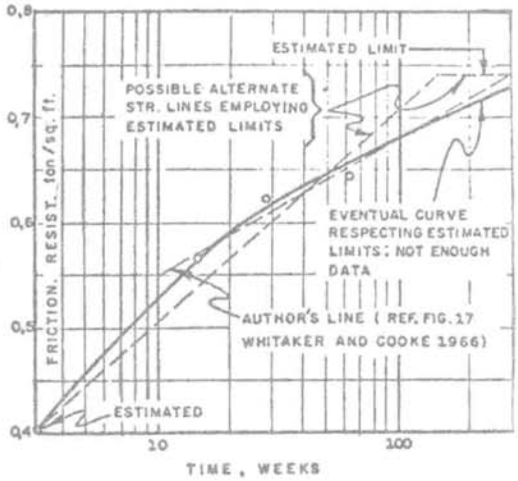
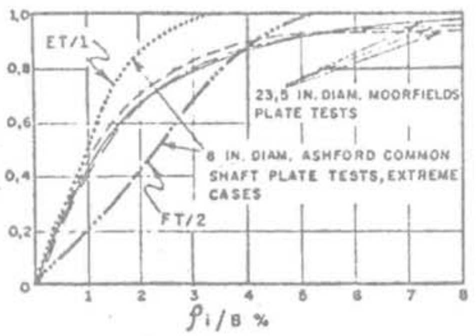


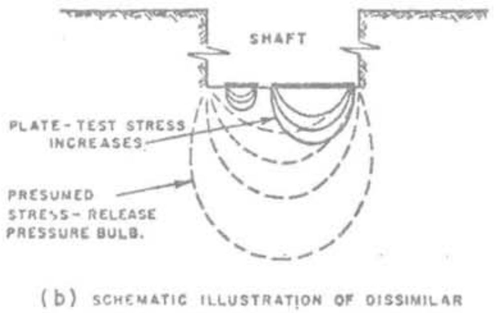
Fig. 23

Increase of frictional resistance with time shown by tests on anchor piles (Apud Whitaker & Cooke, 1966)

limiting value of about $\beta = 0.7$ as suggested by Skempton 1959. As a consequence either a curve might be suggested, or a double straight-line relationship be maintained. Moreover, it is not understood why according to the authors' Fig. 31 (loc.cit) herein reproduced as Fig. 24, at a given measured shaft strain the mean shaft stresses (or β factors) developed by the piles without enlarged bases would result about 25 % lower than for the piles with enlarged bases. Would statistical dispersions or execution effects cover the differences, justifying the use of the mean curve ? Finally, as regards the importance of settlement of the base, it appears most useful to employ the non-dimensional plot (Burland et al., Figs 6 and 7), herewith reproduced as Fig. 25, to estimate the immediate settlements. Quoting them: "The results of plate-loading tests carried out at various depths at several sites indicate that there exists a unique non-dimensional relationship between load and settlement for a wide range of depths at a given site. The relationship is nearly linear up to one-third of the ultimate load and may be written $\rho_m/B = K(\sigma/\sigma_f)$. Values of K have been found to lie principally between 0.01 and 0.02 for London Clay. When plotted in non-dimensional form the results of a



(a) NON-DIMENSIONAL PLOTS OF PLATE-BEARING TEST RESULTS



(b) SCHEMATIC ILLUSTRATION OF DISSIMILAR STRESS-RELEASE AND APPLIED STRESS CONDITIONS FOR DIFFERENT DIAMETERS

Fig. 25

Precautions in interpretations of plate tests at bottom of shafts.

plate-bearing test can be used to derive the load settlement characteristics of piles at that site". It may be recalled, however, that if the stress-release effects postulated are as important as presumed, it becomes necessary to run the load test with the overall base diameter in order really to reproduce the behaviour desired. In Fig. 25 the curves of two 6" plate load tests of the Ashford Common Shaft (Ward et al. 1965, Fig. 9) have been reproduced to exemplify the problem. It would be of interest to investigate within an enlarged base, the influence of diameter of loaded plate on the dimensionless deformability factor postulated by Burland et al., which derives from the assumption of a homogeneous material within the "pressure bulb". It appears that the accentuated stress-release effects, and the consequent importance of the plate dimension, may be associated with highly preconsolidated clays under present effective overburden stresses within that crucial range (overconsolidation ratio of about 5 - 15 %) in which the variations of properties with stress are very significant.

Juarez-Badillo (1964) discussing the influence of compressibility of soils as affecting the separation, in large-bored piles, of point and skin friction loads, suggests the interest in working with a compressibility parameter that does not vary appreciably with stress level: he thereupon suggests that a theory such as published (1965) may offer a simpler means of studying the problem, taking due account of volume changes due to both normal and shear stresses. Zeevaert (1964) uses empirical coefficients, in a formula based on Terzaghi's (1943) to take into account the compressibility of soils in connection with the design of pile and pier foundations.

2.8 The Standard Penetration Test, used in clays.

Unquestionably the use of the Standard Penetration Test, SPT, as a means of evaluating the allowable bearing pressures of footings on clays has occupied and still occupies a very important place in foundation engineering (Terzaghi and Peck, 1948, 1967).

Notwithstanding the many cautionary and restrictive clauses emphasizing the rudimentary and often erroneous indications furnished by the test, employed routinely in dry-sample exploratory borings, it is a fact that possibly more than 95 % of foundation studies are not apt to justify resorting to more specialized subsoil investigations and design analyses. Moreover, there are a great many clay deposits, usually rather preconsolidated, within which the use of a thick-walled driven spoon sampler constitutes almost a necessity, short of resorting to rotary-drilled Denison sampling. Finally, there has doubtless been an acceptance of the method somewhat wider than would normally be reasonable, because of its practical appeal, and the authority of the authors that gave it international divulgation.

The criticisms that have been levelled at the SPT concern principally the unstandardized nature of the test (notwithstanding its name), and the major errors that may be committed by the variability of executional routine from place to place. On the one hand, if the basic concern is with "random" errors, it is obvious that the errors of individual test results become statistically less important as the number of tests increases: and, it seems easy to affirm that in a soil in which the said test is subject merely to such random errors, the standard deviation of results obtainable from n SPT tests is no greater than that resulting from an equal expenditure in undisturbed sampling and UC laboratory testing (not to mention UU or CU tests !). Fig. 26 is herewith presented as an example of the reproducibility of routine SPT results as compared with laboratory strength determinations, for the case of a 26-storey building foundation in São Paulo: the in-situ undrained strengths were determined from UU tests on 2-inch undisturbed samples: the results are from a total of 8 borings spaced about 16 m apart, of which only four are represented:

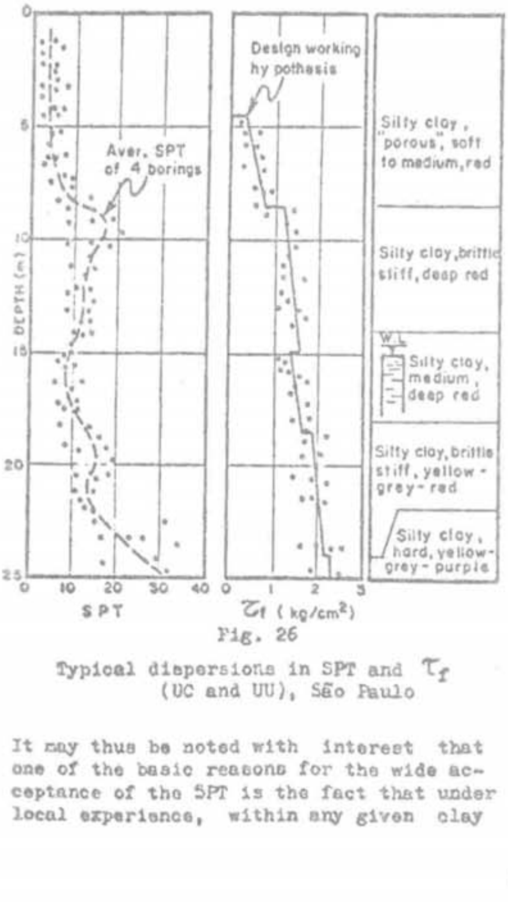


Fig. 26

Typical dispersions in SPT and τ_f (UC and UU), São Paulo

It may thus be noted with interest that one of the basic reasons for the wide acceptance of the SPT is the fact that under local experience, within any given clay deposit, the scatter of values has not been found to be any greater than that of any other routine strength tests (e.g. the UC tests). Thus, under any local set of conditions the confidence in the reliability of the test (as regards giving reproducible results) has established itself in many different places into unshakable convictions. The scatter of the test itself being much smaller than the natural scatter of properties and behaviour of individual elemental volumes of clay within a deposit, it is obvious that in establishing an average behaviour for the stratum for design purposes, the scatter of the SPT is not the crux of the problem.

What really fails is: (a) the transposition of correlations from one would-be "standard routine" to another: (b) the fallacious nature of some of the attempted single-parameter correlations $Y = f(X)$, (c) the transplant of empirical correlations to conditions far different from those for which they acceptably obtain, an evil consequent upon the otherwise laudable practice of publishing results.

Terzaghi and Peck (1948, p. 300) basically furnished an approximate means of establishing the UC strength of clays from the SPT value, through the Table III herein transcribed, that corresponds practically to the linear correlation UC (kg/cm^2) = = SPT/8.

Table III

Consist.	V.Soft	Soft	Med.	Stiff	V.Stiff	Hard
SPT	2	2-4	4-8	8-15	15-30	30
q_u = =2c t/m^2	2.5	2.5-5	5-10	10-20	20-40	40

And, thereupon, through the application of F = 2 or 3 to the ultimate bearing capacity approximately established as 6c, they admitted the possibility of establishing allowable bearing pressures for footings on clay.

Thus, with regard to the use of the SPT values for preliminary evaluation of the allowable bearing pressure of footings on clay, the basic premises may be synthesized into the following separate steps: (a) that the SPT value measured in clays can be offered as an approximate , adequately established, unique function of the respective UC strength provided the correlation is sufficiently conservative; or, conversely $c = f(SPT)$; (b) that the allowable bearing pressure σ_{all} for design of footings on clays may be satisfactorily established purely on the basis of a factor of safety $F \approx 3$ applied to the ultimate bearing capacity: (c) it may be observed in addition that a few scattered later references occur as to the possibility of establishing correlations between the SPT values and the compressibilities or deformabilities of soils (e.g. Hough 1959, Peck discussion 1960, Schultze and Menzenbach 1961, Teixeira 1966).

Attention will herein be concentrated on the first premise. It must be emphasized that with regard to it the authors originally cautioned "However, at a given number SPT of blows per foot, the scattering of the corresponding values of q_u from the average is very large. Therefore, compression tests should always be made on the spoon samples"(Terzaghi and Peck 1948, p. 300).

In an extensive survey of bibliography on the subject the following few references have been found with respect to the application of the SPT values in clays. The respective indications regarding correlation with UC strength values are plotted for comparison in Fig. 27. Trow (1952)

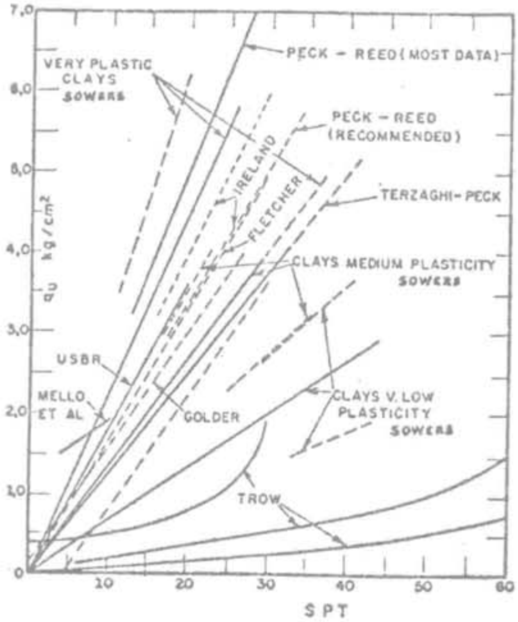


Fig. 27

Attempted correlations between SPT and q_u (kg/cm^2) for clays

correlates 2,000 data from eight different clays, claiming to have restricted the study to lean clays in order to avoid problems with sensitive clays. Sowers (1954) refers to many investigations on the correlations applicable, and offers three distinct bands for highly plastic clays, clays of medium plasticity, and clays of very low plasticity, plastic silts and fissured clays. Peck and Reed (1954) plot hundreds of data on Chicago clays and suggest the conservative boundary q_u = SPT/6 although an average curve approximates SPT/4. Schultze and Knausenberger (1957) claim to be in accord with Sowers (1954). Mello et al (1959) obtain for an unsaturated silty clay of São Paulo a statistical correlation $q_u = 0.061$ SPT + + 1,3 (kg/cm^2). The U.S. Bureau of Reclamation's (1960) results on a summary investigation are plotted. Golder (1961) refers to a good correlation with a clay of $W_L \approx$ ≈ 33 %: the corresponding data may be represented by the relation

$$q_u = \frac{SPT - 2}{7} \pm 0.4 \ (kg/cm^2).$$

Ireland (1963) refers to correlations in clays ranging between SPT = 6 q_u and SPT = 5 q_u. Fletcher (1965) again refers to the Chicago clay correlation SPT = 6 q_u and mentions a somewhat different correlation obtained by Cummings (1949) for Detroit clays, for which "the numerical ratio cannot be compared directly with the Chicago tests because the measure of the shear strength was not the same".

In a recent study (Mello 1967) an attempt was made to introduce at least a second recognizedly very significant parameter into the correlations: the "partial sensitivity" S_t' of the clay, probably related to the sensitivity S_t and representing the degree of loss of UC shear strength due to the remoulding effect of driving of the thickwalled sampler. Assuming a constant S_t' value for a clay deposit irrespective of its consistency, it was estimated that with an increase of S_t' from 1 (insensitive material) to 10, the SPT values corresponding to a given undisturbed q_u value would drop to as little as 15 %, as shown by curve (a) on Fig. 28. On the other hand, however, since it is well-recognized that the sensitivity of a clay tends to drop with higher consolidation, it can be seen that curved (probably second-order) regressions such as curve (b) would be more appropriate for each clay deposit.

The importance of the sensitivity (independently of many other factors discussed elsewhere, principally in connection with sands) is such as to make it meaningless to apply

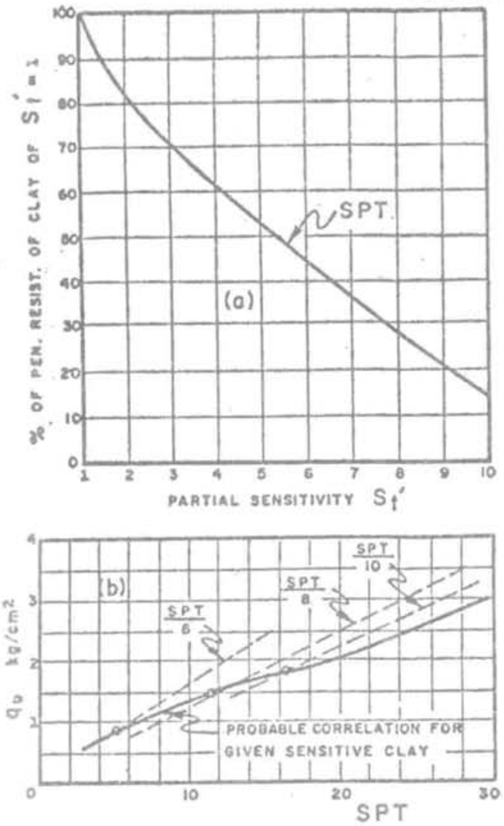


Fig. 28

Indications on probable effect of sensitivity of clays on correlations of q_u vs. SPT blowcounts

a generalized tabulation of classifications of clay consistencies (defined with regard to qu values) on the basis of the SPT values. At low consistencies the test is too crude a measuring unit, and at higher consistencies the differences in SPT blowcounts that reveal themselves because of sensitivity are very significant. Thus the obvious conclusion is that separate (and often very distinct) statistical correlations must be established for each clay, whereupon the test may be used satisfactorily within the same strata.

2.9 <u>Interference of deformability and settlements in the interpretation of load tests</u>

A renewed look at Fig. 1, curve C_2, reminds us of a classically emphasized fact that in quite a number of cases, in loose or soft soils, the settlements of the loaded plate are appreciable, tend to follow a non-linear development with increase of pressure almost from the beginning of the loading, and show a gradually steepening slope that makes it difficult to define the true failure condition, and furthermore, alert the designer to the probability that settlement considerations will be the controlling criterion on establishing allowable bearing pressures. The subject is commonly subdivided into the two separate issues: first, the necessity of arbitrating a bearing capacity failure criterion, so as to apply the factor F against any risk of such an eventuality; second, the establishment of suitable limiting settlement criteria for the load test, so as to avoid undesirable settlements of the building (it must be reminded that only one component of the overall settlements of a building is herein being discussed, cf. item 3).

With respect to the first problem the traditional and, it is believed, only solution as yet formulated is Terzaghi's(1943)"local shear" concept for the case of soils that exhibit an accentuatedly curved stress--strain curve. Transforming this stress--strain condition into an equivalent elasto--plastic diagram (for use in the rigid-plastic failure analysis), Terzaghi proposed, on the basis of the then available data, that a satisfactory conservative approach would result from an acceptance of the plastic failure as defined by strength parameters $c' = 2/3\ c$ and $\tan \phi' = \frac{2}{3} \tan \phi$ where the c and ϕ parameters are routinely defined by the peaks of the stress-strain curves. Thus the well-known "local shear" bearing capacity factor $N'c$, $N'q$ and $N'\gamma$ were established.

Whereas the concepts of the comparative phenomena of local and general shear are crystal clear once postulated, the foundation designer has been faced with a difficult choice between the two, having to draw an arbitrary line separating idealized types of behaviour where the variation from one type to the other is obviously continuous (for example, for sands Peck et al, 1953, introduced a continuous change in $N\gamma$ and Nq with relative density, to accomodate the transition).

The transformation of an excessive-deformation problem into a reduced shear strength problem was a justified artifice, that, however, withdrew attention from the deformation problems attendant to bearing capacity theory. It is thus foreseen that through the new computational facilities available one of the first important tasks ahead will be to suppress the N' factors, and to introduce, in their stead, correction factors depending on the deformability (cf. Fig. 8); such new correction factors will probably further depend on the dimension of the footing, and may even take into account parameters c_1 and ϕ_1 developing with strain at different rates (instead of a fixed proportion of the failure parameters) as is known to obtain. It has not been possible to examine the early evidence that led Terzaghi to his recommendations: it is of interest to note however, that Yong et al (1963) in their model bearing tests on a remoulded clay reached almost exactly the 2/3 values recommended for reduction on the cohesion.

One final consideration regarding the use of plate load tests for conservative indication on the bearing capacity of a soil concerns the test procedure. In normally consolidated and lightly overconsolidated clays the loading tends to create positive u and therefore a rapid undrained loading furnishes the critical condition. Since most load tests are of the ML type, obviously some degree of drainage is being permitted, not merely increasing the strength but also decreasing the pore pressure increments due to subsequent loading increments (cf. Bishop 1957). Very little information is available concerning the time of consolidation drainage under routine load tests as compared with theoretical derivations (Gibson and Lumb, 1953, Gibson, 1961) based on numerical computations. Lumb (1965) concludes that for a 2 ft diameter plate loaded at a constant rate "the total time of duration of the test necessary to develop the fully drained strength would be 1/2 hour for a silty sand, 6 days for a clayey silt, and 5 months for a clay". Christensen and Bent Hansen (1959) assume that after the "instantaneous" settlement, the settlements under each constant pressure may be treated as due to consolidation, being plotted essentially as data from the oedometer, starting with a straight lines vs $\sqrt{t}$ and following at some point as a straight line vs log t: they thus estimate the t_{50} % which for the Skive Septarian clay under a 30 cm plate turned out to be about 430 minutes. For other diameters times are considered to vary proportionally to the square of the diameter. Lumb's results indicate for a permeable plate one-quarter of the time t_{90} % as for an impermeable plate: thus the frequently employed technique of levelling the area for a plate with a thin sand cushion may considerably aid in draining the clay.

The important point is that a load test procedure should not be standardized without reference to such factors. Christensen and Bent Hansen (1959) obtained lower σ_f in a drained plate test than in the quickly loaded tests: the explanation may be that, due to the "highly preconsolidated" nature of the clay, a quick loading would develop negative u, and thus a drained loading would furnish the desired conservative result. Naturally, as regards the use of the load test for settlement estimates, under all circumstances the slower the test the more conservative the result (with the risk of some superposition with the consolidation settlement estimates discussed under item 3).

As regards the second problem, of the interference of load-test settlement data in the establishment of allowable bearing pressures for shallow footings, it may be recalled that Building Codes frequently specify a bearing pressure in accordance with settlements on a standard size of loaded plate. Such a criterion is, for the "local failure" types of soils, most frequently the controlling one, rather than that of σ_f/F. It is well recognized, however, that under a given pressure the "direct" settlements under a footing increase with its dimension, either in direct proportion in stiff clays, or somewhat less rapidly in sands (Terzaghi and Peck 1948, Bjerrum and Eggestad 1963, et al.). Moreover, as the building loads increase, the maximum differences between loads of adjacent columns tend to increase proportionally, without any increase of column spacing. Thus the respective footing dimensions would increase proportionally to $\sqrt{Q}$, and, if the same "allowable settlement bearing pressure" from the standard-size load test is used for two buildings of significantly different heights the Code is implicitly allowing the potential differential settlements between columns to increase in proportion to $\sqrt{Q}$. Approximately, therefore, for footings on clay a 20-storey building is being potentially permitted twice the limiting magnitude of differential settlements permitted a 5-storey building. Since Codes do not tend to be revised as rapidly as building heights increase, unless unexpected damages from differential settlements are being observed from the use of a hitherto satisfactory Code, it might be concluded that the increase in rigidity of the higher building compensates for the unfavourable tendency herein discussed.

Finally, it is considered necessary to attempt a clarification with respect to controversies that often arise as to whether the failure criterion or the allowable settlement criterion is the controlling one in decisions concerning the allowable bearing pressure for footings on clays. It has long been known (e.g.Taylor 1948) that for a given clay, for smaller sizes of footings the failure criterion controls ($\therefore \sigma_{all} = \approx 6\ c/F$) and with larger sizes of footings (i.e. for higher buildings) the settlement criterion is the controlling one. Whereas the position of the failure criterion horizontal straight line is well determined, the position of the curve for the settlement criterion would depend intrinsically on local experience on allowable settlements for a certain type of construction.

But the controversies naturally arose in the face of attempts at generalized design recommendations, such as, for instance, those based on single-parameter correlations with SPT blowcounts: whereas for a given size and type of construction in some places the failure criterion was evidently the controlling one, it appeared that on other clays the reverse was true (Hough 1959, and discussion). The distinction may be simply related to the plasticity of the clay as an additional first important parameter. Since the c/p ratio (despite its many intervening factors and dispersions) is known to vary essentially proportionally with the Ip, for a given consistency (c) the preconsolidation pressure pc will be smaller in a clay of high Ip than in clays of low Ip. Moreover, if we accept that the Ip and W_L of clays are essentially related through a rough parallelism to the A-line in Casagrande's Plasticity Chart, and use the empirical correlations establishing the Compression Index of a clay as almost directly proportional to W_L, we conclude that for a given applied pressure σ_{all}, the consolidation settlements of the footing of given breadth B will be larger in the clay of higher Ip, through an overriding effect that derives both from the lower preconsolidation pressure and from the higher compression (and recompression) index. Thus in clays of higher plasticity the settlement will be accentuated and will become the controlling criterion more frequently, as shown on Fig. 29, for hypothetical conditions.

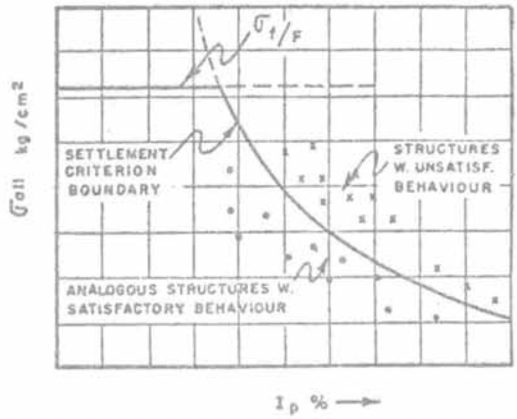


Fig. 29

Schematic illustration of influence of clay plasticity Ip, on determining the controlling criterion (failure vs. settlement) for given weight and type of structure.

3 SETTLEMENTS OF SHALLOW FOUNDATIONS

The subject of settlements of shallow foundations on clayey subsoil has certainly merited a place of great relevance in Soil Mechanics, both historically and as regards practical application, theoretical developments, and responsibilities involved in design decisions. Considering the limitations imposed by the original hypotheses of the Terzaghi theory and the oedometer test, which well reproduced those hypotheses particularly as regards no lateral strains in the clay during the one-dimensional vertical consolidation, it has long since been recognized (Skempton and Bjerrum 1957) that for most cases a more general treatment is indispensable, wherein the Terzaghi theory continues to be valid as a particular case, applicable to thin and deep clay layers under uniform loading extending over an area of large dimensions in comparison with the thickness of the clay layer. We shall therefore forego discussion of the earlier developments connected with settlement computations, and begin with the Skempton-Bjerrum proposal which is presently recognized as a minimum satisfactory approach.

The basic developments comprised in the latter proposal include a subdivision of the overall settlements into two components ("immediate settlements" and "consolidation settlements"): the former are considered to occur in a saturated clay by shear strains under no-drainage conditions, and subsequently the latter would take place under the induced pore pressures based on the changes of σ_1, σ_2 and σ_3, and not merely under a pore pressure assumed to equal the induced vertical stress. It is deemed worthwhile maintaining this subdivision in the following discussion. Thus, for their theory, $\rho_{final} = \rho_i + \rho_c$ where ρ_i = = immediate settlement and ρ_c = consolidation settlement.

3.1 Immediate settlements

According to the theory of elasticity the settlement of a loaded area is given by the classical expression

$\rho = q\,B\,\frac{1-\mu^2}{E}\,I$ where q = net foundation pressure, and I = influence value, depending on the shape of the loaded area and the depth of the clay bed, as given by Steinbrenner (1934) and adapted in Terzaghi (1943). (Note Bozozuk's discussion, 1963). Thus, adopting $\mu = 0.5$ for rapid loading on a saturated clay, the immediate settlement is $\rho i = q\,B\,\frac{3}{4E}\,I$, and "E is determined from undrained compression tests with a correction for sampling disturbance if necessary". For the more general case in which $\mu \neq 0.5$, Silveira (1967) has developed a method, using statistical regression, for the simultaneous determination of mutually compatible E and μ values from undrained triaxial tests. Ladd (1964) and others show, however, that many factors contribute to make the determinations of E through such laboratory tests very sensitive to sampling and testing conditions, depending on the nature of the clay: appropriate CU tests are indicated as preferable. In fact, dispersions on determinations of E for use in the above equation are certainly of such magnitude as to make it meaningless, at present, to discuss differences of I values based on shape and rigidity of foundations, as well as, possibly even, the effect of an E value varying linearly with depth.

Since the direct determination of E and from laboratory tests is subject to inescapable errors, many studies have employed load test data, either for establishing correlations with stress-strain behaviour measured in the laboratory compression tests, or for direct application through reasoning of model to prototype scale effects. Within the first approach, for instance, Skempton (1951) established approximately, within the range of allowable stresses, that at "the same ratio of applied stress to ultimate stress in saturated clays the strain in the loading tests is related to that in the compression test by the equation $\rho/_B = 2\epsilon$ ". Anand and Makol (1963) report results whereby such indications might be qualified in unsaturated clays. Yong (1959) in the case of silty material establishes empirical correlations with oedometric data. Recognizedly, however, the principal studies connected with the use of load test data have been towards direct relationships with size. Osterberg (1947) employing a log-log plot of pressure vs. ratio of settlement to diameter confirms that the consequent straight line "generalizes the data making it possible to predict settlements of any size and shape area". Kummeneje (1955) reports a satisfactory application of the idea to a tank foundation. Kondner and Krizec (1962) do not employ elastic theory but resort to dimensional analysis to establish linear correlations of experimental data.

Finally, special mention must be made of the paper by Resendiz et al (1967) reporting on investigations of the elastic properties of saturated Mexico City clays for the purpose of computing immediate heave due to foundation excavation, and consequent immediate recompressions. As a novel approach geoseismic measurements were used to compute the elastic parameters from vibrations. These yielded results that compared well with the measured heave in ten building excavations. "In spite of careful sampling and testing, sample disturbance probably reduces the modulus of elasticity of dynamically tested 'undisturbed specimens' up to 30 per cent, with respect to field values. The effect of sample disturbances in the modulus measured in undrained compression is even greater". Such evidence favours the non-destructive type of testing that has been mentioned as a recommended trend for the future.

3.2 Consolidation settlements

The subject of computation of the stresses induced in the subsoil due to surface loadings with be excluded from this Report, since the use of elastic theory has been quite routine for such problems, and there are evidences that the results are acceptable for most practical purposes. Moreover, the use of computer oriented methods essentially excludes limitations to developing solutions for most practical cases, both for the stress tensor and for the displacement vector (e.g. Dominguez and Moavenzadeh, 1967).

For the computation of the consolidation settlements ρ_c Skempton-Bjerrum (1957) first establish the induced pore pressure through the use of the A and B coefficients $u = B[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3)$ where the changes in stresses $\Delta\sigma_3$ and $\Delta\sigma_1$, would be computed from elastic theory. However, in order to retain the practicability of the recommended procedure, maintaining the use of the oedometer for calculation of the consolidation settlements, the authors develop a correction factor j, so as to dispense with consideration of lateral strains in this phase (although stating that this simplification could be made "without involving an error of more than roughly 20 % in the value of the vertical consolidation movements"). The correction factor $j = A + \alpha(1 - A)$ depends on the pore-pressure coefficient A, and on a coefficient varying from 1.0 to about 0.2 depending on the shape of the loaded area and the ratio Z/B of depth to width. Thus $\rho_c = j\,\rho_{oed}$ where $\rho_{oed} = \int_0^z m_v\,\Delta\sigma_1\,dz$ in routine oedometer settlement computations, $\Delta\sigma_1$ is obtained from the standard stress distribution formulae and charts, the m_v value is determined from the oedometer, and the value A is to be obtained from appropriate undrained triaxial tests with pore pressure measurements. It may be noted that values of j are shown to vary from about 0.2 or 0.5 to about 1.2 depending on the Z/B value and principally as a function of A, for which variations are indicated from 0 - 0.2 for heavily consolidated clays to 1.0 - 1.2 for very sensitive clays.

For time-settlement relations the degree of consolidation U is applied from the theory of consolidation, as will be discussed under item 3.4, so that at any instant $\rho_t = \rho_i + U j \rho_{oed}$. Practical applications were exemplified in comparison with observed settlements to confirm the preference for the method recommended.

It is not certain that the use of the Skempton-Bjerrum theory has really penetrated into foundation practice. The latitude of variation of j depending on A threw a heavy responsibility on the judgment for choice of a value of A, or for the choice of appropriate stress conditions for which to define A if the necessary triaxial tests are available. The theory did represent, however, an important step forward in exposing the limitations of the Terzaghi theory and in pointing towards the necessary solutions. In retrospect it seems that the practical decision to retain reference to the oedometer test limited the perspectives of the method.

3.3 Recent advances in settlement prediction

Some important recent developments in connection with computation of settlements merit a special reference in this Report, although their penetration into the practice of foundation design belongs to the future. They may be classified under the subdivision: (a) the stress-strain-time path method, using triaxial testing; (b) the generalized three-dimensional elasticity-consolidation theory; (c) empirical stress-strain-time formulations.

3.3.1 Stress-strain-time path method, using triaxial testing.

Lambe's publication "Methods of Estimating Settlements" (1964) describes fully the general treatment proposed by himself for the problem, and further expatiates on some comparisons with the Terzaghi theory and the Skempton-Bjerrum method. The principle is very simple, and well justified by the gradual international recognition, through careful triaxial testing, that the behaviour of soils is significantly influenced by the "stress-path" that the specimen or soil element underwent in reaching a certain final state. The best experimental approach presently available to follow such a trajectory of stresses and strains is the triaxial test with pore pressure measurements.

In principle one knows the vertical effective stresses to which representative soil elements of the clay layer are subject initially: moreover, the corresponding horizontal initial effective stress is estimated from the value $K_0 = 1 - \sin\phi'$. Thus a sample can be initially consolidated in the triaxial stresses. Thereupon, by elastic theory it is possible to compute the increases of a set of σ_1, σ_3 stresses caused by the construction loading. It is therefore possible to submit the specimen to an analogous stress-path (time of construction and consequent partial drainage conditions can also be approximately simulated at will, when desired), while the resulting vertical strains and volume changes are continually recorded. Lastly, under the consequent finally induced pore pressures, the specimen can be allowed to consolidate, usually under constant applied stresses , with continued recording of vertical and volumetric strains. The settlement is directly related at all times to the vertical strain.

In principle, the procedure is clearly a necessary development in the right direction. In practice, the present limitations are (besides those of sample disturbances etc. that affect all methods) the difficulties in choosing few "representive" soil elements to test in a rather complex procedure, thus raising the risks of error due to too few tests of exaggerated precisions, incapable of accounting for the inescapable statistical heterogeneities of the clay stratum.

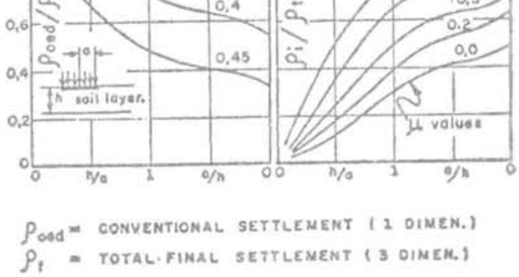
Davis and Poulos (1963) independently developed the same basic idea and expatiated somewhat more on the refinements of triaxial testing procedure required for this new three-dimensional strain settlement analysis, which, incidentally, eliminates the subdivision into "immediate" and "consolidation" settlements. Indeed, it is recognized that the standard triaxial equipment presents limitations particularly as regards possible membrane leakage reflecting on the time-settlement studies which rely on measurements of volumes of water (a measurement of smaller precision than that of strain measurements).

Incidentally, since the $\Delta\sigma_3$ values calculated from elastic theory depend significantly on μ values, and the latter change in a clay as it goes from an undrained to a drained stress condition (cf. Davis and Poulos 1968), a refinement in the stress-path technique may well take into account gradual changes in the applied stresses (principally $\Delta\sigma_3$) to account for this factor. Kerisel and Quatre (1968) recommending essentially a stress-path method, add the suggestions of considering both $\Delta\sigma_2$ and $\Delta\sigma_3$, and of revising computations to take into account realistic estimates of μ . Because of the limitations of the triaxial test it is suggested that for each soil element (for which $\Delta\sigma_1$, $\Delta\sigma_2$ and $\Delta\sigma_3$ have been computed) two tests be run, establishing boundary condition, one for ($\Delta\sigma_1$, $\Delta\sigma_2$, $\Delta\sigma_2$) the other for ($\Delta\sigma_1$, $\Delta\sigma_3$, $\Delta\sigma_3$). Moreover formulae and graphs are given for determination of $\Delta\sigma_1$, $\Delta\sigma_2$ and $\Delta\sigma_3$ as a function of applied load p, for different loaded areas and depths, and it is suggested that as a first approximation the stresses be obtained for an average $\mu = 0.25$. Finally, through formulae pertaining to the elastic isotropic material the observed volume of water expelled allows the calculation of μ , and if it is found to differ from the initial assumption, a new, more realistic, set of induced stresses may be calculated for a second cycle of approximation of the triaxial consolidation test recommended.

3.3.2 Generalized elastic theory treatment for settlement prediction under three-dimensional conditions

A general solution for the immediate and final settlements, and the rate of settlements, of footing foundations within the realm of elastic theory was recently put forth by Davis and Poulos (1968).

The immediate settlement is retained as determined from CU tests. A very interesting result presented (cf. Fig. 30) concerns the importance of the value of μ in determining (a) the error in the conventional one-dimensional approach for total final settlements, and (b) the relative importance of the immediate settlement in comparison with the total final one.



ρ_{oed} = CONVENTIONAL SETTLEMENT (1 DIMEN.)

ρ_t = TOTAL-FINAL SETTLEMENT (3 DIMEN.)

ρ_i = IMMEDIATE (UNDRAINED) SETTLEMENT

Fig. 30

Errors in conventional 1-dimension. approach for total settlement; and relative importance of immediate settlement, depending on μ (Apud Davis & Poulos, 1968)

The authors further discuss the interferoence of rigidity of footings in causing conditions of local yield in undrained loading, and conclude that for overconsolidated clays at loadings respecting $F = 3$ the local yield should not develop so as to affect computation of elastic displacements. Another important indication concerns the depths that are to be takan as "representative" for sampling, testing, and consequent computations. The triaxial test is used with a preliminary consolidation to K_o conditions, and with the appropriate undrained loading to determine E_u, and finally with the consolidation drainage under constant loading to permit determination of E' and μ', as well as C_{v3} (the three-dimensional consolidation coefficient, based on the observed rate of consolidation strains).

Notwithstanding the theoretical relationships between the various parameters, it is emphasized that preference must be given to extraction of values from tests, with due selection of appropriate ranges of interest.

Finally, for the computation of rates of three-dimensional consolidation an approximate solution is offered which will be further discussed under item 3.4. Obviously the rate of dissipation of u is very much greater than indicated by the one-dimensional theory.

3.3.3 Empirical stress-strain-time formulations.

An approach that coincides closely with the trend prognosticated for the future, within the key-note of the present Report is that suggested by Brinch Hansen (1965). The author observes that the present subdivisions into ρ_i, ρ_c, and ρ_s (secondary settlement), the first assumed to be instantaneous, and the following two constituting portions of the consolidation settlements easily distinguishable in the combined $\sqrt{t}$ - log t vs. settlement plot (Brinch Hansen 1961), run the risk of sacrificing on behalf of practicality, or of presently imposed rheological models, the reality that the stress-strain-time behaviour of a soil is obviously a continuous one, for which a suitable more complex rheological model has not been described. Clearly all three "components" of present settlement analyses occur superposed: there can be no instantaneous deformation, without superposition with the "primary consolidation", and the secondary or creep deformations must start as soon as the effective stress increments are applied during the primary consolidation pore pressure dissipation, and, finally, the secondary compression still requires some excess pore pressures, however small and eventually imperceptible to our present precision of measurements, to squeeze out the corresponding amount of water.

A more realistic approach, therefore, might be to express in a purely empirical way, through simple equations, the actual stress-strain-time relationships of soils as derivable from appropriate laboratory tests. Thus the necessary correlation constants of such equations may be defined through statistical regressions, with confirmation of its significance and determination of its confidence levels, so that the design estimates may be based on such empirical equations. The author suggests some formulae, and by relating them to the routine laboratory test results proves their applicability, reproducing measured behaviour within 90 % levels of confidence.

It must be observed, however, that in order to permit valid extrapolations and small dispersions such empirical formulations must be based on tests conducted in a truly representative manner, taking due account of the factors that at a given stage of knowledge, derived from analytical testing, have already been proven not to be random. Thus, in principle, the tendency should be, in the face of a given problem, to run a series of tests as closely as possible representing appropriately the stress-strain-paths and ranges desired, and, thereupon, with full recording of first-order intervening parameters and an evaluation of perceptible trends of behaviour, employ suitable statistical regressions to represent such behaviour empirically.

The development of testing equipment and techniques more suited to a given theoretical concept occupies an important place in any such reasoning. For instance, Calderon (1967) criticizing the use of the oedometer recommends the separate investigation of the consolidation behaviour due to isotropic stresses and due to deviator stress, and develops a special apparatus for the latter. Josselin de Jong and Verruiht (1965) also develop a special apparatus to investigate, in a spherical sample, volume compression without distortion. As has been frequently pointed out, but is continuously overlooked to the detriment of progress in the subject, the oedometer was specifically connected with the Terzaghi theory hypotheses.

3.4 Rate of settlement solutions

The basic concerns of settlement computations are not merely to anticipate their total or final magnitudes, but also the rates at which these settlements will develop. Despite a vast amount of testing, and a rather disproportionate amount of mathematical derivations, it may be said that the confidence levels in present-day estimates of rates of settlements are extremely low if unaided by judgment derived from field observations on the same deposits. One of the reasons may be, precisely, the fact that the subject has not been treated in terms of dispersions and significances of the intervening parameters. Mention must first be made of some of the principal theoretical developments, before appraising the sources of errors and dispersions.

A first distinction that must be made, however, of importance to the tone of this general Report, suggests the separation of the theoretical developments into three groups: one that is directly associated, or easily associatable, to the corresponding laboratory test procedures in use and required for design application of the respective conclusions, without basic questioning of the simplified rheological model; another, that has applied itself to eliminating some of the more purely theoretical limitations of the mathematics involved, for transplant of the equations to situations presumed needed as regards application to clay layers; and finally the third, connected with the practical interpretation of more complex rheological models.

3.4.1 Solutions connected with interpretation of laboratory tests, under the basic premises of the Terzaghi theory

Based on the original one-dimensional consolidation theory and the oedometer double-drainage test, the well-known Terzaghi-Fröhlich mathematical solutions for rates of average pore pressure dissipation and settlement were employed by Taylor and Casagrande to develop the classical $\sqrt{t}$ and log t "time-fitting" methods for direct determination of the coefficient of consolidation c_v, assumed constant for each pressure increment and for the height of the sample. It is stressed, as has been repeatedly done, that it is clearly preferable to deduce c_v directly through such interpretations of the test as a model of the prototype clay layer, it seldom being acceptable to attempt computations of c_v through its component parameters (principally permeability, compressibility and void ratio) or vice versa. Unless the model-prototype scale laws become partly invalidated, as obtains in regard to the errors introduced by the "initial compression" and the "secondary compression", both extraneous to the Terzaghi rheological model, much less error is involved in applying such implicit scaling relationships than in attempting to compose the c_v from the indirectly measured, theoretically related, basic parameters.

Moreover, clearly from the point of view of practical foundation problems the $\sqrt{t}$ method is the more useful tool because of defining trends from early indications, whereas the log t method has retained greater interest with regard to the arbitrary definition of the end of primary consolidation and the trends of further settlement along the so-called Buisman secular law, empirically established. Finally it is yet worth noting that were it not for the "experimental error" introduced in the oedometer model for saturated clays by the initial compression, the linear relationship U vs $\sqrt{t}$ is theoretically fundamented for the initial stages of a consolidation process, whether linear or nonlinear, and whether normal rectilinear, radial, or spherical drainage is taking place (Scott 1961).

In an attempt to reduce the incidence of the errors mentioned, Naylor and Doran (1948) suggested a method based on fitting a straight line to the range of readings between 60 and 80 %, "where there is every reason to believe that both the initial and the secondary consolidation are quite negligible"; but the method is not believed to have spread.

Meanwhile, because of a greater concern with soils exhibiting proportionally more secondary compression, Buisman and Koppejan (cf. Karst and Bourges 1964) developed methods for determining from the oedometer test, with some pressure increments maintained for 10 days, the coefficients required for computing the long-duration "final" settlements, the Terzaghi primary compression phase being accounted for under the conception of being the hydrodynamic retardation of the linear ρ vs log t law: (Fig. 31). Each method naturally deduces two coefficients of the linear law assumed (taken as independent of the

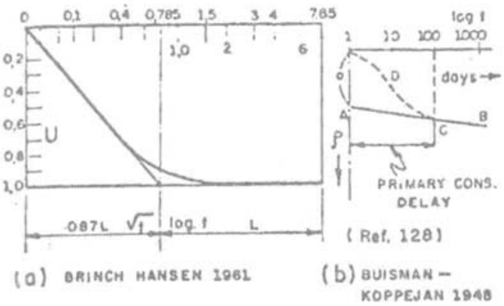


Fig. 31
Illustration of (a) Brinch Hansen (1961) consolidation time-fitting method ; (b) Buisman-Koppejan secondary compression time-fitting

thickness of the layer) $\rho = a + b \log t$, which Buisman writes as $\rho = H\,(n_1 + n_2 \log t)$ $\Delta\sigma$ and Koppejan writes as

$$\rho = H\left(\frac{1}{m_1} + \frac{1}{m_2}\log t\right)\log_e \frac{p_o + \Delta\sigma}{p_c}$$

where p_o is the initial effective stress, $\Delta\sigma$ the pressure increment, and p_c the preconsolidation pressure.

Zeevaert (1955) proposes a fitting method based on defining the boundary between primary and secondary compressions as defined by the point at which the slope of the ρ vs t graph of the primary consolidation equation equals the slope of the empirical ρ vs log t secondary compression line.

With the advent of triaxial testing with pore pressure measurements for determining consolidation parameters (cf. Bishop and Henkel 1957), preference was given to determining c_v by measuring the rate of dissipation of pore pressure at the lower surface of a cylindrical sample drained from the upper surface, the corresponding Time Factor $T_{50\%}$ being 0.38 (loc. cit.). Gibson and Lumb (1953) furnished solutions for T for the average pore pressure dissipation U, for the clay cylinder (of diameter 2R and height 4R) for the cases of radial drainage and end drainage: no special fitting methods are suggested but the $\sqrt{t}$ plot is used.

Scott (1961) discussing the disadvantages of the Taylor and Casagrande methods, recommends the determination of c_v employing ratios of compressions taking place up to different times. The only disadvantage it retains, common to the two earlier solutions, is the necessity of determining the "corrected zero point" by the accepted application of the near-parabolic form of the U vs T curve at small values. The method is rapid and convenient in use, permitting spot checks of computations of c_v during the very process of compression, and, incidentally, permitting an appraisal of the variation of the coefficient at different points in the process (cf. Lo 1960, correspondence). Moreover, the same principle can be applied to other test conditions, such as, for instance, the triaxial test with radial drainage, provided an analytical or numerical solution to the problem has been obtained and the respective volume changes during the consolidation process are observed.

Brinch Hansen (1961) suggeststhe combined ρ vs ($\sqrt{t}$ and log t) plot (Fig. 31). It appears that for practical purposes Scott's and Brinch Hansen's methods should by now have supplanted the others.

The oedometer has been employed for the determination of anisotropic c_v values in "laminated" alluvial soils, by cutting the specimens for vertical drainage in the desired directions (Rowe 1959), but also it has been adapted (McKinlay 1961) to furnish the desired radial porewater drainage and the respective calculated horizontal c_v , c_{vh} , through an adaptation of the $\sqrt{t}$ plot method. The data are plotted in the form of U against $T^{0.465}$, obtaining an initial linear part up to about U = 50%. "The point on the curve corresponding to U = 90 % has an abscissa 1.218 times the abscissa of the continuation of the initial straight line, and at U = 90 %, T = 0.3345". Thus $c_{vr} = 0.335\ R^2/t_{90}$ is computed.

Escario and Uriel (1961) use special triaxial tests with radial drainage only and measurements of expelled volume (or especially measured volumetric strains), to determine the value c_{vr} through a special fitting method. Aboshi and Monden (1963) use both the oedometer and the triaxial test under three conditions of drainage (one-dimensional vertical, one-dimensional with specimen strata vertical, internal radial, and external radial), always employing for calculation of c_v values the log t plot and the respective $T_{50\%}$ time factor. Kawakami (1964) develops a method employing the triaxial tests, and appropriate paper drains to force radial drainage, for determining c_{vh}: the curves for mean pore and end pore pressure versus time factor are deduced for various ratios of T_h/T_v ;

both the $\sqrt{t}$ and the log t fitting method are used, with the appropriate adaptations in a manner analogous to that used for the oedometer.

More recent developments include the spherical clay specimen consolidation by Josselin de Jong and Verruijt (1965), the specially developed oedometer-type cell developed by Rowe and Barden (1966), (cf. Northey and Thomas, 1965, with respect to u measurements in the oedometer), and the three-dimensional consolidation solutions employing the triaxial test, for which Davis and Poulos (1963) offer the curves of U vs T_V for determination of c_{v3}, the three-dimensional coefficient. Fig. 32 is herein reproduced from the latter publication, to furnish an indication of the order of magnitude of some of the changes of time factors at play depending on test conditions.

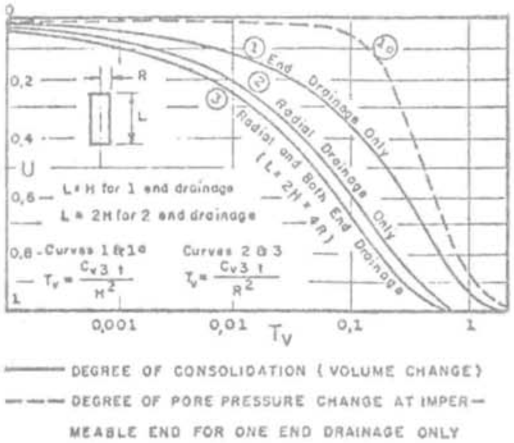


Fig. 32

Theoretical rate of triaxial consolidation for different drainage conditions. (Apud Davis and Poulos, 1963)

3.4.2 More general solutions for the consolidation of clay layers.

As has been summarized recently by Seed (1965) in a review of the subject, the basic differential equation governing consolidation by the dissipation of excess hydrostatic pressures in one-dimensional flow has been solved for a number of cases of great practical importance, among which the most commonly employed concern the cases of contiguous clay layers (Gray 1945) and multilayered soils (Abbott 1960) as well as those of time-dependent loading and varying permeability. More recently, in addition, some mathematical developments have been reported, especially in connection with eliminating from the Terzaghi theory some of its limitations concerning the constancy of coefficients that are really varying. Such studies need but be cited, because thus far their function has been limited to alerting to some possible reasons for the often observed discrepancies in rates of settlement. In this sector the mathematical refinements presently within reach (Christie 1966, et al.) are far greater than is warranted in the face of limitations in field investigations and in the laboratory consolidation testing, even if remodeled.

Gibson (1961) studies the three-dimensional rate of settlement of an uniformly loaded circular or rectangular area on an anisotropic semi-infinite clay stratum. Schiffmann and Gibson (1964) develop solutions for nonhomogeneous clay layers in which k and m_v are taken to vary with depth: although normally the calculation of rates of settlement is based directly on values of cv (which is very nearly constant with depth in a clay deposit) and not on values of k and m_V, their conclusion for a case in London clay is that the modified theory could justify a significantly more rapid initial settlement than estimated by the conventional assumptions. Martins (1965) gives the solution for the clay layer of k varying with depth while m_v is constant. Davis and Raymond (1965) develop a more accurate consolidation theory for normally consolidated clay assuming a constant c_v while the compressibility and permeability were both allowed to decrease with increasing pressure, which seems a more realistic set of assumptions based on present information. Yamaguchi and Kimura (1967) solve the problem for two assumed conditions of variation of k with depth, linearly where $k = k_o\ (1 + a\ \frac{z}{H})$ and exponentially where $k = k_o\ e^{b\ z/H}$: for the double-drained clay layer they conclude that for a wide range of conditions the errors in the time factors obtained by the conventional method do not exceed 30 % throughout the consolidation process. Finally, Gibson et al. (1967) further generalize the one-dimensional consolidation theory not to impose the limitation of small strains, and to include variation of soil compressibility and permeability while consolidation is taking place: the solution is limited to thin homogeneous layers, making negligible the self-weight stresses in comparison with applied stresses.

3.4.3 Rate of settlement solutions based on other rheological models.

The problem of secondary compression has been a fertile source of discussion and research for almost thirty years, being directly connected with all estimates of rate of settlement. The formulations of more complex rheological models than Terzaghi's, or the formulations of purely empirical laws to describe the observed behaviour, have been frequent; and the introduction of other theories (rate process theory, Marsal's stochastic theory for analysis of the behaviour of the grain skeleton, or Josselin de Jong's model consisting of an assembly of viscous elements or a cavity channel network, etc.) will doubtless continue to contribute to the investigation of the important problem.

Within the scope of this Report, however, mention will be limited to the few of such theories that have formulated practical recommendations, through the necessary data fitting methods, for application to design estimates.

Marsal (1961) developed a theory assuming that the relationship between intergranular pressures and volumetric strains is not independent of time, the strains being supposed to occur in part instantaneously and in part as an exponential funcion of time: the theory was limited to small pressure increments for linearization of the consolidation process. The author compared the predictions of the theory to some laboratory and field data, but conceded that the fitting of the theory to experimental data required lengthy computations. Lo (1961) subdivides the observed secondary compression behaviour of clays into three types of curves. "Briefly, for Type I curve, the rate of secondary compression gradually decreases with time, in Type II curve, the rate is proportional to the log of time for a considerable range of time, then rapidly decreases, and for Type III curve, the rate increases with time, then gradually vanishes". The method of determining the soil parameters is given, but since long duration observations are required under carefully controlled ambient conditions, it is not known to have had practical application yet. Gibson and Lo's (1961) rheological model requiring the determination, from test data, of four basic parameters, has been employed with a slight modification by Christie(1965) in comparisons with controlled laboratory tests. Barden (1965) develops a digital computer solution for a non-linear viscosity rheological model, and concludes that the amount of secondary compression is strongly dependent on the thickness of the sample and on the pressure increment ratio. The solution provides a simple method of accounting for this in extrapolation of settlement-time data obtained in the laboratory tests to the field scale: it is considered that the one-dimensional secondary compressions will be negligible at the field scale. Zeevaert (1967) develops a theory for materials showing intergranular viscosity, and recommends the fitting methods for oedometer test results both for curves showing a break and for those without it. Finally, Barden (1968) develops a rheological model in terms of structural viscosity and thixotropy and solves the one-dimensional consolidation process showing it to correspond adequately with known behaviour. An important result concerns the conclusion as to the scaling law. "Discussion has also centred on whether the scaling law for secondary compression depends on H^2, $H^{1.5}$ or H ... the effect of H is complex and the scaling law cannot always be stated in as simple a form as previously attempted".

In conclusion it may be summarized that the complex subject is still in the phase of formulation for practical applications to foundation design.

3.5 Significant parameters interfering in settlement analyses and their ranges of dispersions

It may be affirmed as the conclusion of the present state-of-the-art study, that before the practice of foundation design can benefit from many of the past and current theoretical and practical developments, an indispensable step comprises the quantification of the confidence levels of the presently all-important "judgment factor" on the principal intervening parameters within a design. The following comments attempt to summarize examples of such a concept.

3.5.1 Field information

The more significant necessary predesign information concerning a compressible clay layer resumes itself into the definition of the thicknesses of substrata that will be assumed homogeneous, for computation purposes, and their idealized boundary conditions.

As regards the calculation of "total" settlements, the subdivision into idealized layers must be based simultaneously on: (a) values of W_L as an indication of the plasticity of the clay and of its virgin compressibility, or even, within a rough approximation, of its recompression characteristics; (b) values of p_e insofar as there may occur some dried crusts, or some other discontinuities within any homogeneous trend presumed, as compared with the initial overburden effective stresses p_v; (c) the vertical rates of changes of the induced stresses Δp within each sublayer, so as to permit linearization.

Moreover, as regards the calculation of rates of settlement, the most important information required concerns the occurrence of drainage layers or seams, besides a measure of the very likely anisotropic consolidation coefficients c_{vh} and c_{vv}. (cf. a statistical conclusion by Rowe, 1959).

There is no difficulty in handling the problems concerning variations of W_L with depth, and concerning the anticipated change of Δp with depth. As regards the latter, the obvious trend is to devote greater attention to the upper horizons. But, since the really important lumped parameter implicit in present practical judgement on the problem may be taken as approximately a function of $W_L \log (1+\frac{\Delta p}{p_o})$, it would be of interest to employ linear regressions of such functions vs. depth so as to determine within what levels of confidence selected substrata may be linearized. Though such indications are but indirect and approximate, nothing short of a statistical treatment will really begin to give the design engineer a measure of the upper and lower confidence bounds within which he is working, and upon which he may apply to highest practical benefits, any further efforts at improvement of precisions.

The subject of stress transmission and preconsolidation pressures will be mentioned below. In connection with the latter suffice it to emphasize herein that several publications have pointed to the significant need of auxiliary geologic interpretations to explain unexpected behaviour.

The greatest limitation in field information concerns the fundamental parameters necessary for rate of settlement computations. Since rates of dissipation are influenced by the square of drainage distances, the need for obtaining information on the continuity of sand or silt lenses is a problem of preeminent importance that dwarfs most of the effects of the different theoretical formulations discussed under item 3.4. Moreover, it is not merely continuous sand laminations that have to be detected, if possible, and taken into account as drainage paths. As interesting possibility for computer investigation would be the influence on drainage rates in three-dimensional consolidation, that would be introduced by permeable lenses of limited area, that without effectively constituting drainage paths, would merely behave as pore pressure equalization conductors cutting across the pressure-bulb of induced pore pressures. By investigating comparatively the influence of the dimensions of such finite conductor planes on the idealized diagram of induced pore pressure distributions, and the consequent rates of dissipation, the results will provide the engineer with an indication of the lower boundary of confidence levels permitted by a given grid of borings, even if the latter were so perfect (through the use of preliminary impregnation of lenses with detectable dye solutions, and subsequent continuous Swedish foil sampling) as to appropriately describe all the sandy seams on a vertical profile.

3.5.2 Stress distribution theories as regards differential settlements.

Some problems pertaining to calculation of Δp stresses have not been discussed as regards settlement analyses. Initially it may be recalled that in the choice between the Boussinesq isotropic medium and the Westergaard medium, the latter hypothesis tends to attenuate appreciably the differences between the pressures under the center and the edges of the loaded area, with a consequent attenuation of the estimated differential settlements. The designer's range of decision on this item merely extends between the boundaries set by the two theories available.

Furthermore there is some interest in evaluating the possible stress redistribution due to an upper dense sand layer used for support of shallow foundations subject to major settlements due to deep compressible clay layers such as obtain in Santos (cf. Teixeira and Geotécnica, 1959). There is at present no direct information concerning the order of magnitude of the effects of such relatively rigid soil mats in possibly altering stresses induced in underlying compressible clay layers. The existing confirmations on pressures transmitted to deep layers by shallow loads, are indirect, based on carefully recorded pore pressure developments, subject to other sources of error and dispersions (cf. Gibson and Marsland's report to the London Conference on Pore Pressure and Suction in Soil, 1960, on "Pore-water pressure observations in a saturated alluvial deposit beneath a loaded oil tank", p. 112).

At any rate, it is generally conceded that there is considerable circumstantial evidence from overall results of calculated vs. observed settlements, to indicate that either there are smaller-order errors tending to compensate in the overall process, or the factors above mentioned are of themselves not significantly subject to the doubts raised.

3.5.3 Pore pressure coefficients A and B

The linearized concepts under which the A and B coefficients were postulated makes it very simple to determine the two unknowns A and B as absolutely defined values for a given set of conditions of the two basic variables assumed, $\Delta\sigma_3$ and $(\Delta\sigma_1 - \Delta\sigma_3)$.

In any such case of preestablished linearization of the real more complex behaviour, the responsibility of obtaining appropriate values for use in a given practical problem falls entirely upon the selection of the representative set of conditions for which the coefficients are to be calculated: and, obviously, the further the real conditions depart, consciously or unconsciously, from the assumed one, or the wider the range of conditions to be encompassed within the problem, the greater will be the errors introduced. Naturally, if several would-be analogous tests are available, and/or several sets of conditions of the two assumed variables are admitted, the least that can be done in practice is to establish average values of the coefficients and their range of variation. Such a solution, however, usually exposes rather dissatisfying dispersions.

Stated in general terms the fact is that $u = f(\sigma_1, \sigma_2, \sigma_3)$, at least, and for given ranges of variations of the three stresses it is possible to allow statistical regression, and consequent analyses of the significance of these regressions, to establish the most appropriate functions for each case, accompanied by the respective confidence levels. For any such regressions any test can furnish an ample number of data, but a minimum number of separate tests should be employed in order to evaluate dispersions due to specimen-to-specimen differences.

If such general regressions be found cumbersome, the least that can be done is to establish, for the ranges of stress variations desired, separate "best" or "significant" regressions $\Delta u = f(\Delta\sigma_3)$ and $\Delta u = f(\Delta\sigma_1 - \Delta\sigma_3)$ under the concept that any point defined by a set of coordinates $(\Delta\sigma_3, \Delta\sigma_1 - \Delta\sigma_3)$ is arbitrary if it is not defined as a member of the families of similar points among which it belongs. In very few cases can one really impose a linearization without inviting much more dispersion or error than necessary. For instance, the form of the Δu vs. $\Delta\sigma_3$ function in unsaturated soils is well known to be concave upward and asymptotic to the 45° line.

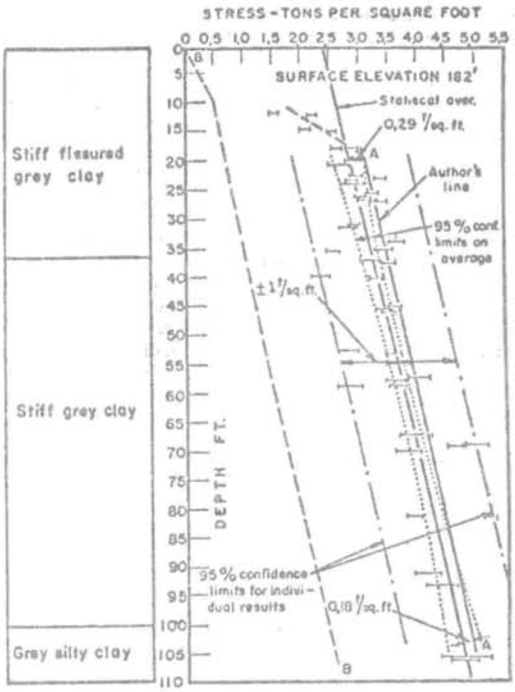
As an example, the data published in the "First Progress Report on Investigation of Stress-Deformation and Strength Characteristics of Compacted Clays" by Casagrande, A. and Hirschfeld, R.C., Harvard Soil Mechanics Series no. 61, 1960, have been used to confirm the order of magnitude of the improvements resulting from the technique above mentioned, long since in use, in comparison with the use of the A and B coefficients. The data on the PH tests, Fig. 10 (loc.cit.) for degrees of saturation $S_r < 76$ % indicated excellent regressions of the type $u^2 = a + b\,\sigma_3^2 + c\,\sigma_3^4$, for the range $6 < \sigma_3 < 14$ kg/cm^2. For each regression the computed u values were within an error of less than 0.1 kg/cm^2 from the measured ones. On the other hand by using a B value determined for the average $\sigma_3 = 10$ kg/cm^2, for the specimen of $S_r = 62.5$ % errors of over- and under-estimates at the extremes of the range were 2 to 5 times greater (20 to 25 % of the respective measured u). Meanwhile, however, by using the data of Δu vs $(\sigma_1 - \sigma_3)$ of the corresponding Q_4 to Q_7 tests (Figs. 21 to 24 loc. cit.) in the range corresponding to $\sigma_1 - \sigma_3 / (\sigma_1 - \sigma_3)_{max}$ between 0.35 and 0.7, despite the parabolic appearance of the overall Δu vs $(\sigma_1 - \sigma_3)$ curve, sufficiently significant statistical regressions did result linear $\Delta u = d + e\,(\sigma_1 - \sigma_3)$, which would suggest a satisfactory application of an A coefficient in the case.

Similar comments would apply to other pore--pressure coefficients postulated. The principle is general, and, if applied, would eliminate reference to specific coefficients that are presently subject to misuse without exposure of the range of errors. A generalized suggestion would be to apply tests run under the stress path concept, and to allow statistical regressions to define the observed behaviour.

3.5.4 Preconsolidation pressure p_c of a stratum

Despite the general recognition of the importance of preconsolidation pressures to settlement computations and behaviour of buildings, and despite the multiple laboratory investigations directed at elucidating the consolidation behaviour of clays, it is quite apparent that the typical boring profile on any clay presents a major problem of decision to the design engineer when obliged to design for conditions closely approximating the preconsolidation pressures. A typical dispersion is illustrated in Fig. 33. Moreover, such generalized solutions for all soils, as suggested by Janbu (1963) should similarly be tested with respect to levels of confidence before they can be accepted as reasonable or improved substitutes to the admittedly poor situation.

To begin with, for the purpose of this Report the p_c value will be accepted as defining the pressure at which under a field loading, the compressibility of a clay stratum indicates a very rapid change, corresponding effectively to a break in the curve. In general terms the laboratory curves on reasonably undisturbed samples indicate a similar break, but at a pressure that we shall assume to be p'_c. And it is possible that the maximum past overburden pressure under which the clay may have been consolidated in geologic, geohydrologic, and geochemical history (Bjerrum 1967) may be yet a third value p_g. Rarely will the engineer be equipped through geologic information, to interpret the relationship between p_g and p_c, due to the multiple effects of leaching, desiccation, chemical bonds, thixotropic phenomena, delayed consolidation, and other effects difficult to quantify for purposes of predesign estimates: and, probably, the need to establish any connection between p_c and p_g lies beyond the need, possibility, or expectation of the engineer in most cases. The important problem is to try to establish the connection between p'_c and p_c, or, really, to establish an adequate estimate of p_c through whatever means may be available.



INDICATES RANGE OF PRECONSOLIDATION LOADS FROM CONSOLIDATION TESTS

A——A PROBABLE PRECONSOLIDATION LOAD

B- - -B INITIAL EFFECTIVE OVERBURDEN STRESS

Fig. 33

Illustration of dispersions in p_c values determined within clay strata (Apud Burn & Hamilton 1968)

What has been especially emphasized recently is the fact that there is no great difficulty in designing a structure for satisfactory behaviour either when the loading is entirely below the p_c value, or when the compressions to be anticipated be essentially along the virgin line of a normally-consolidated clay. The problems of unanticipated unsatisfactory behaviour arise when the loading is such as to lie mostly within the recompression condition, and only partly overshoot into the normally--consolidated condition.

This is comprehensible of one remembers that what is especially deleterious to a structure (cf. item 5) is the unanticipated differential settlements and rates of differential settlements affecting it when it is essentially ready and rigid. Thus, for instance, L. Casagrande (1964) showed very clearly for the simple case of a tank that the greatest differential settlements occur when the difference between the induced pressures under the center and under the rim are such as to straddle the p_c values, so that the center suffers accentuated virgin compression settlements while the rim lies within the relatively incompressible stretch of a recompression case. For the cases of reinforced concrete buildings, for instance, for which the live loads are of the order of 10 to 15 % of the total load, it was observed in São Paulo in connection with three buildings that were promptly subjected to corrective treatments, that considerable damages occur (with a load-settlement curve alarmingly simulating, for a while, a failure type of curve) under conditions in which the building essentially ready for occupancy, suddenly suffered a sharp increase in settlements and settlement rates; this was interpreted as a behaviour connected with the overtopping of the natural precompression of the underlying clays and loose slightly clayey sands.

Thus, very much depends on a determination as correct as possible of the p_c values. For a long while concern concentrated on the inability to predict satisfactorily the settlements of buildings on highly preconsolidated clays; but that was a preocupation of relatively little moment because observed settlements were always 2 to 4 times lower than those directly computed from standard oedometer results. The subject appears adequately solved (cf. Keinonen 1963, et al) by use of the second recompression curve of the oedometer as more representative of in-situ conditions, through suitable empirical evaluations (cf. Brinch Hansen 1964), or through more complex interpretation of the probable field recompression in comparison with the laboratory curve, by accepting the parallelism of unloading curves to compose a three-step log graph (horizontal-recompression-virgin compression) as suggested by Rutledge et al (cf. Schmertmann, 1955).

Subsequent laboratory research interest continued to be influenced by the practical observation that observed settlements tend to be smaller than computed ones, suggesting that $p'c < p_o$. Thus, all recent consolidation studies have apparently been directed at emphasizing the "quasi-consolidation effects" earlier detected by Leonards and co-workers (Crawford 1964, Raymond 1966, Bjerrum 1967, Brzezinski 1968, et al), and at the fact that the laboratory test procedures may give appreciably different results of $p'c$ depending on items of the test procedure such as load-increment ratios and duration of loading etc.. (Crawford 1964, Kenney 1968), as well as on the triaxial test as compared with the oedometer test (Kenney 1968). It cannot fail to be noticed, however, that such work was limited principally to comparisons between different p'_c values resulting from the effects under investigation, with no direct suggestions for improved methods of determining p'_c in a manner destined to reproduce more faithfully the preconsolidations imposed, or those p_c values eventually detectable by other means in the field.

From a practical point of view, therefore, whether they be quasi-consolidation pressures or other factors affecting the clay's $e - \log p$ curve, the fact is that until recently the engineer was equipped with but two methods (Casagrande's classical construction, and Schmertmann's, 1955) to determine the laboratory p'_c from a test result. New test procedures are being suggested, such as the constant rate of strain test (Simons 1963, et al) and constant rate of stress test (Hamilton and Crawford 1959, et al) for better determinations of p'_c. The only novel approach is that suggested by Bjerrum (1967), meriting special notice because of introducing into this subject the idea of field testing: ... "the critical pressure can also be determined quite reliably from the results of vane tests. ... The values of p_c computed from the vane tests show a reasonably good agreement with the results obtained from consolidation test .... An exception is leached clays of low plasticity where errors involved in the determination of the plasticity index produce large scattering of the p_c values determined from the vane tests."

The practical importance of an improvement in the situation can be assessed from the results reproduced in Fig. 33, wherein besides any possible constant error, a 95 % confidence range on the average of width of about 0.4 t/sq.ft. is involved, equivalent to the surface loading of about a 4-storey concrete-and-brick office building.

3.6 Some special engineering solutions to problems of settlements of foundations

In a state-of-the-art report on foundations on clays it is not possible to avoid mention of two important engineering solutions in very frequent use to attenuate or obviate problems of settlements: however, only cursory mention will be made since both appear to be employed with success, qualifiable as quite satisfactory, but on neither is there enough information to discuss difficulties with respect to ranges of confidence of design decisions, specifically applicable to these solutions as distinct from the basic principles on which they rely.

The first is the floating foundation. In a recent state-of-the-art paper Golder (1965) summarized the principal problems of such foundations as excavation, bottom heave, settlement and tilting, and structural problems. Mention is herein restricted to the problem of heave and bottom failure due to the excavation. If the excavation is not too deep and too slow, the heave of the bottom is essentially instantaneous, permitting a good measure of the modulus of elasticity of the clay (cf. Serota and Jennings 1959, and Skempton's correspondence on the paper; also Bozozuk 1963, Correa 1963, Girault discussions 1965, and many others). Obviously the foundation suffers immediate settlements as the load is reapplied, and, as should be expected, these tend to be a little larger than the heave. If the excavation exceeds a certain depth, determinable from bearing capacity considerations, a bottom heave failure can occur. Thus the depths of application of the floating foundation principle are limited by these phenomena, besides such problems as costs, bracing design and movements, and effects on adjacent buildings, which make the solution imperatively linked to construction procedures. Ground-water lowering and electrosmotic treatment (successfully applied in many other cases, e.g. Bjerrum et al, 1966) are the more indicated technical solutions available to the designer to obviate or attenuate the difficulties mentioned (e.g. Serota and Jennings 1959, Correa 1963, and many others). Lambe(1968) reports on a well-instrumented case of a building for which the stress-path method was used to estimate the heave, and indicates an empirical expression of heave as 0.5 % of the depth of excavation for three similar nearby buildings on Boston Blue Clay. If the heave were principally elastic (not too much affected by clumsy construction and delays) such an indication, disregarding the effect of plan dimensions, which should be expected to influence in essentially direct proportion, appears strange, since in practice it has often proved successful, though cumbersome, to resort to excavating in stages by small areas at a time. However, Lambe's data lead him to the conclusion that "most of the heave was time-dependent, i.e., consolidation heave". Actually, as was pointed out earlier, the "elastic" and "consolidation" deformations being really inseparable in time, the proportions of participation of each arbitrary component of the stress--strain-time function may vary considerably from case to case.

The second is the preloading solution. On this subject the state-of-the-art paper by Aldrich (1965) summarizes the principal techniques employed, time-settlement problems involved and methods of acceleration applicable, and the areas of further study required. It appears that the items of difficulty of design decision pertaining herein have already been covered in connection with consolidation and reconsolidation settlements, and principally in connection with dispersions that obtain in rate of settlement estimates, directly affecting costs in the engineering solution. The only additional factor is connected with the load-unload-reload cycle and the secondary compressions thereby accentuated. Bjerrum (1968) stresses the importance of large variations in live loads on secondary settlements. Laboratory tests (e.g. Simons 1965) indicate the advantages of a surcharge somewhat higher than the net foundation loading, with a view to absorbing some of these extra tendencies towards secondary compression in application of the preloading solution.

4 PILE FOUNDATIONS

4.1 Earlier routine design practices

Having discussed as carefully as possible, under item 2.5, the basic principles and knowledge that presently allow the design engineer to estimate the ultimate bearing capacity and the load test settlement behaviour of individual piles, it must still be recognized that the design of a pile foundation, and the estimation of its probable behaviour in supporting building loads, involves quite a set of additional problems.

In the earlier routine design practices within the problem, direct concern was restricted to two phases. Firstly, through the acceptance of some "Efficiency" formulae or factors, the groups of piles were designed for working loads per pile somewhat lower than admissible on the individual pile. Secondly, by assuming each column load to be distributed on the area circumscribed by the respective group of piles, at arbitrarily selected depths (e.g. 1/2 or 2/3 the pile length for friction or friction-point piles, or the full length of the pile for purely point-bearing cases), any stresses transmitted to underlying compressible soils would be computed by elastic theories or simplified geometrical distribution diagrams, and possible long-term consolidation effects of such underlying strata would be estimated (as described under item 3). Grillo (1948) began employing Mindlin's stress-distribution solution for the latter component (cf. Geddes 1966). The basic concepts were that the deformations of the zone of "effective embedment" could be treated by the Efficiency formulae referred to load-test data; and the underlying long-term compressibilities would constitute the only additional significant factor. Long-term pile-soil interaction within the "zone of effective embedment" was implicitly not considered.

For the first component, several rule-of-thumb Efficiency formulae were introduced. such as the Converse-Labarre formula, Feld's rule, etc. (cf. Chellis 1962), wherein the principle consisted in reducing the working load per pile when in a group, depending on the array of piles co-participating. "No reduction due to grouping occurs with end-bearing piles. For combined end-bearing and friction piles, only the load carrying capacity of the frictional portion is reduced. Reduction of bearing value is caused by overlapping zones of pressure around each pile. Tests have indicated that a spacing of at least 10 % of the length is required to avoid group-action reduction" (loc. cit. p. 671). Such rules are doubtless much in use, although it appears that the practice of altogether ignoring any group reduction factor is quite as widespread: obviously the success of such a practice depends entirely on the soils traversed and the piles used, factors that were not specifically considered in the rule-of-thumb methods.

Two further items of standard design practice that must be remembered and emphasized concern the examination of ultimate bearing capacity of the entire group (Terzaghi and Peck 1967 p. 538) considering the piles and the confined mass of soil capable of sinking as an overall "caisson" or pier: this only affects friction piles embedded in silt or soft clay, or cases of point-bearing piles that transfer their load onto a firm but thin stratum underlain by a thick deposit of soft clay. As regards settlements of such an "overall caisson", if it comprises piles driven through sands , or end-bearing on sands, the settlements may be estimated to vary with the dimensions of the area similarly to what obtains for rafts and footings (cf. Skempton 1953, Bjerrum and Eggestad 1963), if the compaction effects due to driving can be suitably estimated, to adjust for the part, usually negligible, of compressions within the volume of embedment (for a partial approach see Nishida 1961): if it comprises a hypothetical overall caisson bearing on clays, the settlements may be estimated, for the assumed bearing area, as discussed under item 3.

Further mention regarding the consolidation effects in underlying layers will not be necessary, since these can be adequately treated by consolidation theory if the stresses transmitted to the compressible stratum can be evaluated.

The short- and long-term effects that merit special consideration as distinguishing between single piles and groups of piles are naturally concerned with the zone of embedment, as will be considered in the following items.

4.2 "Immediate" load-settlement behaviour of groups of piles

The first problem that may be set aside as subject to investigation is that of estimating the behaviour of a group of "similar" piles, cooperating, under a rigid or flexible cap, in the support of a given rapidly applied load. The problem merited interest because of its evident connection with the concept of bearing capacity failure, and load tests; moreover it is thus a problem more easily investigable; finally, since in the case of single piles it seemed proven that the greater proportion of settlement occurs as the "immediate" settlement, a significant part of the overall group-action would be encompassed by the study of the "immediate" load-settlement behaviour In comparing with single pile load-settlement behaviour, the only difrence that arose since the earliest studies, was the prompt recognition that settlement and general failure considerations are inseparable, in group action, since, depending on lengths and spacings of piles in the arrays, the distinction between the predominance of the two phenomena becomes altogether arbitrary.

Early contributions to the treatment of the subject in a more rational manner resulted from model tests both in the laboratory and in the field. Whitaker (1957, 1960), Saffery and Tate (1961), Sowers and Fausold (1961), and Tate (1963) have made the principal contributions of laboratory model tests in clays.

Whitaker (1957) used very soft remolded clay of c 0.4 to 0.9 t/m^2 and 1/8" diameter brass rods in lengths of 12d, 24d, 36d and 48d, held to a rigid cap which will be called "Free-Standing" (Whitaker 1960) insofar as the cap did not rest on the soil during the loading tests. Square arrays 3^2, 5^2, 7^2, and 9^2 were tested in comparison with single piles, and the loads carried by each pile could be recorded separately. Spacings between piles were varied between 1.5 and 8 diameters. The piles were displacement piles, simulating driven pile conditions for the surrounding clay. The tests were analogous to the CRP load test, aiding in defining a maximum load independent of interpretations of failure based on settlements. The following terms have been defined, of which the first and the third have more practical significance to the design engineer.

$$\text{Efficiency Factor } EF = \frac{\text{average load per pile at failure of the group}}{\text{average load at failure for single pile}}$$

$$\text{Settlement ratio of failure} = \frac{\text{settlement of the group at failure}}{\text{settlement of the single pile at failure}}$$

$$\text{Settlement ratio at } Q_{50\%},\ SR = \frac{\text{settlement of the group at 50\% of its } Q_f}{\text{settlement of the single pile at 50\% of its } Q_f}$$

The results indicated clearly that there are two types of failure at play: at closer spacings, especially in the bigger groups, the failure takes place as a block; at larger spacings it takes place progressively with considerable redistributions of loads among the piles. As a result of this concept and observation Whitaker represents his results in pairs of intersecting lines. Unfortunately, as could be expected for "driven" piles, the order of installation of the piles had a considerable influence on this gradual redistribution of loads. Since this problem is dependent on so many factors not yet definable under working hypotheses for design (it varies with the order of driving, therefore varying vastly from soil to soil in accordance with "execution effects", varies with the loading, and varies with the structural rigidity of the cap), and since knowledge on it would concern principally the structural design of the cap, the subject will be excluded from further discussion herein.

Saffery and Tate (1961) also conducted similar tests using 1/4" steel rods of lengths 12d, 18d, 24d, 30d, for a single 2 array (they investigated further the problem of eccentric loading which will not be discussed herein), using a remolded clay of $c = 2 - 4\ t/m^2$. Initially through a series of tests on single piles of different lengths they obtained $\beta = 0.52$ for the shaft adhesion, and confirmed the anticipated trend that point bearing contribution decreases with length of pile (in their results the drop was from 27 % at 12d to 13 % at 30d, at failure). Sowers and Fausold (1961) report on similar model tests using a very soft clay of $c = 0.3$ to $0.6\ t/m^2$ and $\beta = 0.83$, for aluminium piles of diameters 0.5 and 1.2 inches, in lengths 12d, 24d and 36d, and in arrays of 2, 2^2, 3^2 and 4^2. The distribution of friction along the piles was measured by SR4 gages.

The results of these three investigations are so similar in general trend, and yet present so much relative scatter, that it is difficult to reproduce more than a sample of them without confusing the picture. For a summary indication of the principal findings Fig. 34 has been prepared.

The first item of importance is the Efficiency Factor, EF. The results show that higher EF occurs (a) for smaller piles (b) for greater spacings (c) for smaller numbers of piles in the group. Since piles smaller than about 20d are not of much interest, it can be summarized that the EF tends to be of the order of 0.7 to 0.85 for the (2.5 to 4)d spacings commonly employed, and there is very little increase of EF (at the

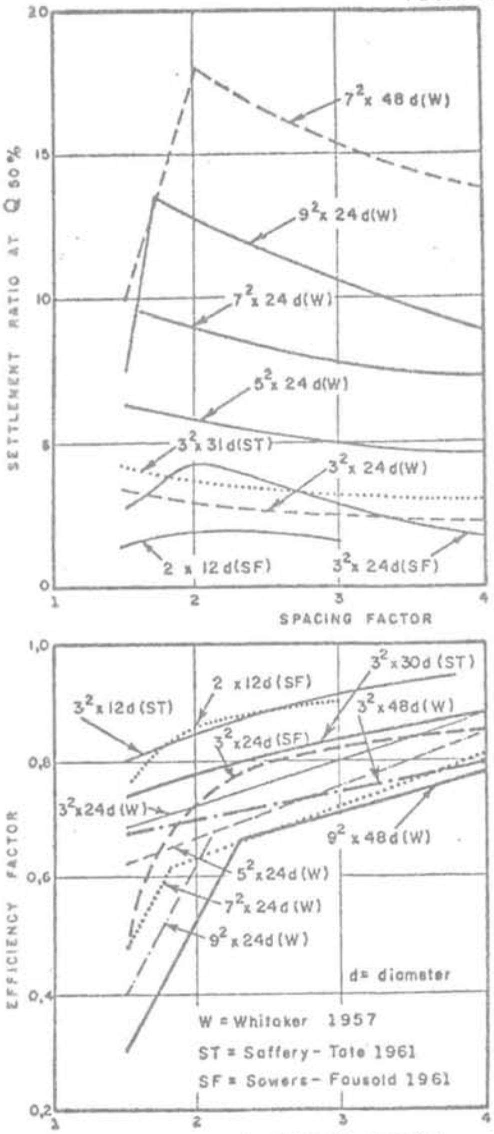


Fig. 34

Relationships for groups of 2^2 to 9^2 piles of lengths 12d to 48d, by model tests, free-standing

expense of flexibility of the pile cap or of exaggerated cost in its reinforcing) beyond that spacing, except for the big groups of deep piles.

The second item of practical interest is the Settlement Ratio SR at the likely working load $Q_{50\%}$. The results summarized in Fig. 34 are quite within the expected trend, showing appreciable increases of SR with length and number of piles, and some smaller-order attenuation with increased spacing beyond 2d. Worthy of notice is the fact that the SR values ranged between the order of 3 to 5 frequently cited from practical experience, to about 15 for the worst cases tested.

Whitaker (1960) later repeats the investigations with a view to including the influence of the pressure applied on the homogeneous soil by the pile cap as it settles together with the piles. Thus he distinguishes between so-called "Free-Standing" group of piles as he describes the earlier investigations, and the more realistic "Piled Foundations". The comparative results of EF and SR are presented in Fig. 35. It is seen that under these conditions EF does reach values higher than 1, although at correspondingly higher SR values. Koizumi and Ito (1967) ran field loading tests under conditions analogous to Whitaker's "piled foundations", measuring the reaction at the base of the pile caps both for the single pile and for the 3^2 group. For the 5 m piles it was observed that about 12 % of the load was carried by the cap at Q_{50}, and about 28 % at the Q_f. Tate (1963) also furthers his investigations, principally to evaluate differences in behaviour of displacement piles driven as earlier, compared with piles driven in pre-bored holes of 0.75d. Two-inch diameter tube piles are used, and the sequence of driving in the array is also investigated summarily in a comparison between two sequences for a 2d spacing and 3^2 group. For the remoulded clay used a very noticeable effect was observed (up to 40 % of the maximum) in individual pile capacities of the group, although the average of the entire group did not change much.

Complementing these laboratory tests, mention must be made of some field model tests available, although under somewhat less controlled conditions regarding soils, and apparently influenced principally by compaction effects within the sandy strata. Cambefort (1953) pushed 25 m long steel rods of 3.3 m and 5.0 cm diameters into a subsoil profile compressing an upper clay layer and a lower fine sand. For the spacing of 2d and 3d measured EF values were of the order of 1.2 to 1.7 in groups of two to seven piles, an optimum tending to occur with groups of 3 piles. The central pile had a remarkably greater capacity than the others, which tallies with an interpretation of compaction effects. It must be noted also that the load tests on the groups were compared with the penetration

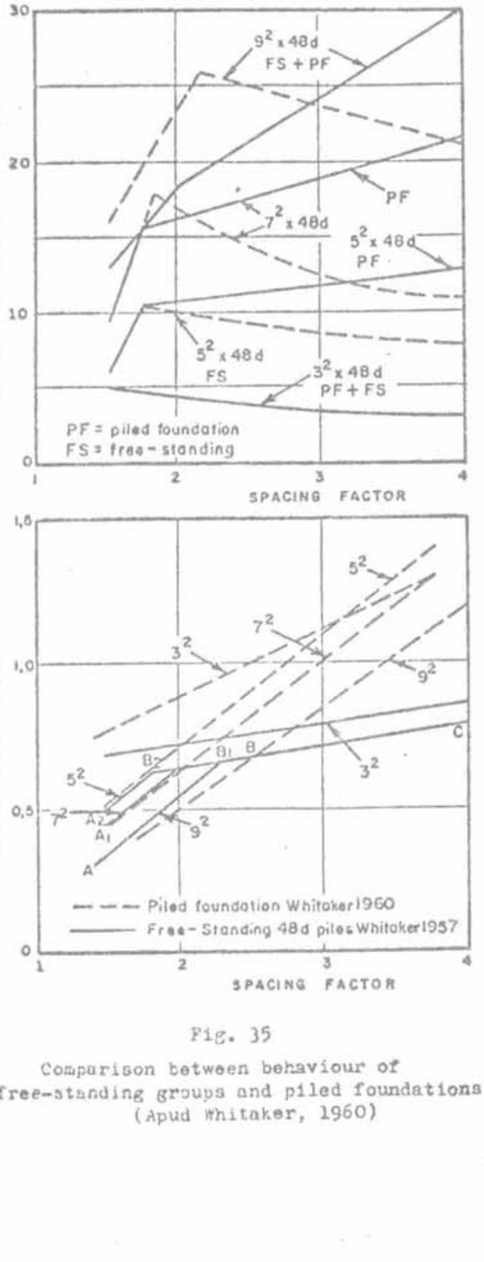


Fig. 35

Comparison between behaviour of free-standing groups and piled foundations (Apud Whitaker, 1960)

resistances measured on the individual piles as pushed-in: therefore some significant part of the increased EF may be due to strength gain generally observed on piles within a short duration after "driving". Kedzi (1959) reports, for a fine silty sand, two sets of tests on fours piles (2 m long and 0.1 x 0.1 m^2 in section) once in a row and once in a square array, having observed EF ≈ 2. Once again the result is connected with compaction of sand, being an optimum at about (2 to 3)d spacing and dropping back to EF = 1 at a spacing of about 6d. The fact that shorter piles give higher EF (Whitaker 1957) must also be observed. Berezantzev et al (1961) also report on load tests both in the laboratory and in the field, on groups of piles 20d long of lengths 3.6, 5.6, and 7.5 m, driven in sand: they indicate that the bearing capacity of the larger groups depends mainly on settlement, but only on first loading; "If reloaded, the relationship between settlement and load for a single pile group does not alter appreciably from single piles". Stuart, Hanna and Naylor (1960) and Hanna (1963) report similar general conclusions of EF ≈ 1.2 to 1.5 and SR ≈ ≈ 2 to 10 for groups of model piles driven in sands, but wisely remind that "simple extrapolation of the model trends is difficult in the absence of field confirmatory tests. It is believed that the compaction of and sand strength properties are the chief causes of scale differences between model and field tests". In this respect it should be reminded that different types of piles will give different results: for example Robinsky and Morrison (1964) prove that sand displacement and compaction around model friction piles was considerably higher for tapered piles than for straight piles.

In trying to analyze the problem under appraisal, of comparison between "immediate" behaviour of group vs. individual piles, a simplified subdivision of fundamental cases may be drawn up as follows:-

(A) Compare the cases of piles without execution effects: thus, for example, for the model tests discussed, the closest would be the use of non-displacement piles, in the direction sought by Tate (1963). This would eliminate the highly erratic contribution of execution effects, varying from soil to soil, as well as, significantly, with type of pile and time. It must be conceded, however, that since the principal model tests used remoulded clays and waited a few days after "driving", the

execution effects may be similar, and of lesser importance. Furthermore, preferably both the FS and PF cases should be treated since there are cases in which either occurs, and there are other cases in which the effect of the cap should be separated because of a desiccated crust of clay, etc.. Finally, elastic compresions of the pile should be considered as far as possible.
(B) Examine the interference on the above conclusions, introduced by different "immediate" execution effects.

Concerning (A) we may draw up the following comparisons of available theoretical solutions and model tests. For the moment let us restrict attention to friction piles in a homogeneous medium. Using the abreviations: R_P = rigid pile; FP = elastic pile; F_S = freestanding group; PF = piled foundation; RC = rigid cap, equal ρ piles; FC = flexible cap, equal Q piles. The cases are:

- A
  - A.1 = RP
    - A.1.a = FS — { A.1.a.I = RC ; A.1.a.II = FC }
    - A.1.b = PF {repeat analogously
  - A.2 = EP
    - A.2.a = FS — { A.2.a.I = RC ; A.2.a.II = FC }
    - A.2.b = PF {repeat analogously

Theoretical analyses recently put forth indicate the vast potentiality offered by mathematical formulations, through Mindlin's solution for an elastic medium (it is recognized that up to loads of about 80 % of the ultimate, pile behaviour is reasonably elastic), considering an elasto-plastic behaviour of the shaft adhesion, and employing computer solutions as already indicated under item 2.5.2. For the cases of group action of the piles themselves two solutions are known. Pichumani and d'Appolonia (1967) basically consider case A.2.a.I. The results are presented principally for the friction pile with a finite depth of clay deposit of constant E and τ_f, for comparison with the available model tests: but the method is applicable also to end bearing conditions, and to a medium in which E varies with depth. The results for constant E are shown to compare well with Sowers and Ramsold (1961) for clays (assuming negligible point resistance, and β = 0.6, which dispense with more complex analysis of pile-soil strain interaction): also the results for varying E are found compatible with the sand model tests by Hanna (1963) and others.

On the other hand Poulos (1968) basically considers the cases A.1.a.I and A.1.a.II, both for the case of a semi-infinite elastic medium (constant E) and for the case of a boundary at a finite depth. The theoretical curves for settlement ratio SR are derived for ratios of h/L and L/d where h = thickness of the elastic medium, and L = length of the pile of diameter d, as before. It is shown that all the model test data for the 2^2, 3^2, 4^2 and 5^2 groups scatter essentially between the pairs of curves derived for ($h/L = \infty$, L = 25d) and (h/L = 1.5, L = 25d). Since the analysis is "elastic" it is applicable to estimation of SR values throughout the range of linear stress-strain behaviour: however, the Efficiency Factor EF cannot be formulated. A Group Reduction Factor is studied on the basis of settlement considerations, and found to be in accord with the indications of group settlements (Skempton 1953) above mentioned.

Still under the item of "immediate" effects, mention is yet made of solutions by Nishida (1960) and Nair (1967). Nishida applying the basic equations of equilibrium to the elastic medium develops equations for the induced stresses in the soil by the piles, and arrives at essentially the already mentioned conclusions, further states that in a saturated clay no effects should be felt beyond 8d, and that the pile groups will begin to fail as a block when spacings are smaller than 2.8d. Nair (1967) applies the Mindlin idealized case to superpose effects and consider the consequences of group action on stresses and strains in the surrounding medium. Both of these contributions, and other of a similar nature would come principally under subdivision (B) above mentioned, but in this area the adoption of the idealized mathematical model cannot possibly account for the extreme variations bound to occur depending on well-recognized factors of soil and pile behaviour.

4.3 Execution effects in, and due to pile foundations

In the analytic approach to interpreting and forecasting the behaviour of pile foundations, the earlier procedures based on the use of overall adhesion factor β and group efficiency factor EF and settlement ratios SR, are being substituted, as is understandable, by the attempt to establish separately the "immediate" stress-strain behaviour of piles and pile-groups (simulating load test conditions) and the execution effects, both short-term and long-term.

The former can be rationally treated if only the appropriate soil parameters are introduced. But in the light of the designer's problem, the execution effects are known to alter the soil parameters predetermined in the subsoil investigations. Thus, the study of execution effects would have to responsibility of permitting a more rational interpretation of behaviours differentiatable from case to case, not only as qualifying the soil properties necessary for forecasting the "immediate behaviour", but also in establishing the trends of subsequent long-term behaviour.

This is clearly a complex area to which the bulk of the dispersions has thus been shifted and wherein some data have been accumulated, but hitherto, it can be said, without any clear vision of the principal parameters involved which have to be observed and analyzed.

Notwithstanding the evident criticism of some overlap with the treatments of items above (e.g. the β factor obviously incorporates execution effects, and item 4.2 above also involved them as an implicit conditioning), the fact is that an attempt must be made to interpret and forecast how different piles and pile groups create different effects in different soils, both within the immediately surrounding soil affecting the behaviour of the piled foundation itself, and also in a more distant surrounding soil, affecting adjacent structures. Moreover, whereas in the earlier analysis it had been useful for the sake of theoretical developments and model tests, to separate "immediate" from "long-term" effects, as regards execution effects it is not practicable to separate the two: for instance, although a consolidation effect is usually treated as a long-term effect, in the case of many a pile there is irrefutable evidence that a large percentage of the consolidation can take place in a few days.

The principal factors at play would seem to be, (a) a remolding effect immediately surrounding the pile, or, in more general terms, so as not to restrict merely to sensitive clays, we may consider it an effect of partial or complete alteration of the very soil material at play: this effect should bear some relation to the density of the displacement piling to be driven, causing the intense shear distortions, and to the effects of the latter on the properties of the material (remoulding of sensitive clays, destruction of cementing bonds in lateritic clays, and so on); (b) secondly, a complete simultaneous alteration of the stresses in soil elements in the immediate vicinity of the pile; (c) thirdly, a dissipation of the excess pore-pressures in the vicinity of the piling and its surroundings, with consequent settlement and strength-regain phenomena; (d) and finally, a set of long-term unclarified phenomena of thixotropic regain in the soil, and pile-soil interactions.

With respect to (a) the interest has been in detecting the scale of the remoulding effects felt, and the distances to which they spread. Item (b) has been of interest principally in connection with analyses of some of the boundary conditions under which the phenomena of item (c) will begin. Item (c) has been observed both in connection with the piled foundation itself, and with respect to influences on adjacent structures. Item (d) has not received any direct attention.

4.3.1 Remoulding around displacement piles

The observation on the extent of remoulding has been direct, by visual observation of the distortions of striations of the deposit, in undisturbed samples taken after the pile driving, and indirect, by comparison of the original undisturbed strength with the strengths at several distances from the face. The latter procedure however incorporates the obvious interference of time and strength recovery after remolding (unless special precautions are taken to attenuate the effect, such as, for instance, Orrje and Broms (1967) in the Brunnsbo tests, by coating the concrete piles with an asphaltic solution to prevent drainage of excess pore water through the piles). It must be emphasized that a truly direct observation of the extent of remoulding should be important because whatever properties are attributed to the soil contiguous to the pile, will certainly bear closer resemblance to the remoulded (partially)material than to the undisturbed material: and there is no difficulty in introducing such testing into design routines. Such observations are very few, and not consciously related to volume displacement of the driven piling, although a routine design measure when such effects are feared is to resort to steel piling or even pre-augering (e.g.

Lambe and Horn 1965). Cummings, et al.1950) reporting on a group of cast-in-place displacement piles, observed the conditions of striations within the samples, as well as differences in strengths: they conclude that close to the pile there is about a 50 % remoulding (compared to the laboratory total remoulding) and that partial remoulding extends to about 2d. Zeevaert (1957) reports for driven piles in Mexico City clay a skin of 0.05d thickness of completely remoulded material, and on the basis of a plot of undrained strengths several months after pile driving concludes that the initial undisturbed strength value is applicable at a distance of 1.0d from the face. Rubinsky (1957) reporting on a single pile driven in a varved clayey silt of W_L = 27 %, W_p = 17 % and S_t = 1.5 to 3, states that "visible stratum disturbance was found most pronounced in the uppermost samples where volume displacement was greatest,Maximum distortion occurred within 8 inches (≈ 0.7d) of the pile and at 18 inches (≈ 1.5 d) almost no disturbance would be discerned." Vey (1959) reporting on a steel H-pile driven in a clay of S_t = 1.8 - 2 observes no loss of strength. Seed and Reese (1957) reporting on a well-instrumented 6-inch diameter cased pile driven in an insensitive clay, conclude that at the pile face the strength loss corresponds to about 67 % of that corresponding to a complete remolding. Eide et al (1961) having taken undisturbed samples that skimmed the surface of a pile state "The zone of vertical layering was seen to extend to at least 6 cm from the surface of the pile shaft: the total thickness of the disturbed zone there must be several times greater than this". Orrje and Broms (1967) report on four test piles at Brunnsbo in a clay of S_t = 30 to 50, concluding that strength losses were from 40 % to 0 % up to 1.5d: however, at Backa with a clay of S_t ≈ 28 to 65 in the case of a big group of piles the average τ_f measured after the driving of the piles was about 90 % of the initial; since in this case very rapid dissipations of pore pressure were observed it is not possible to judge how much of this conclusion concerns the recovered strength upon reconsolidation.

In conclusion, it is feared that there is no satisfactory indication at present as to how much remoulding will take place around a displacement pile and how it might relate to soil properties and to the displaced volume (e.g. Bjerrum and Johannessen, 1960, report essentially the same u, equivalent to total overburden pressure, within the mass of soil surrounding two driven pile clusters when the concrete piles of equivalent displacement of 3 % were substituted by steel piles of equivalent displacement of 1.1 %). Moreover, assuming partial remouldings, laboratory testing techniques will have to be developed to simulate such conditions so as to permit the determination of the appropriate geotechnical parameters for design.

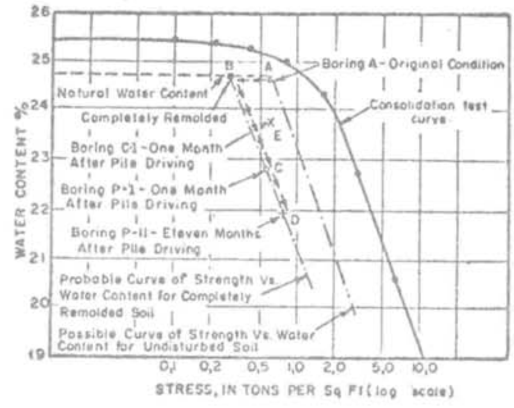


Fig. 36

Interpretation of strength regain of remolded clay around displacement pile (Apud Rutledge 1950, discussion Ref. 62)

Fig. 36 from Rutledge's discussion to Cummings et al. (1950) exemplifies a procedure that has been frequently employed to establish the probable remolded and reconsolidated clay strength: it is based on the premise of the parallelism (in virgin compression) of the e vs. $\log \tau_f$ and the consolidation curves. Seed and Reese (1957) apply the same premise to partial remouldings, by establishing the initial point for the parallel line depending on the percentage strength loss estimated.

4.3.2 Stress changes near and around the pile.

The second point in question regarding the execution effects of driven displacement piles is under what stresses and boundary conditions will the partially remoulded clay immediately surrounding the piles be reconsolidated. It has been frequently assumed that the final reconsolidation will be under the preexisting vertical effective stresses without any significant change in the lateral stresses, which will indeed be valid at some distance away from the pile group. The field observations on this important point until recently suffered from two significant limitations: one was to limit interest to observing u, and the other was to measure u at varying though small distances from the pile face. Only recently is the necessary importance being attached to measurements of both total stresses and pore pressures at the very face of the pile, to investigate the important boundary condition.

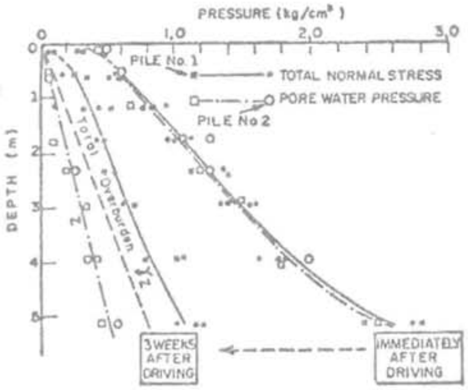


Fig. 37

Total stress and pore pressure measurements at the face of 30 cm diam. piles driven into soft clay (Apud Koizumi & Ito, 1967)

Hanna (1967) and Koizumi and Ito (1967) have published results on measurements of u at the very face of the pile during driving, and have thereby demonstrated that the clay "liquefies" (i.e. pore pressures attain values equal to or even higher than the overburden stress) around the pile although these excess pore pressures dissipate very rapidly both with distance from the pile face and with time. Figs. 37 and 38 herein reproduced from the papers mentioned furnish an idea of conditions at the boundary, and of how during the very time of execution of a piling project the rapid dissipation of these excess pore pressures is at play. These indications must be borne in mind in interpreting earlier data on measured pore pressures, insofar as they are meant to reflect conditions around the pile itself, (cf. Bjerrum and Johannessen 1960, Mogami and Kishida, 1961, Milligan et al. 1962, Lo and Stermac 1964, Orrje and Broms 1967, among others).

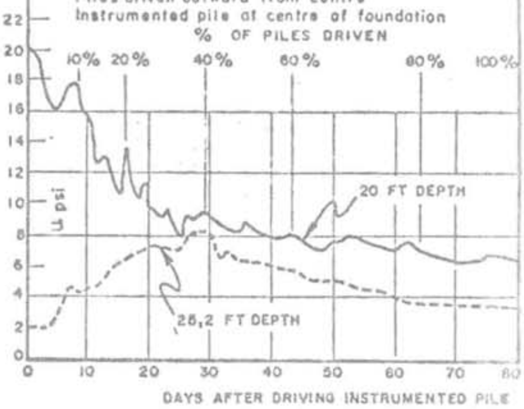


Fig. 38

Pore water pressures due to pile driving illustrating rapid dissipation. (Apud Hanna, 1967)

Thus from a designer's point of view the first problem that arises is the estimation of the stresses that may be created around a pile immediately after driving. Soderberg (1962) interested in developing a theory for consolidation effects due to piles recognized that the basic problem was the postulation of the initial pore pressure distribution at the pile face and made the assumption that the excess pore pressure is proportional to the radial stress imposed by the pile driving. This assumption was, however, criticized. Ladanyi (1963) used the theory of expansion of cavities in clay to estimate the excess pore pressure distribution around a pile driven in a saturated clay during and immediately after driving, and compared it with published measured values. Lo and Stermac (1965) present a theory for estimating the maximum pore pressure due to pile driving assuming a radial displacement of a soil volume, the radial stress becoming the major principal stress: the method involves the determination of the coefficient of earth pressure at rest and the maximum pore pressure ratio (measured is the conventional CU test with pore-pressure measurements, as the value $\Delta u/p$, which increases with applied stress difference, reaching a maximum and remaining constant after a certain strain).

The second problem concerns the consolidation theory itself, for calculation of the probable rates of dissipation of the excess pore pressures considered. Soderberg(1962) offers a solution, considering an impervious pile and purely a radial diffusion problem. Poulos and Davis (1968) develop a solution for cases within the applicability of elastic theory, for piles either permeable or impermeable, and for various ratios of length to diameter of pile, both for a semi-infinite clay medium and for finite depths. The confirmation of these theoretical results constitutes an interesting task for the immediate future.

Finally, the third problem comprises the evaluation of the strength recovery with time. Insofar as this strength regain will be connected with dissipation of pore pressures, the above solution furnishes and approach to the problem. However, field evidence demonstrates that the subject involves additional complex phenomena. It is frequently observed that the final strength may be higher than the initial (Peck 1965), and especially that in the vicinity of the pile the strength regain may be significantly higher than in the middle of the mass of clay (e.g. Rubinsky 1957).

4.3.3 Execution effect on adjacent structures.

The pore pressures created by the piles driven will naturally be transmitted to the subsoil below adjacent structures, thereby causing problems. Bjerrum and Johannessen (1960) have considered the effects of these pore pressures on the stability of slopes within which the bridge abutment piled foundations were executed. Lambe and Horn (1965) present data on excess pore pressures measured under an existing building, due to the adjacent pile driving, and use the stress-path method to calculate the resulting heave and subsequent settlement. Although the computed results did not tally closely with the observations (possibly due to sampling disturbances etc.), the methods for employing the data for estimating the consequences of such transmitted excess pore pressures, are presumed not to involve additional difficulties. The basic problem lies in anticipating the Δu that will be created, as above discussed.

4.3.4 Execution effects of different pile types.

Not enough attention has been devoted to comparisons of execution effects due to different pile types. For instance, Lo and Stermac (1964) compare timber piles with a Franki pile, but merely with respect to adhesion values in a stiff clay. Chadeisson (1961) compared for bored piles the use of drilling muds as against a driven casing, and concluded that the bearing capacity of the pile in the first case was somewhat better than in the second. Lambe and Horn's (1965) paper refers to partly preaugered piles: however there appear to be no other cases of bored piles as against displacement piles, and none that permit establishing a quantitative estimate of the importance of the volume of soil displaced by displacement piles.

4.4 Negative skin friction

Within the subject of negative skin friction loading on piles, and the consequent reduction of available load-supporting capacity of the pile, simultaneous with increase of the pile-foundation settlements, there has been a gradual increase in the recognized complexity of the factors at play. Unfortunately, meanwhile, there is record of only one fully instrumented prototype-scale test (Johannessen 1965) with which to compare hypotheses: moreover that was a particular case of a steel pile point-bearing on rock, it suffered from interferences of an adjacent slide and of bending stresses introduced in the pile, and the interpretation apparently could not extend beyond an indication on the applicable general formula for the ultimate drag adhesions developed.

Moretto and Bolognesi (1959) indicated the effect that the negative friction has in reducing the vertical stresses consolidating the clay, while Zeevaert (1959) further emphasized the simultaneous effect, of this reduction of vertical effective stresses around point-bearing piles in sands, in decreasing the point bearing capacity, thus postulating conditions of a dynamic equilibrium of the pile penetrating into the end-bearing stratum. Buisson, Ahu and Habib (1960) postulate a "neutral point" at some depth along the pile, such that above it the clay layer is settling with respect to the pile and develops negative friction, whereas blow it the net movement of the pile is of sinkage with respect to the surrounding clay so that a positive friction would develop. A graphical procedure is indicated for solution of the problem, requiring: (a) knowledge of the load-settlement curve of the pile (during which the total positive adhesion is assumed to have acted); (b) knowledge of the pressure-settlement (consolidation) behaviour of the clay; (c) postulation of two simultaneous equations for extracting the two unknows (Q_p, assumed constant, and depth of the neutral point), these equations being that of static vertical equilibrium and that of equality of vertical movements of pile and clay at the neutral point. In cases of large differences in clay and pile deformabilities (e.g. steel piles to rock) the neutral point would be located at the bottom of the layer.

For the case of comparable large deformations of the pile and the clay, Zeevaert (1963) further develops his concept of the dynamic equilibrium of the pile for the special conditions of friction pile foundations in Mexico City, treating the large-deformation case as essentially within plasticity conditions. Many different cases of loadings (of pile and clay layer), and respective relative deformations, are compared (simulating construction phases and different design cases), with the resulting uniform adhesion stresses on the piles depicted for each case. For the case where the soil has a downward velocity (unloaded piles within a consolidating clay mass) a neutral point at mid-height is postulated, with a uniform full negative friction above, and a corresponding uniform full positive skin friction below: equilibrium is maintained, but with the pile sinking at a velocity corresponding to that of the neutral point. It is apparent that the simplified plastic state analysis tends towards an incongruent friction distribution in such cases. The use of the theory in the design of foundations is described, as well as the advantages of such dynamically behaving pile foundations both as regards reducing excavation heave, improving bearing capacity of the upper layers of soft clays, and reducing consolidation settlements, within the very special problem of major subsidence and foundation settlements that obtain in Mexico City. Data are furnished confirming satisfactory correlations of computed and observed behaviour of some buildings under the different construction phases, as well as over a nine year period of settlement of a building.

Poorooshasb and Bozozuk(1967) develop a mathematical solution of a "kinematically admissible displacement field describing the movement of the particles of a clay body surrounding a single prebored end-bearing pile" considering a uniform clay underlain by bedrock and subjected to a surface loading.

Finally, as mentioned before, Brinch Hansen (1968) formulates a general theory for skin friction on piles, and Mazurkiewicz complements it with a state-of-the-art discussion, accompanied by model tests, on skin friction of model piles in sands. The publication cannot be profitably summarized, since the treatment is very general and covers many cases. For the purpose of the discussion that the topic will merit, it may be summarized that the negative skin friction, on piles due to a surface loading p placed after the execution of the piles, is given in terms of three limiting conditions as follows: (a) one corresponding to the limit such that at no point along the pile the compression of the soil (due to p and due to Δu from execution effects) change into expansion by the decreased vertical stress? (b) one corresponding to the pulling-up resistance of the pile, which cannot be exceeded; and (c) one, for the case of groups, that the total negative friction on a pile cannot exceed the surface load per pile. Calculations are exemplified in comparison with Johannessen's (1965) data, proving the applicability of the theory.

The subject is certainly very important, and subject to considerable additional testing, discussion, and development.

4.5 Some special piles for special purposes

Considering the many problems above summarized in connection with the execution effects and the subsequent behaviour of pile foundations in clays, obviously a great number of ingenious ideas have developed for special cases. The following inevitably incomplete summary may serve as but an illustration of some that are available for consideration in the face of given design problems.

The use of slender steel point-bearing piles was early introduced for minimizing remoulding effects in sensitive clays, and the fact that the allowable loading suffers practically no reduction because of buckling, makes it a very successful solution for many cases (e.g. Bjerrum 1957). More recently such slender metal piling has been used as friction piling, with the very successful use of electrosmosis to increase the adhesion several-fold (e.g. Soderman and Milligan 1961, Tamez 1964, et al.).

At the other extreme, several ideas have been successfully applied to reducing negative skin friction: for instance, a step-tapered pile with cross-section increasing upwards (Resendiz 1964), a mixed cast-in-place and precast pile such that the precast element inserted into the hole is surrounded by a loose sand filling before withdrawal of the shell (Locher 1965), or even the use of especially studied bituminous coatings (Hutchinson and Jensen 1968). Mohan and coworkers (1967) have introduced the multiple under-reamed pile for development of greater bearing capacity. Finally, under the unusual conditions that obtain in Mexico City, the special automatically controlled-load pile ("Pilotes Control®) has been successfully developed for reducing settlement to accompany the surrounding general subsidence.

On all such cases the semiquantitative feel aided initial design decisions, and experience subsequently accumulated furnishes a measure of the confidence level sought.

5. ALLOWABLE SETTLEMENTS

Initially it must be clarified that the "allowable settlements" of a foundation must be established on the basis of the two criteria, that of imminent failure conditions of the foundation itself, and that of intolerable conditions to the functional behaviour of the structure. The considerations summarized under item 2.8 with respect to deformation failure criteria applied to plate load tests must be extended also to other foundations, explaining, for instance, the justifiable tighter settlement criteria of most Codes in routine interpretation of load tests on piles: if a 5 mm settlement on a given pile is indicative of proximity to failure, obviously the allowable settlement would have to observe such a rigid criterion, irrespective of the fact that for the same structure a different foundation may be permitted a settlement ten times greater. However, by the same reasoning it will be concluded directly that since the settlements at proximity to failure vary from one pile type and dimension to another, it will be desirable to revise such codified prescriptions so as to qualify the single generalized criterion. No further discussion will be spent on this item, therefore, though it is emphasized as an important preliminary condition: the discussions of items 2.5 and 2.8 will provide bases for appraisal of the recommendable approach.

The subject that concerns this final item of design decision is the allowable settlement, as seen from the point of view of tolerable conditions to the functional behaviour of the structure .

Once again, as a start, the best present-day practice is herewith presented: it may be taken essentially as synthesized by Bjerrum (1963) in the indications reproduced in Fig. 39, although both the author himself simultaneously, and numerous other

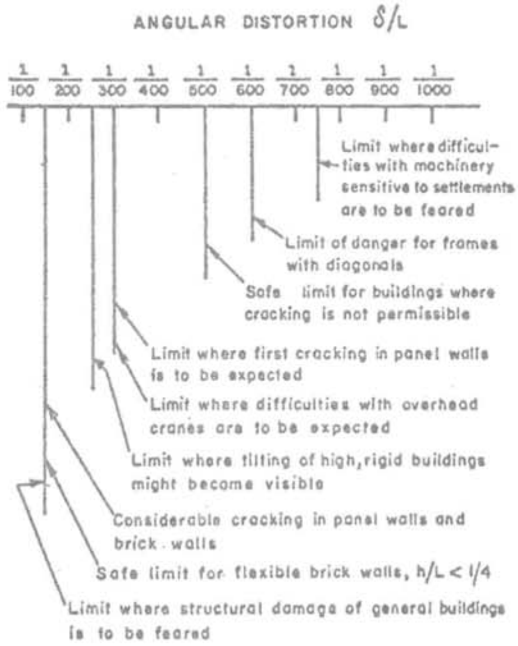


Fig. 39
Damage criteria (Apud Bjerrum 1963)

contributions, both earlier and later, have repeatedly alerted to a number of important factors to be considered jointly, as will be briefly discussed below.

The engineer should at this stage have arrived at a set of settlement computations involving probable (average) values, as well as upper and lower confidence bounds: moreover, the values that are of direct interest from such computations involve many aspects, such as settlements, differential settlements, rates of settlements, and so forth. However, for the sake of simplicity and abbreviation of the explanation, throughout the remainder of the discussion reference will be limited to a single pattern of "working hypothesis" computational results, and, for a preliminary formulation of concepts involved the single term "distortion" will be used to encompass the family of related differential settlement parameters of direct interest.

The decision problem is: what magnitude of computed distortions $\Delta\rho_1$ will be acceptable to the structure ? Obviously it is a problem on which direct, uninterpreted observation of existing structures may furnish a confused picture. There are two basic steps at play. First, there is the problem of a possible correction factor η between computed and real settlements, under strictly analogous office to field situations: for instance, if a uniform (flexible) loading was assumed for calculation of settlements, such cases as tanks would supply the bases for establishing η, so that, for similar cases of flexible uniform loadings assumed for buildings, it could be concluded that the real distortions should be $\eta\Delta\rho_1$. The second, independent, step comprises the observation, on a particular type of semi-rigid structure, of the distortion $\Delta\rho_2$ which causes the threshold condition of cracking.

What is the relation between $\eta\Delta\rho_1$ and $\Delta\rho_2$? It depends on the rigidity and resistance of the structure as reflected through its capacity to accept secondary stresses until reaching the distortion $\Delta\rho_2$, with consequent redistribution of the initial loads: obviously, if the structure has cracked with the distortion $\Delta\rho_2$, ipso facto there are redistributed stresses at play which have changed the anticipated $\eta\Delta\rho_1$ values, so that $\eta\Delta\rho_1$ and $\Delta\rho_2$ are not equatable. If by some iterative computations one could redistribute the applied loads until the computed distortions are equal to $\Delta\rho_2$, we might possibly, by a considerable simplification of the problem, establish a kind of distortion-redistribution factor λ, pertaining to the structure under appraisal, such that $\eta\lambda\Delta\rho_1 = \Delta\rho_2$. Thereupon the design decision would be taken on the basis of the estimated value of $\eta\lambda\Delta\rho_1$, and the knowledge that the tolerable limit has been observed to be $\Delta\rho_2$. It may be noted that under such a situation the maximum computed settlement ρ_1 which would be expected to be transformed into $\eta\rho_1$ for field conditions will suffer a change to a value ρ' which need not have any relation to $\eta\lambda\rho_1$: in fact, in changing from a totally flexible to a perfectly rigid case the maximum (center) settlement would reduce to about 80 % whereas the distortion would reduce to zero. However, we may write that the maximum expected settlement $\eta\rho_1$ would be changed to a value $\eta\lambda_1\rho_1$.

The correction factor η would depend on the many errors, cumulative and/or compensating, discussed earlier, and therefore cannot be dissociated from local practices of sampling and testing, as well as from the computational procedures, stress-distribution hypotheses. Further, it would be different for cases of normally consolidated clays and of preconsolidated clays. As regards exclusion of hidden interference of the λ factor, only the ideal cases can be compared, such as a flexible uniform loading computation versus a tank or an ideally rigid footing (with its appropriate influence factor) versus such structures as tall chimneys or rigid silos. The case of compressions of very deep clay layers may also be employed, under the approximation that for such cases $\lambda \approx 1$ because the stresses distributed to great depths by either flexible or rigid footings are the same.

As regards the λ factor, assuming $\eta = 1$, it is clear that if a structure cracks with real distortions $\Delta\rho_2$, it means that it could be designed for greater computed $\Delta\rho_1$ values (for flexible loading): therefore $\lambda < 1$, λ being the reduction factor due to rigidity by which through redistribution of applied loads the computed flexible-load $\eta\Delta\rho_1$ will be attenuated as the secondary stresses tending to cause cracking develop.

Finally, as regards observed values of $\Delta\rho_2$ on structures it would almost seem that each case will be a distinct one, depending on construction materials and techniques, design standards (reflecting capacity to absorb additional stressing without damage), significance of the distortions to the portion of the structure existing at a given time, rates of distortion and corresponding creep behaviour of the structure, and multiple other parameters. But these observations could be collected by the myriads, many of them from the same structures at various floor levels, and are equally valid for all factors causing the assumed structural distortion, be they foundation heave or settlement (due to whatever causes), be they due to any deformations from purely structural causes.

Soil mechanics has already for almost two decades been collecting information on settlements of structures, with a view to revising and developing the theoretical concepts. However, in retrospect it can be stated that much of the work as published cannot contribute to generalized knowledge on the problem, simply because it does not permit evaluation of the separate contributions from the above three basic lumped parameters postulated. Thus, whenever observed and calculated settlements are reported pertaining to cases on which η is not separately known or estimated, and/or $\lambda \neq 1$, the conclusion will refer to a presumed mixed correction factor $(\eta\lambda)_p$.

Essentially similar formulations to that of the separate η and λ corrections have been presented by Polskin and Tokar (1957) and Peynircioglu (1961), although apparently without attempting to distinguish between λ (referred to distortions) and λ_1, (referred to maximum settlements).

A brief summary will herein be attempted of some of the bibliography surveyed, in order to appraise the present condition with regard to the three parameters η, λ and $\Delta\rho_2$ needed for the final design acceptance of a flexible-loading computed $\Delta\rho_1 \lessapprox \Delta\rho_2/\eta\lambda$.

The most cited basic reference on the subject is the paper by Skempton and MacDonald (1956) which included also a summary of earlier pronouncements. An ample statistical survey gave indications on $\Delta\rho_2$ (the much quoted limits of 1/300 for load--bearing walls or for panels in traditional-type frame buildings, and so on ...). Furthermore, statistical correlations were developed for observed $\eta\lambda_1\rho_1$ values versus observed $\Delta\rho$, but although these cases were roughly separated according to foundation conditions and building types, the collection of data having been from very widely different sources, there are no real means of compensating for the unknown $\eta\lambda_1$ factors so as to connect the information with the ρ_1, which is the sole implement of decision available to the engineer at this phase.

The subject subsequently received treatments of broad coverage from Polskin and Tokar (1957) and Bjerrum (1963), and has been discussed qualitatively from many facets by McKinley (1964), while Feld (1965) formulates an up-dated summary of the quantitative data from most of the earlier sources.

Polskin and Tokar (1957) define a "coefficient of accuracy" as the ratio of "actual subsoil deformation to the value determined by calculation" (employing the methods of Tsytovitch, Yegorov, Vios I, Vios II) and, by an analysis of observed cases conclude that "when the coefficient of accuracy is from 1.3 to 1.8 (for various design methods) the calculated settlement will not be less than the actual value in eight out of ten cases. Thus, the settlement obtained by a given method of calculation must be increased approximately one and a half times, after which the revised value of ρ should be compared with the allowable limit value ρ_{lim} ...". Thereupon, illustrating the reasoning with respect to brick dwellings, they plot data on nine points for walls of ratios $2.4 < l/h < 4.8$ some of which cracked and others did not, and so establish a boundary for the tolerable linear distortion $\Delta\rho_2$ which they interpret to increase linearly with l/h, from 0.0003 for the $l/h = 2$ to 0.001 for $l/h = 8$. The prescriptions of the USSR 1955 code are summarized, which are interpreted to have been similarly established through direct observation of $\Delta\rho_2$. It is seen that the implicit assumption is that the lumped correction parameter $(\eta\lambda_1)_p$ for computed settlements (maximum, average, or which ?) would adequately reflected the $\eta\lambda$ correction factor on distortions, desired for relating the measured limiting $\Delta\rho_2$ with the $\Delta\rho_1$. Is such an assumption valid, and within what degree of accuracy ?

Bjerrum's (1963) damage criteria (Fig. 39) are interpreted to result from field observations, i.e. $\Delta\rho_2$ values. Besides mention of important first-order parameters (such as rates of settlements) qualifying those $\Delta\rho_2$ criteria, Bjerrum specifically points to the decision problem of the engineer, and lists four categories of factors that contribute to differential settlements affecting this decision. As regards the present discussion he furnishes indications principally towards the estimate of λ, by plotting points of maximum $\Delta\rho$ vs. maximum ρ (observed) and by establishing two distinct upper limiting curves, one labeled for flexible structures and the other for rigid structures (i.e. practically rigid). A comparison of the two curves would suggest an essentially constant limiting value of λ of 0.60 to 0.66 for the entire range of observed settlements up to 45 cm, which would imply that for "rigid" structures admissible computed distortions could be 50 % higher than the $\Delta\rho_2$ criteria. However, it is not clear whether the boundaries indicated for the "rigid" case are connected with any damaging distortions. Moreover as he well points out the position of the plotted points is influenced by the rigidity of the structure (not easy to define), and by the "depth of the seat of settlement", lower differential settlements being obtained when the compressible layer is at greater depth.

Rethati (1961) emphasizes the importance of sudden distortions, and establishes damage criteria on the basis of 12 buildings (1 to 6 storeys), in which he claims to observe the role of rigidity inasfar as 91 % of the buildings that suffered damage were up to 3 storeys high only. He thus suggests a linear increase of the permissible $\Delta\rho_2$ from 1:300 for 1 storey to about 1:80 for 6 storeys. The result calls for explanations besides that of redistribution of stresses, since the reference is to $\Delta\rho_2$ values. Would it be connected with the difference between the original stresses of the construction materials, the stress levels to which the measured $\Delta\rho_2$ would carry them, and any different capacity to absorb such stressing without cracking ?

Several publications have been dedicated specifically to the problem of computing structurally the iterative stress and load redistribution problem in connection with differential settlements (Chamecki 1956, Grasshof 1957, Krsmanovic discussion 1961, Sved and Kwok 1963, Getzler 1968, et al.). But apparently the method, although straight-forward in principle, has not yet had application. It appears that by resorting to computers it would not be difficult to develop charts for λ correction factors for typical cases: the principal problems appear to be the difficulty at establishing rigidities due to brick panels (although there is already a vast amount of collateral experimental data that would permit formulating working hypotheses), and the need of introducing the problem of rates of distortion and creep, without which the computations lead to absurd results. In this regard special reference is made to Fjeld's (1963) observations and calculations for a 3-storey concrete-framed structure with masonry filling that suffered severe cracking and furnished much information on $\Delta\rho_2$, but led to discouraging results of structural calculations.

Within this subject of allowable settlements, special mention must yet be made of the publication by Jennings and Kerrich (1962,'63) which focusses attention on the problem as closely connected with widths of cracks and maintenance techniques and costs; thus the really determining $\Delta\rho_2$ values are themselves highly variable depending on "levels of acceptance" which can well be equated, statistically, with economic analyses.

Finally, an important question that may be asked. What has the profession learnt since the early years of settlement computations, as a result of the great number of publications on observed versus calculated settlements of buildings ? And with what level of confidence can this knowledge be generalized and transplanted from case to case ?

The answer can only be formulated through a qualitative feel, based on evidence that within similar situations overall errors can be limited to about ± 20 %, while quite a number of publications attest to the fact that cases beyond the routine may present surprises up to 1/2 to 2 times the anticipated values. The conclusion is therefore that the art has not yet arrived at the stage of fairly confident generalization.

There is a wealth of data available for reexamination, but most of the data as published are incomplete for such reexamination, and, if taken at face value, lead to the conclusion that "judgment" is always present as a compensating factor so that, locally, the same overall agreement can be claimed for clearly different cases, or clearly different techniques. Indeed, most cases that have been published relate situations such that $(\eta\lambda)_p \approx 1$, and therefore, depending on the value that may applicable to λ, it would be necessary to recognize a value $\eta \neq 1$: moreover, η is inseparable from the details of all the steps taken to reach the final computed results. Unless these steps are fully described and the margins of errors in each step impartially exposed to statistical reappraisal, the desired generalization will not prove fruitful.

6 CONCLUSION

Throughout the report attention has been focussed on the problems and errors attendant to steps in the design sequence, principally with a view to catalysing the reappraisal from which additional developments may sprout. However, no pessimism in the state of the Art is implicit in the approach undertaken. A great number of truly outstanding cases of buildings and other projects successfully designed and constructed on clays, under conditions so adverse as to challenge responsibility to the point of daring, attest to the fact that there has been a very considerable progress in the field.

However, the great complexity of the factors at play and the dispersions and errors involved in each of the steps leading to such a conclusion, result in a situation wherein a very considerable portion of the success lies within the factor of judgement, much associated with local and individual experience. It is felt that only a systematic reanalyses of every facet through statistical treatments, destined to exclude personal factors and to distinguish first-order influences from secondary parameters, will permit the profession to cull from such multiple and routine attainments, the generalized lessons for an appraisal of the intuitions and experience contained within such judgement.

SOMMAIRE

1. Introduction. L'analyse séquentielle et la discussion des niveaux de confiance des décisions de l'ingénieur devant un projet de fondations ont été les deux facteurs principaux qui ont orienté l'élaboration de ce rapport général. Chaque problème spécifique s'entoure d'un grand nombre d'éléments dont l'ingénieur doit étudier l'importance relative afin d'en faire la synthèse, éliminant ceux qui à chaque instant puissent appartenir au groupe de facteurs d'ordre secondaire. Telle analyse et synthèse résultera d'une façon claire seulement grâce à l'emploi des méthodes statistiques pour l'examen des données disponibles.

2. Capacités portantes. En dépit des tendances modernes d'unifier le comportement tension-déformation-temps de problèmes de portée diverse, l'ingénieur doit examiner séparément les problèmes de rupture et de tassements, chaque cas en fonction des conditions critiques respectives, le premier entraînant une responsabilité plus grave. La capacité portante des semelles est calculée le plus couramment en faisant appel aux formules de Terzaghi (1943), Meyerhof (1951, 1963), et Brinch Hansen (1961, '66), et, dans le cas d'argile saturée, à celle de Skempton (1951). Les nombreuses théories mathématiques développées entretemps se résument en fait à de variations peu significatives de la théorie de Prandtl-Reissner, comme le prouve l'analyse statistique. En outre, la dispersion de valeurs obtenues dans la détermination de la cohésion, dues aux modes d'échantillonnage et aux méthodes d'essais, est de loin supérieure à celle du coefficient N_c. Le seul facteur très important mis au jour dernièrement est celui de l'influence de la compressibilité sur les coefficients de la capacité portante (cf. Vesic, 1965). Une analyse rapide du domaine d'intérêt pour l'ingénieur, de données sur la capacité portante des sols, fait apparaître que l'on dispose presque seulement de nombreux résultats sur les argiles saturées ($\phi = 0$). Il est à souhaiter de réunir plus de données concernant de cas général (c, ϕ) particulièrement dans le domaine de $\phi < 25°$ pour lequel les études statistiques indiquent des niveaux de confiance bien inférieurs pour les coefficients N_q et N_γ.

On résume les pratiques courantes de calcul de la capacité portante de pieux individuels et les données disponibles pour la détermination de la résistance de pointe $c\,N_{cp}$, et celle de frottement, à partir du coefficient d'adhésion moyen β. Il apparaît que les niveaux de confiance pour l'application de paramètres uniques, moyens, sont relativement bas. On résume les méthodes de Seed et Reese (1964), et al, de détermination de courbes chargement-tassement de pieux, ainsi que diverses méthodes d'analyses basées sur la théorie de l'élasticité et la formule de Mindlin (ex. Davis et Poulos 1968). On discute également les méthodes d'exécution et d'interprétation des essais sur pieux. Enfin, on commente d'autres essais tels que du pénétromètre statique et du pressiomètre : en ce qui concerne la grande variation de résultats de résistance à la pointe R_p du pénétromètre dans les argiles, on analise les données sur l'argile de Londres qui démontrent la nécessité d'utiliser dR_p/dz pour les régressions statistiques en fonction de $d\tau_f/dz$; telle méthode permet de réduire de nombreuses dispersions et semble confirmer la relation $R_p \approx 10c$. On résume les nombreuses études publiées sur les pieux foncés, moulés en place et épanouis, dans les argiles de Londres, en insistant sur les précautions à prendre pour l'application à d'autres argiles de ce riche dossier d'informations et recommandations.

On critique l'emploi courrant du Standard Penetration Test sur les argiles, soulignant une importance prononcée de la sensibilité des argiles qui exige des corrélations q_u vs SPT différentes pour chaque formation argilleuse.

On discute l'importance de la compressibilité dans l'interprétation de la rupture et du taux de travail admissible à partir d'essais de chargement sur plaques. On suggère d'abandonner la solution des coefficients N' de "local shear" et de la substituer par des coefficients de correction que l'on pourrá formuler au moyen d'analyses élasto-plastiques, en ayant recours à l'ordinateur électronique.

3. Tassement de fondations superficielles.

On résume la méthode de Skempton-Bjerrum (1957) de calcul des tassements soulignant la difficulté de choisir les paramètres A et B appropriés. Comme progrès récents, il faut noter la méthode du vecteur tension--déformation-temps de Lambe: cette méthode, théoriquement idéale, présente cependant certaines difficultés dans la pratique dues à la limitation du nombre d'essais qui ne pourront décrire l'hétérogénéité de la couche de sol. Deux autres développements récents comprennent la théorie tridimensionnelle élastique de Davis et Poulos (1968), et une méthode semi-empirique de Brinch Hansen (1965) justifiée par la conception que les trois subdivisions classiques du tassement total son arbitraires.

Diverses solutions mathématiques du phénomène de consolidation, soit par le modèle rhéologique de Terzaghi soit par d'autres modèles rhéologiques plus complexes, n'apportent de contribution effective à la pratique du à l'inexistence de développements nécessaires colatéraux en appareils et méthodes d'exécution et d'interprétation d'essais.

L'essai de consolidation triaxiale est interprété d'une manière similaire à celle de l'oedometre. Parmi les facteurs importants dans le calcul des tassements, on discute d'une part la détermination de la pression de consolidation, sujette à des erreurs appréciables, qui peuvent avoir d'importantes répercussions sur les tassements différentiels, et, d'autre part, les difficultées de déterminer les paramètres nécessaires à la définition de la courbe temps--tassement. Pour la détermination de la pression de consolidation on souligne l'importance des essais in situ: une mention intéressante est celle de l'emploi heureux de l'essai Vane pour telle finalité (Bjerrum 1967).

On décrit de façon résumée les deux principales solutions de project appliquées avec succès afin de réduire les tassements de bâtiments: la solution de fondation flottante et celle du pré-chargement.

4. Fondations sur pieux. Alors que dans la pratique on utilisait des formules de conception intuitive "d'efficacité" afin de réduire le taux de travail par pieu d'un groupe de pieux, des essais en modèles de groupes permettent aujourd'hui de comparer le comportement "immédiat" de pieux groupés avec celui d'un pieu isolé grâce à de coefficients de correction appelés "Coefficient d'efficacité" et "Rapport de Tassements" (à 50 % de la charge de rupture). On résume les études expérimentales de Whitaker (1957, 1960), et al, ainsi que les indications obtenues à partir des analyses théoriques de Pichumani et d'Appolonia (1967), et de Poulos (1968), toutes relatives à effets "immédiats". Ensuite, on discute les effets dus à la construction d'un groupe de pieux, mettant en évidence les facteurs qui influencent le propre groupe et les effets sur la masse de sol environnante. Les innombrables indications sur le degré de remaniement observé ne permettent pas encore une généralisation en fonction du volume d'argile déplacée par les pieux. Des observations indirectes basées sur des mesures de u au sein du sol avoisinant permettent seulement une évaluation semi-quantitative des problèmes. Les pressions interstitielles se dissipent très rapidement, tant dans l'espace que dans le temps. Des perspectives encourageantes se font jour suite à des observations simultanées de pressions totales et des pressions interstitielles mesurées sur les propres faces des pieux (Koizumi et Ito 1967), car de telles mesures fournissent les conditions de frontières nécessaires au développement des études de la consolidation (Soderberg 1962, Poulos et Davis 1968, et al). A respect des études concernant le frottement latéral négatif, on prend conscience graduellement de la complexité des facteurs en jeu à cause de la réduction simultanée des tensions verticales au voisinage du pieu. Zeevaert (1963) propose, pour les cas de grands tassements d'ensembles pieux-sols argileux, comme c'est le cas à Mexico, un équilibre dynamique de pieux flottants dans un état plastique, en accord avec le tassement général de l'ensemble, celui-ci étant toutefois atténué par conséquence inévitable. La théorie générale des frottements sur pieux proposée par Brinch Hansen (1963) accompagnée par l'étude de Mazurkiewicz sur le frottement négatif dans le sable, constitue une mise au point qui sert de base aux discussions et recherches futures.

A titre d'exemple on cite quelques types spéciaux de pieux qui ont été développés afin de solutioner des problèmes exceptionnels, (emploi de l'électrosmose pour augmenter l'adhérence de pieux flottants, usage de peinture bitumineuse pour diminuer la charge de frottement négatif, etc.).

5. Tassements admissibles. Les décisions de l'ingénieur doivent être basées sur des distortions Δf_1 calculées (on emploie le terme générique "distortion" Δf_1 pour tous les facteurs associés considérés aujourd'hui comme de première importance). En premier lieu il faut rechercher un coefficient de correction η applicable à des cas purement flexibles ou totalement rigides, afin d'adapter les valeurs calculées aux déformations mesurées. On ne peut dissocier un tel facteur de toute la séquence d'erreurs, hypothèses et décisions qui varient d'un endroit à l'autre et même d'un individu à l'autre. On met en évidence qu'il est de pratique courante de déterminer la relation η_1 entre les tassements calculés et mesurés; toutefois, d'une manière générale on ne peut mettre en équation les coefficients η et η_1; d'ailleurs l'intérêt principal réside dans la valeur de η relative aux distorsions. On propose ensuite un facteur de correction λ dû à la raideur partielle d'une ossature, correspondant à la redistribution des tensions qui s'installent jusqu'à la fissuration admissible, en accord avec les observations Δf_2 (pour chaque type d'ossature, vitesse de distorsion, etc., mais toutefois essentiellement indépendant de la cause des déformations). Une fois connu Δf_2 pour une ossature donnée, l'ingénieur pourrait accepter des distorsions $\eta \lambda \Delta f_1 \leq \Delta f_2$. En

général, $\lambda < 1$, donc les projets peuvent être calculés pour des valeurs de $n \Delta \rho_1$ supérieures aux limites $\Delta \rho_2$ observées.
On résume finalement les valeurs $\Delta \rho_2$ indiquées par Bjerrum (1963), aussi comme les informations comparatives entre ossatures "rigides" et "flexible" qui paraissent indiquer un coefficient $\lambda = 0,65$. On souligne encore que très peu de publications permettent de cueillir séparement des information sur n, λ et $\Delta \rho_2$.
6. Conclusion. Les succès spectaculaires obtenus dans de très nombreux cas de fondations complexes sur argiles prouvent l'existence d'une importante contribution de l'élément "jugement", lié à l'experience locale, qui compense les grandes marges de dispersions et d'erreurs dans toutes les étapes qui conduisent au projet d'un ouvrage. Seulement l'analyse statistique permettra de definir des critères moins subjectifs pour mettre en valeur, dans la pratique de la profession, les grands progrès déja obtenus, qui ont été publiés sous la forme de cas particuliers mais qui sont encore difficilement applicables à des conditions plus générales.

BIBLIOGRAPHY

1. Aas, G., 1965. "A study of the effect of vane shape and rate of strain on the measured values of in-situ shear of clays". 6th Int. Conf. Soil Mech. Found. Eng., Canada, Vol I pg 141
2. Abbott, M.B., 1960. "One-dimensional consolidation of multi-layered soils". Geotéchnique, Vol 10 no. 4 pg 151.
3. Aboshi, H., & Monden, H., 1961. "Three dimensional consolidation of saturated clay". 5th Int. Conf. Soil Mech. Found. Eng., Paris, Vol I pg 559
4. Aboshi, H., & Monden, H., 1963. "Determination of the horizontal coefficient of consolidation on an alluvial clay". Proceed. 4th Australian-New Zealand Conf. on Soil Mec.Found. Eng., pg 159
5. Aldrich, H.. "Precompression for support of shallow foundations". ASCE Vol 91 no. SM2 pg 5 1965.
6. Amand, H.N. & Makol, R.L., 1963. "Comparison of triaxial compression test and field-plate-loading test for bearing capacity of soils". A.S.T.M., Materials Research & Standards pg 201.
7. Artikoglu, N.O., 1961. "Determining ultimate bearing capacity of precast reinforced concrete piles from deep sounding tests in Alsancak harbour. 5th Int.Conf., Paris, Vol II pg 3.
8. Ballisager, C.C., 1959. See refer. to Christensen & Hansen (56)
9. Baker, C. 1965. Discussion pg 119. A.S.C.E. Vol 91 no. SM6
10. Barden, L. 1965. "Consolidation of clay with non-linear viscosity". Geotechnique, Vol XV, no. 4 pg 345
11. Barden, L.,1968 "Primary and secondary consolidation of clay and peat". Geotechnique Vol 18 no. 1 pg 1.
12. Begemann,. H.K.S., 1953. "Improved method of determining resistance to adhesion by sounding through a loose sleeve placed behind the cone".3rd Int.Conf. Montreal Vol I pg 17.
13. Begemann, H.K.S., 1965. "The friction jacket cone as an aid in determining the soil profile" 6th Int.Conf.Montreal Vol I pg 17
13A. Bergfelt, A., 1956. "Loading tests on clay". Geotechnique Vol 6 pg 15.
14. Berezantzev, V.G., et al, 1961. "Load bearing capacity and deformation of piled foundations". 5th Int.Conf.Paris Vol II pg 11.
15. Bergfelt, A., 1957. "The axial and lateral load bearing capacity and failure by buckling of pilesin soft clay".4th Int.Conf. London Vol II pg 8.
15A. Biarez, J., et al, 1961. "Contribution à L'étude de la force portante des fondations". 5th Int.Conf. Paris, Vol I pg 11
16. Bishop, A., 1957. "Some factors controlling the pore pressure set up during the construction of earth dams". 4th Int.Conf., London, Vol II pg 294
16A. Bishop, A. & Henkel, D.J., 1957. "The measurement of soil properties in the triaxial test, London, Arnold.
17. Bishop, A. & Bjerrum, L. 1960. "The relevance of the triaxial test to the solution of stability problems". ASCE pg 437. Research Conf. on Shear Strength of Cohesive Soils.
18. Bishop, A. et al, 1965. "Undisturbed samples of London Clay from the Ashford Common shaft". Geotechnique, Vol 15 no. 1 pg 1.
19. Bishop A., 1966."Strength soils as eng'g mats. Geotechnique Vol 16 no. 2 pg 89.
20. Bishop, A. & Little, A., 1967. "The influence of the size and orientation of the sample on the apparent strength on the London Clay at Maldon, Esex". Proceed. Geotech. Conf. Oslo, Vol I pg 89.
21. Bjerrum, L., 1954. "Stability of natural slopes in quick clay". Norwegian Geot. Inst. publ. 10.
22. Bjerrum, L. & Øverland, A. 1957. "Foundation Failure of an oil tank in Fredrikstad, Norway". 4th Int.Conf., London, Vol I pg 287.
23. Bjerrum, L., 1957. "Norwegian experi- -

ences with steel piles to rock.Geotechnique, vol. 7, p. 73.

24. Bjerrum, L. & Johannessen, J., 1960."Pore pressures resulting from driving piles in soft clay". Proc. Conf. on Soil Suction in Soils, London, p.108.

25. Bjerrum, L. & Eggestad, A., 1963. "Interpretation of loading test on sand ". Proc. European Conf. Soil Mech.Found. Eng., Wiesbaden, vol. I, p. 199.

26. Bjerrum, L., 1963. "Discussion p. 6". European Conf., Wiesbaden, vol. III.

27. Bjerrum, L., 1963. "Discussion p. 135". European Conf., Wiesbaden, vol. III.

28. Bjerrum, L. 1966. "Problémes de fondation dans les argiles sensibles de Norvege". Norwegian Geot.Inst., bull. 67.

29. Bjerrum, L., 1967. "Engineering geology of Norwegian normally consolidated. marine clays as related to settlements of buildings". Geotechnique,v.12,no,2, p. 83.

30. Bjerrum, L., 1968. "Secondary settlements of structures subjected to large variations in live load". Norwegian Geot. Inst., Bull. 73.

31. Bjerrum, L., 1968. "Application of electro-osmosis to a foundation problem in a Norwegian quick clay ". Norwegian Geot. Inst., Bull,74.

32. Blight, G.E., 1967. "Observations on the shear testing of indurated fissured clays". Proc.Geot. Conf.,Oslo, p.97.

33. Bozozuk, M., 1963. "The modulus of elasticity of Leda clay from field measurements". The Can.Geot.Jour.v.I,no.1,p.43

34. Bozozuk, N., 1963. "Discussion". Geotechnique, vol. 13, no. 3, p. 259.

35. Brinch-Hansen, J., 1961. "A general formula for bearing capacity". Danish Geotechnical Institute, bull. 11.

35A.Brinch-Hansen, J., 1961. "A model law for simultaneous primary and secondary consolidation". Danish Geot.Inst,bull 13

36. Brinch-Hansen J., 1965. "Some stress-strain relationships for soils". 6th Int. Conf., Montreal, vol. I, p. 231.

37. Brinch-Hansen, J., 1966. "Comparison of methods for stability analysis".Danish Geot. Inst., Copenhagen, bull. 21.

38. Brinch-Hansen, J., 1968. "A theory for skin friction on piles". Danish Geot. Inst., bull. 25.

39. Broms, B.,1968. "End bearing and skin friction resistance of piles". A.S.C.E., v. 94, no. SM2, p. 421.

40. Burland, J. et al., 1966. "The behavior and design large diameter bored piles in stiff clay". Symp.on Large Bored Piles, London, p.51.

40A.Brzezinski, L., 1968. "The performance of a timber crib founded on a soft very sensitive lacustrine clay". Can. Geot. Jour., vol. 5, no. 1.

41. Duisson, M., 1960. "Le frottement négatif". A.I.T.B.T.P., Paris, no. 145.

42. Burmister, D., 1959. "Strain-rate behavior of clay and organic soils".A.S.T.M., USA, Sp.publ. no. 254, p. 88.

43. Button,S.J., 1953. "The bearing capacity of footings on a two-layer cohesive sub soil".3rd Int.Conf.Zurich,v.I,p.332

44. Cadling, L. & Odenstad, S., 1950. "The vane borer". Proc.Royal Swedish Geot. Inst., Stockholm, no.2.

45. Calderon, H., 1967. "Consolidation isotropica y desviadora en arcillas de la ciudad de Mexico. 3rd Pan.Am.Conf.Soil. Caracas, vol. I, p. 477.

46. Cambefort, H., 1953. "Le comportement des pieux forés et les ésais de pénétration" 3rd Int.Conf., Zurich, vol. II, p.29.

47. Cambefort, H., 1953. "La force portante des groupes de pieux". 3rd Int.Conf., Zurich, vol. II, p. 22.

47A.Cambefort, H. & Chadeisson, R., 1961. "Criterion for estimating the bearing capacity of a pile". 5th Int.Conf.,Paris, vol. II, p.23.

47B.Cambefort, H., 1965. "Pieux et groupes de pieux en terrain homogène". 6th Int. Conf., Montreal, vol. II, p. 238.

47C.Carlson, E. & Fricano, S., 1961. "Tank foundations in Eastern Venezuela". A.S.C.E., vol. 87, no. SM p. 69.

48. Casagrande, A., 1932. "The structure of clay and its importance in foundation eng'g".B.S.C.E., vol.I,contrib.1925-1940 p. 72.

49. Casagrande, L., 1964. "Effect of Pre-Consolidation on settlements". A.S.C.E., vol. 90, no. SM5, p. 349.

50. Casagrande, L., 1965. "Settlement of mat foundation on thick stratum of sensitive clay". Can.Geot.Jour., nov.,vol.II, no. 4.

51. Casagrande, L., 1966. "Sub-soils and foundation design in Richmond, Va.". A.S.C.E., vol. 92, no.SM5, p. 109.

52. Chadeisson, R., 1961. "Influence of the boring methods on the behavior of cast-in-place bored piles ". 5th Int. Conf. Paris, vol. II, p. 27.

53. Chellis, R., 1962. "Foundation Engineering". (Leonards, editor) McGraw-Hill.

54. Christie, I.F., 1965. "Secondary compression effects during one-dimensional consolidation tests". 6th Int.Conf., Montreal, vol. I, p. 198.

55. Christie, I.F., 1966. "The solution of consolidation problems by general-purpose analogue computer". Geotechnique, vol. XVI, no. 2, p. 131.

56. Christensen, N.H. & Hansen, B., 1959. "Bearing capacity of piles in Aarhus Septarian clay". Danish Geot.Inst., bull. 7.

57. Coyle, H. & Reese, L., 1966. "Load transfer for axially loaded piles in clay". A.S.C.E., vol.92, no. SM2, p. 1.

58. Cooke, R.W. & Whitaker, I., 1961. "Experiments on model piles with enlarged bases". Geotechnique, vol. 11, p. 1

58A.Correa, J.J., 1963. "Problemas de uma excavación profunda en arcilla de la ciudad de Mexico". 2nd Pan.Am.Cong.Soil., Brasil, vol. II, p. 189.

59. Crawford, C., 1953. "Settlement studies on the National Museum Building Ottawa, Canada". 3rd Int.Conf., Zurich,v.I,p.33

60. Crawford, C., 1962. "Settlement studies on the Mt. Sinai Hospital, Toronto. The Eng'g Journal, Canada, p. 31.

60A. Crawford, C. 1964. "Interpretation of the consolidation test ". A.S.C.E., vol. 90, no. SM5, p. 87.

61. Cummings, A. et al. 1950. "Effect of driving piles into soft clay". Trans. A. S.C.E., vol. 115, p. 275.

63. D'Appolonia, E. & Hribar, J. 1963. "Load transfer in a step-taper pile". A.S.C.E., vol. 89, no. SM6, p. 57.

64. D'Appolonia, E. & Romualdi, J. 1963. "Load transfer in end bearing steel H piles". A.S.C.E., vol. 89, no.SM2, p.1.

65. Dastidar, A.G. 1967. "Experience with a slicken-sided clay". Proc. of the 3rd Asian Conf.Soil Mech.Found.Eng., Haifa, vol. I, p. 63.

66. Davis, E.H. 1956. "A general theory of the undrained strength of soils and its application to stability analysis". 2nd Australia-New Zealand Conf.on Soil Mech. Found. Eng., p. 164.

67. Davis, E.H. & Poulos, H.G. 1963. "Triaxial testing, and three-dimensional settlement analysis". Proc. 4th Australia New Zealand Conf. p. 233.

68. Davis, E.H. & Raymond 1965. " A non-linear theory of consolidation". Geotechnique, vol. XV, no.2, p. 161.

69. Davis, E.H. & Poulos, B.E. 1968. "The use of elastic theory for settlement prediction under three-dimensional conditions ". Geotechnique, vol. XVIII, no. 1, p. 67.

70. de Beer, E. & Ladanyi 1961. "Experimental study of the bearing capacity of sand and circular foundations resting on the surface". 5th Inf. Conf., Paris, V. I, p. 577.

70A. Deere, D.U. & Davisson, M.I. 1961. "Behaviour of grain elevator foundations subjected to cyclic loading". 5th Int.Conf., Paris, V. I, p. 629.

71. de Beer, E.E. 1963. "The scale effect in the transposition of the results of deep-sounding tests on the ultimate bearing capacity of piles caisson foundations". Geotechnique, vol. 13, no.1, p39

72. de Beer, E. 1965. "Bearing capacity and settlement of shallow foundations on sand". Symp.on Bearing Capc.and Settlem. of Found. Duke University, p. 15.

73. Dinesh, Mohan, M. 1961. "Bearing capacity of bored piles in expansive clays". 5th Int.Conf., Paris, V.II, p.117.

74. Dominguez, A. & Moavenzadeh, F. 1967. "Stresses and displacements in semi-infinite media". 3rd Pan-Am.Conf.Soil Mech., Caracas, V. I, p. 145.

75. Drucker, D. C. 1953. "Limit analysis of two and three-dimensional soil mechanics problems". Journ.Appl. & Phys. of Solids, V. I, p. 217.

76. Dubose, L. 1956. "Discussion p.1028-7". A.S.C.E. no. SM3.

77. Dubose, L. 1957. "Small-scale load tests on drilled and cast-in-place concrete piles". Highway Research Board, U.S.A., vol. 36, p. 132

78. Earth Manual - U.S.B.R. 1963.

79. Eden, W.& Bozozuk. 1962. "Foundation failure of a silo on varved clay". The Eng'g Journal, Canada, p. 54.

80. Eden, W.J. & Poorooshasb, H. 1968. "Settlement observations at Kars Bridge" Canad.Geot.Journal, V.V, no.1, p. 28.

81. Eide, O. et al. 1961. "Short and long-term test loading of a friction pile in clay". 5th Int.Conf., Paris, V.II, p.45.

82. Egorov, K.E. et al. 1957. "The observed settlements of buildings as compared with preliminary calculations". 4th Int. Conf. London, V. I, p. 290.

83. Erb, ch. 1963. "Setzungsmessungen an Zwei Schornsteinen". European Conf.Soil Mech., Wiesbaden, V.I, p. 107.

84. Ertel, W. 1967. "Determination of the initial strength of stiff tertiary Frankfurt clay". Proc. Geot.Conf., Oslo, V. I, p. 109.

85. Escario, V. & Uriel, S. 1961. "Determining the coefficient of consolidation and horizontal permeability by radial drainage". 5th Int.Conf., Paris, V.I, p.83

86. Farrent, T.A. 1960. "The interpretation of vane tests in soils having friction". 3rd Australian-New Zealand Conf. Soil Mech. p. 81

87. Flaate, K. 1966. "Factors influencing the results of vane tests". The Canadian Geot.Journ., Canada, V. 3, no.1, p.18

88. Fleming, K. & Steger, E.H. 1966. "Large diameter pile foundations: settlement considerations". Symp.on Large Bored Piles, London, p. 73.

89. Fletcher, G.F.A. 1965. "Standard penetration test: its use and abuse". A.S.C.E., vol. 91, no. SM4, p. 67.

90. Florentin, J. et al. 1961. "Observations faites sur la craie comme couche de fondation". 5th Int.Conf., Paris, V.I, p. 101.

91. Flores, M. G. 1964. "Recimentacion y enderezado de una iglesia del siglo XVII por medio de pilotes de control em Mexico, D.F.". Congreso de Cimiento Profundo, Mexico, V. I, p. 345.

92. Francis, A. J. et al. 1963. "The behaviour of slender-point bearing piles in soft soil". Symp.on the Design of High Build. Hong Kong, p. 24.

93. Geddes, J.D. 1966. "Stresses in foundation soils due to vertical sub-surface loading". Geotechnique, V.16, no.3, p.231

93A. Getzler, Z. 1968. "Design of foundations under differential settling ". A.S.C.E. ST1, p. 53.

93B. Geuse, E. C. 1953. "Results d'essais de penetration en profondeu. et de mise en charge de pieux-modele. A.I.T.B.T.P., Paris, no. 63.64, p. 313.

94. Gibson, R.E. & Lumb, P. 1953. "Numerical solution of some problems in the consolidation of clay". Proc. of the Inst.of Civil Eng., London, p. 182.

95. Gibson, R.E. 1961. "The progress of Three-dimensional consolidation in an anisotropic clay stratum". Symp. on Found. Eng., Bangalore, p. 1

96. Gibson, R. E. 1967 "The theory of one-dimensional consolidation of saturated clays". Geotechnique, V. XVII, no. 3, p. 261.

97. Girault, P. 1965. "Discussion p. 182". A.S.C.E., vol. 92, no. SM1.

98. Golder, H. Q. 1941. "The ultimate bearing pressure of rectangular footings". Journal Inst.Civil Eng.,V.17, p.161.

98A.Golder, H. Q. & Leonard, M. 1954. "Some tests on bored piles in London clay". Geotechnique, V. 4, p. 32.

99. Golder, H. 1961. "Discussion p. 160". 5th Int.Conf., Paris, V. III.

100.Gorbunov & Possadov 1965. "Calculations for the stability of a sand bed by a solution combining the theories of - elasticity and plasticity". 6th, Int. Conf., Montreal, V. II, p. 51.

101.Gotolski, W. 1965. "Discussion p. 127". A.S.C.E., V. 91, no. SM5.

102.Goulet, G. 1964. "Comparison entre les resultats de chargement statique d'un pieu et les prévisions déduits des essais géotechniques". Sols-Soils, V.III, no. 11, p. 21.

102A. Grasshof, H. 1957. "Influence of flexural rigidity of super-structure on the distribution of contact pressure and bending moments of an elastic combined footing". 4th Int.Conf., London, V. I, p. 300.

103.Gray, H. 1945. "Simultaneous consolidation of contiguous layers of unlike compressible soils". A.S.C.E. Trans. V. 110.

104.Habib, M.P. & Ieheng, M.Y. 1961. "Force portante et déformation des fondations superficielles". A.I.T.B.T.P., Paris, no. 163-164.

105.Haefeli, R. & Bucher, H. 1961. "New methods for determining bearing capacity and settlement of piles". 5th Int.Conf. V. II, Paris, p. 65.

106.Hamilton, J.J. & Crawford, C.B. 1959. "Improved determination of preconsolidation pressure of a sensitive clay". A.S.T.M. Special publ. no.254, p. 254.

107.Hanna, W. S. 1953. "Settlements of - buildings on pre-consolidated clay - layers". 3rd Int.Conf., Zurich,V.I,p.366

108.Hanna, I.H. 1963. "Model studies of - foundation groups in clay". Geotechnique, V. 13, no. 4, p. 334.

109.Hanna, I.H. 1967. "The measurement of pore water pressures adjacent to a driven pile". The Canad.Geot.Journ., V.IV, no. 3, p. 313.

110.Hansbo, S. 1957. "A new approach to determination of the shear strength of clay by the fall-cone test". Proc.Royal Swedish Geot.Inst., Stockholm,no. 14.

111.Helenelund, K. 1953. "Settlement observations in Finland". 3rd Int.Conf., Zurich, V. I, p. 370.

111A. Holcomb, G. et al. 1953. "Building settlement measurements". 3rd Int.Conf. Zurich, V. I, p. 377.

111B. Horn, H. & Lambe, T.W. 1964. "Settlement of buildings on the M.I.T. campus A". A.S.C.E., V.90,no. SM5, p.181.

112.Hough, B.K. 1959. "Compressibility as the basis for soil bearing value". A.S. C.E., V.85, no. SM4, p. 11.

113.Hoshino, K. 1953. "Site exploration for founding a building by sinking method of a large-scale caisson". 3rd Int. Conf., Zurich, V. I, p. 384.

114. Hvorslev, J. 1949. "Subsurface exploration and sampling of soils".

115. Ireland, H. 1964. "Settlement of a friction pile foundation". Congreso de Cimientos Profundos, Mexico, V.I,p.373.

116. Jacobsen, M. 1967. "The undrained shear strength of preconsolidated boulder clay". Proc.Geot. Conf,Oslo, p. 119.

117. Jakobson, B. 1954. "Influence of sampler type and testing method on shear strength of clay samples". Royal Swedish Geot.Inst., no. 8,Stockholm.

117A.Jappelli, R. 1965. "Settlement studies of some structures in South Italy". 6th Int.Conf., Montreal, V. II, p.88.

118. Jappelli, R. 1967. " Sullostato plastico in terreno sollecitato da un carico inclinato uniformemente distribuito su di una retta del piano limite". Revista italiana di Geotecnica, fasc.4

119. Jappelli, R. & Tortorici, M. 1967. "Il regime plastico in un terreno sollecitato da carichi inclinati". Revista del Istituto de idraulica della Univ. di Palermo, Italia, no. 52.

120. Janbu, N. 1957. "Earth pressures and bearing capacity calculations by generalized procedure of slices". 4th Int. Conf., London, V. II, p. 207.

121. Janbu, N. 1963. "Soil compressibility as determined by oedometer". Proc. European Conf.on Soil Mech.Found. Eng., Wiesbaden, V. I, p. 19.

122. Jennings, J.E. & Kerrich, J.E. 1962-3. "The heaving of buildings and the associated economic consequences, with particular reference to the Orange Free State Goldfields", The Civil Eng. in South Africa, V. 5, no. 5, p. 122.

123. Johannessen, I.J. & Bjerrum, L. 1965. "Measurement of the compression of a steel pile to rock due to settlement of the surrounding clay". 6th Int.Conf. Montreal, V. II, p. 261.

124. Josselin de Jong, G. & Verruijt, A.1965 "Primary and secondary consolidation of a spherical clay sample". 6th Int. Conf., Montreal, V. I,p. 254.

125. Juarez Badillo, E. 1964. "Discussion". Congreso de Cimientos Profundos, Mexico, V. II, p. 479.

126. Jumikis, A. R. 1961. "The shape of rupture surface in dry-sand". 5th Int. Conf., Paris, v. I, p. 693.

127. Kantey, B. A. et al. 1953. "Field and laboratory determinations of strength characteristics of a normally consolidated clay in the Durban Bay- head area, South Africa". 3rd Int. Conf., Zurich, V. I,p. 241.

128. Karst, H. & Bourges, F. 1964. "Remblais sur sols compressibles". A.I.T.B.T.P., Paris, no.203.

130. Kaufman, R. & Sherman, W. 1964. "Engineering measurements for Port Allen Lock". A.S.C.E.,V.90,SM5,p. 221.

132. Kawakami, H. 1964. "The measurement of horizontal coefficient of consoli-

dation on by a triaxial testing apparatus". Soil and Foundation, V. IV, no. 2, p. 45. Japan.
133. Kjellman, W. et al. 1950. "Soil sampler with metal foils". Proc. Royal Swedish Geot. Inst., nº. I.
134. Keinonen, L. 1963. "Die Scheinbare vorbelastung der sensitiven Tonboden". European Conf. son Soil Mech. Found. Eng. Wiesbaden, V. I, p 127.
135. Kenney, C. 1964. "Pore pressures and bearing capacity of layered clays". A.S.C. E., V. 90, no. SM4, p. 27.
136. Kenney, C. 1968. "A review of recent research on strength and consolidation of soft sensitive clays". Canad. Geot. Journ., V. V, no. 2, p. 97.
137. Kerisel, J. 1961. "Foundations profondes en milieux sableux". 5th Int. Conf., Paris, V. II, p. 73.
137A. Kerisel, J. 1965. "Vertical and horizontal bearing capacity of deep foundations in clay". Symp. on Bearing Capacity and Settlement of Found., Duke University, p. 45.
138. Kerisel, J. & Quatre, M. 1968. "Settlements under foundations". Civil Eng'g. P.W. Review, London.
139. Kezdi, A. 1957. "Bearing capacity of piles and pile groups". 4th Int. Conf. London, V. II, p. 46.
140. Kezdi, A. 1961. "The effect of inclined loads on the stability of a foundation". 5th Int. Conf., Paris, V.I, p. 699.
141. Kondner, R. & Krized, R. 1962. "Correlation of load bearing tests on soils". Proc. Highway Research Board, V. 41, p. 557.
141A. Kondner, R. 1962. "Friction pile - groups in cohesive soils". A.S.C.E., V. 88, no. SM3, p. 117.
142. Kondner, R. 1963. "Hyperbolic stress strain response: cohesive soils". A.S.C.E., V. 89, no. SM1, p. 115.
143. Koizumi, Y. & Ito, K. 1967. "Field tests with regard to pile driving and bearing capacity of piled foundations". Soil & Foundation Revue, Japan, no.3, p. 30.
144. Kopacsy, A. 1961. "Stress distribution and slip surface on embankments". 5th Int. Conf., Paris, V. II, p. 641.
145. Krizek, R. J. 1965. "Approximation for Terzaghi's bearing capacity factors". A.S.C.E., V. 91, no. SM2, p.1.
146. Krsmanovic, M.D. 1961. "Discussion p. 210". 5th Int. Conf., Paris, V. III.
147. Kulkarni, K. R. et al. 1967. "A study of the properties of Bombay marine clay deposits in relation to certain foundation problems". 3rd Asian Conf. Soil Mechanics Found. Eng., Haifa, V.I, p.211.
148. Ladanyi, B. 1963. "Expansion of a cavity in a saturated clay medium". A.S. C.E., V. 89, no. SM4, p. 127.
149. Ladanyi, B. 1967. "Deep punching of sensitive clays". 3rd Pan-Americ. Conf. on Soil., Caracas, V. I, p. 533.
150. Ladd, C.C. & Lambe, T.W. 1963. "The strength of undisturbed clay determined from undrained tests". A.S.T.M., Spec. Tech. Publ. no.361, p. 342.
151. Ladd, C.C. 1964. "Stress-strain modulus of clay in undrained shear". A.S.C.E. V. 90, no. SM5, p. 103.
152. Ladd, C.C. 1965. "Stress-strain behaviour of anisotropically consolidated clays during undrained shear". 6th Int. Conf., Montreal, V. I, p. 282.
153. Lambe, T. W. 1964. "Methods of estimating settlement". A.S.C.E. V.90, no. SM5, p. 43.
154. Lambe, T. W. & Horn, H.M. 1965. "The influence on an adjacent building of pile driving for the M.I.T. Materials Center". 6th Int. Conf., Montreal, V. II, p. 280.
155. Lambe, T. W. 1965. "Shallow foundations on clay". Symposium on Bearing Capacity and Settl. of Found., Duke University, p. 35.
156. Lambe, T.W. 1968. "The behaviour of foundations during construction". A.S. C.E., V. 94, no. SM1, p. 93.
157. Lebeque, M. Y. 1964. "Étude expérimentale dès efforts d'arrachage et de frottement négatif sur les pieux en milieu pulvérulent". A.I.T.B.T.P., no. 199-200, Paris.
158. Leonards, G. A. & Girault, P. 1961. "A study of the one-dimensional consolidation test". 5th Int. Conf., Paris, V. I, p. 212.
159. Leonards, G. A. & Ramiah, B.K. 1959. "Time effects in the consolidation of clays". A.S.T.M. Spec.Tech.Publ. no. 254, p. 126.
160. Leonards, G. A. & Altschaeffl. 1964. "Compressibility of clays". A.S.C.E., V. 90, no. SM5, p. 133.
161. L'Herminier, M. H. 1957. "Étude des fondations en surface". A.I.T.B.T.P., Paris, no. 109.
162. L'Herminier, R. et al. 1961. "Fondations superficielles". 5th Int. Conf. Paris, V. I, p. 713.
163. Locher, H. G. 1965. "Combined cast-in-place and precast piles for the reduction of negative friction caused by embankment fill". 6th Int. Conf., Montreal V. II, p. 290.
164. Lo, K. Y. 1960. "Correspondence". Geotechnique, V. X, p. 37.
165. Lo, K. Y. 1961. "Secondary compression of clays". A.S.C.E., V. 87, no. SM4, p. 1.
165A. Lo, K. Y. 1962. "Shear strength properties of a sample of volcanic material of the valley of Mexico". Geotechnique, V. XII, no. 4, p. 303.
166. Lo, K. Y. 1967. "Shear strength properties of two stratified clays". A.S. C.E., V. 93, no. SM1, p. 1.
166A. Lo, K. Y. & Stermac, A.G. 1964. "Some pile loading tests in stiff clay". The Canad. Geot. Journ., V.I, no.2, p.63.
167. Lo, K. Y. & Stermac, A.G. 1965. "Induced pore pressures during pile-driving operations". 6th Int. Conf., Montreal, V. II, p. 285.

168. Lumb, P., 1965. "Interference of construction time on pore pressure dissipation beneath foundations". Civ. Eng'g & P.W.Review, May, p. 697.
169. Lumb, P., 1967. "Statistical Methods in Soil Investigation". 5th Aust. New-Zealand Conf., p. 26.
170. Lumb, P. and Holt, J.K., 1968. "The undrained shear strength of a soft marine clay from Hong Kong". Geotechnique, v. 18, no. 1, p. 25.
171. Lundgren, H. and Mortensen, K. 1953. "Determination by the theory of plasticity of the bearing capacity of continuous footings on sand". 3rd Int.Conf., Zurich, v. I, p. 409.
172. Mandel, J., 1963. "Interference plastique de fondations superficielles". Proc. Int.Conf., Budapest, p. 266.
173. Mansur, C. and Kaufmann, R., 1958. "Pile tests, low-still structure, old river, Lousiana". ASCE Trans., v. 123, p. 715.
174. Marsal, R., 1961. "Estudio sobre la prediccion teorica de asentamentos y de presiones de poro en suelos saturados". Rev. Inst.Ing., Mexico.
175. Martins, J.B., 1965. "Consolidation of a clay-layer of nonuniform coefficient of permeability". 6th Int.Conf. Montreal, v. I, p. 308.
176. Martins, J.B., 1965. "Capacidade de carga de Fundações". Lourenço Marques, Africa.
177. Mazurkiewiwz, B.K., 1968. "Skin Friction on model piles in sand". Danish Geot. Inst. Bull. 25.
178. Mc Kinlay, D.C., 1961. 5th Int.Conf. Paris, v. I, p. 225.
179. Mc Kinley, D., 1964. "Field observation of structures damaged by settlement". ASCE, v. 90, no. SM5, p. 249.
180. Mello de, Victor F.B. et al, 1959. "Construction period stability of homogeneous compacted earth dams". 1st Pan.Am.Conf., Mexico, v. II.
181. Mello de, Victor F.B. et al, 1959. "Some field correlations on dynamic penetration resistances in exploratory borings, by Geotécnica-Brasil". 1st.Pan.Am.Conf., Mexico, v.I, p. 959.
182. Mello de, Victor F.B., 1967. "Considerações sôbre os ensaios de penetração e sua aplicação a problemas de fundações rasas". Professorship thesis, Univ., São Paulo.
183. Ménard, L., 1963. "Calcul de la force portante des fondations sur la base des résultats préssiométriques". Sols, Soils, v. II, no. 5, p. 9.
184. Menzenbach, E., 1961. "The determination of the permissible point-load of piles by means of static penetration tests". 5th Int.Conf., Paris, v. II, p. 99.
185. Meyerhof, G.G., 1951. "The ultimate bearing capacity of foundations". Geotechnique, v. II, p. 301.
186. Meyerhof, G.G., 1953. "The bearing capacity of foundations under eccentric and inclined loads". 3th Int. Conf., Zurich, v. I, p. 440.
187. Meyerhof, G.G. and Murdock, L.J., 1953. "An investigation of the bearing capacity of some bored and driven piles in London clay". Geotechnique, v. 3, p. 267.
188. Meyerhof, G.G., 1955. "Influence of roughness of base and ground-water conditions on the ultimate bearing capacity of foundations". Geotechnique, v. V, p. 227.
188A. Meyerhof, G.G., 1956. "Penetration tests and bearing capacity of cohesionless soils". Jour. ASCE, SM1, p. 866-1.
189. Meyerhof, G.G., 1963. "Some recent research on the bearing capacity of foundations". The Canadian Geotec. Journ., v. I, no. 1, p. 16.
190. Meyerhof, G.G., 1964. Discussion, p. 161 ASCE, v. 90, no. SM1.
191. Meyerhof, G.G., 1964. "Some problems in the design of rigid retaining walls." Proc. XV Can. S.Mech.Conf., p. 59.
192. Mikhojev, V., et al, 1961. "Foundation design in USSR". 5th Int.Conf., Paris, v. I, p. 753.
193. Milovic, D., 1963. "Quelques observations concernant la force portante d'une fondation dans un milieu hétérogène" Sols Soils, Paris, v. II, no. 4, p. 16
194. Milovic, D.M., 1965. "Comparison between the calculated and experimental values of the ultimate bearing capacity". 6th Int.Conf., Montreal, v. II, p. 142.
195. Milligan, V. and Sodermann, L.G., 1962. "Experience with Canadian varved clays" ASCE, v. 88, no. SM4, p. 31.
196. Mizuno, T., 1953. "On the bearing power of soil under a uniformly distributed circular load". 3rd. Int.Conf., Zurich, v. I, p. 446.
196A. Mizuno, T. and Tokumitzu, Y., 1960. "On the bearing capacity of a slope of a cohesionless soil". Soil and Found., Japan, v. I, no. 2, p. 30.
197. Mogami, T. and Kishida, H., 1961. "Some piling problems". 5th Int.Conf., Paris, v. II, p. 111.
198. Mohan, D., and Chandra, S., 1961. "Frictional resistance of bored piles in expansive clays". Geotech., v. II, p. 291.

199. Mohan, D. and Jain, S., 1961. "Bearing capacity of bored piles in expansive clays". 5th Int.Conf., Paris. v. II, p. 117.
200. Mohan, D., et al., 1965. "Countering excessive settlement of a warehouse foun ed on piles". 6th Int.Conf., Montreal, v. II, p. 304.
201. Mohan, D. et al., 1967. "A new approach to load tests". Geotech., v. 17, no. 3, p. 274.
202. Mohan, D., et al., "Bearing capacity of multiple under-reamed bored piles. 3rd. Asian Reg. Soil Conf., Haifa, 1967 I, p. 103.
203. Moretto, O. and Bolognesi, A.L., 1959. "Pile foundation stressed by negative friction". 1st Pan.Am.Conf., Mexico, v. I, p. 315.
204. Moretto, O., 1967. "Minimum requirements for a subsoil investigation for foundation purposes". Symp. on site investig. for found., India, p. 21.
205. Mori, H. and Sone, S., 1964. "In-situ bearing test for deep strata". Soil Found., Japan, no. 5, p. 1.
206. Muramaya, S. and Shibata, T., 1960. "The bearing capacity of a pile driven into soil and its new measuring method". Soil Found., Japan, v.I, no. 2, p. 2.
207. Nair, K., 1967. "Load settlement and load transfer characteristics of a friction pile subject to a vertical load". 3rd Pan.Amer.Conf., Caracas. v. I, p. 565.
208. Naylor, A.H. and Doran, T.G., 1948. "Precise determination of primary consolidation". 2nd Int.Conf., Rotterdam, v. I, p. 34.
209. Nishida, Y., 1960. "An analysis of the bearing capacity of group piles in soft clay". Symp. Pile Found., Stockholm, p. 140.
210. Nishida, Y., 1961. "Determination of stresses around a compaction pile". 5th Int.Conf., Paris, v. II, p. 123.
211. Nishida, Y., 1963. "Pore pressure in clay induced by pile friction". 2nd Pan Americ. Conf., Brasil, v. II, p. 123.
212. Nishida, Y., 1963. Discussion, p. 131. ASCE, v. 89, no. SM5.
213. Nishida, Y., 1964. "The elastic settlement of a pile in the ground". Soil Found., Japan, v. V, no.1, p.31.
214. Nixon, I.K,m 1949. Corresp.Geotech., p. 208, p. 274.
215. Noorany, D. and Seed, B., 1965. "In--situ" strength chacaracteristics of soft clays.". ASCE, v. 91, SM2, p.49.
216. Nordlund, R.L., 1959. "Some experiences with the driving and load testing of heavy, large diameter piles in stiff clays". 1st Pan.Am.Conf., Mexico, v. I, p. 349.
217. Northey, R.D. and Thomas, R.P., 1965. "Consolidation Test Pore Pressures". 6th Int.Conf., Montreal, v. I, p. 322.
218. Nuñez, E., et al, 1967. "Algunas relaciones entre los metodos de calculo de la carga permisible y el comportamiento real de pilotes de hormigon". 3rd Pan.Amer.Conf., Caracas, v. I, p. 673.
219. Odenstad, S., 1948. "Loading tests on clay". 2nd Int.Conf., Rotterdam, v. I, p. 299.
220. Orrje, O. and Broms, B., 1967. "Effects of pile driving on soil properties". ASCE, v. 93, no. SM5, p. 59.
221. Osterberg, J.O., 1947. Discussion, p. 128, ASTM, Symp.load tests, Sp.pub. 79.
222. Parry, R.H.G., 1968. Discussion, Geotechnique, no. 18, p. 510.
223. Peck, R.B., 1960. Discussion p. 79. ASCE, v. 86, no. SM1.
224. Peck, R.B. and Bryant, F.G., 1953. "The bearing capacity failure of the Transcona elevator". Geotech. v. 3, p. 201.
225. Peck, R. and Reed, W., 1954. "Engineering properties of Chicago subsoils". Univ. Illinois, Bull. no. 423.
226. Peck, R. and Uyanik, M., 1955. "Observed and computed non-uniform settlements of structures". 4th Int.Conf., London, v. I, p. 402.
227. Peck, R.B., 1965. "Pile and pier foundations". ASCE, v. 91, no. SM2, p. 33.
228. Peck, R., 1965. "Bearing capacity and settlement: certainties and uncertainties". Symp. Bearing capacity and settlement of Found.", Duke Univ., USA, p. 3.
229. Pellegrino, M.A., 1961. Discussion Section 3B. 5th Int.Conf., Paris, v. III, p. 273.
230. Pichumani, R., and D'Appolonia, E., 1967. "Theoretical distribution of loads among the piles in a group". 3rd Pan. Am. Conf., Caracas, v. I, p. 546.
231. Pile, K.C., 1967. "The settlement of two buildings founded on partially saturated clays". 5th Austr.-New-Zealand Conf., p. 256.
232. Plantema, G., 1948. "Results of a special loading test on a reinforced concrete pile". 2nd Int.Conf., Rotterdam, v. IV, p. 112.
233. Peynircioglu, H., 1961. "Observed and calculated settlement of structures". 5th Int.Conf., Paris, v. I, p. 759.
234. Peynircioglu, H., 1961. "Settlements of buildings in Istambul", 5th Int.Conf.

Paris, V. I, p. 771.

235. Peynircioglu, H., 1965. "Adjustable box foundation as a measure against excessive settlement and tilting". 6th Int. Conf., Montreal, V. II, p. 174.

236. Polshin, D.E., & Tokar, A., 1957. "Maximum allowable non-uniform settlement of structures". 4th Int. Conf., London, V, I, p. 402.

237. Potyondy, J.G., 1961. "Skin friction between various soils and construction materials". Geotechnique, V.11, p.339.

238. Poulos, H.G., 1968. "Analysis of the settlement of pile groups". Geotechnique, V. 18, no. 4, p. 449.

239. Poulos, H.G. & Davis, E.H., 1968. "The settlement behaviour of single axially loaded incompressible piles and piers". Geotechnique, V.18, no.3, p. 351.

240. Ray, P., 1961. "Laboratory study on the effect of piling on the shear characteristics of soils". Symp. on Found. Eng., Bangalore, p. 1.

241. Raymond, G.P., 1966. "Consolidation of slightly over-consolidated soils". A.S.C.E., V.92, no. SM5.

242. Raymond, G.P., 1967. "The bearing capacity of large footings and embankments on clays". Geotechnique, V. 17, no. 1, p. 1.

243. Reese, L.C. et al, 1955. "Pressure distribution along friction piles". Proc. A.S.T.M., V. 56, p. 1156.

244. Resendiz, D., 1964. "On a type of point-bearing pile through sinking subsoil" Congreso de Cimientos Profundos, Mexico, V. I, p. 385.

245. Resendiz, D., 1964. "Estudio de campo sobre pilotes de concreto reforzado". Revista Ingenieria, Mexico; XXXIV no. 1. (Publ. no.88, Univ.Nac. de Mexico)

246. Resendiz, D., 1967. "The elastic properties of saturated clay from field and laboratory measurements". 3rd Pan Amer. Conf.Soil. Mech., Caracas, V.I, p. 443.

247. Rethati, L., 1961. "Behaviour of buildings founded in embankments". 5th Int. Conf., Paris, V. I, p. 78.

248. Roberts, J.E., 1961. "Small scale footing studies: a review of literature". M.I.T. USA.

249. Robinsky, E.I., 1957. "Reaction of clays to the driving friction of piles". XI Canad. Soil Mech. Conf., p. 22.

250. Robinsky, E. & Morrison, C., 1964. "Sand displacement and compaction around model friction piles". The Canadian Geot. Journ., V.I, no.2, p. 81.

251. Rodriguez, R. & Rosenblueth, E., 1659. "Pruebas de extraccion con pilotes a escala reducida en arcilas tipicas de Mexico". 1st. Pan Amer. Conf., Mexico, V. I, p. 389.

252. Rosenhaupt, S., 1965. "Discussion p.131 A.S.C.E., V. 91, no. SM6.

253. Rowe, P.W., 1959. "Measurement of the coefficient of consolidation of lacustrine clay". Geotechnique, V.9, p.107.

254. Rowe, P. & Barden, L., 1966. "A new consolidation cell". Geotechnique, V.16, no. 2, p. 162.

255. Safferi, M. & Tate, A., 1961. "Model tests on pile groups in a clay soil with particular reference to the behaviour of the group when it is loaded eccentrically". 5th Int. Conf., Paris, V. II, p. 129.

256. Salas, J.A.J., & Belzunce, J.A., 1965. "Resolution théorique de la distribution des forces dans des pieux". 6th Int.Conf., Montreal, V.II, p.309.

257. Salasar & Reuines, 1959. "Control de las cargas en la cimentacion y de los asentamientos de edificios mediante mecanismos en los pilotes". 1st. Pan Americ. Conf.Soil, Mexico, V.I, p.405.

258. Sanglerat, G., 1967. "El penetrometro y el reconocimiento de los suelos". Servicio de Public. Minist. Obras Public., Madrid. French Edition, 1965, Dunod.

259. Schiffman, R. & Gibson, R., 1964. "Consolidation of nonhomogeneous clay-layers". A.S.C.E., V.90, no. [illegible], p.1.

260. Schultze, E. et al., 1961. "Standard penetration test and compressibility of soils", 5th Int. Conf., Paris, V.I, p. 527.

261. Schultze, E., 1963. "Beispiele fur setzungsbeobachtungen in bindigen und nichfbindigen boden". Proc. European Conf.Soil Mech.Found.Eng., Wiesbaden, V. I, p. 143.

262. Schiffman, R., 1958. "Consolidation of soil under time-dependent loading and varying permeability". Proc. Highway Research Board, V. 37, p. 584.

263. Schmertmann, J., 1955. "The undisturbed consolidation behaviour of clay". A. S.C.E. Trans., V. 120, p. 1201.

264. Scott, R., 1961. "New method of consolidation-coefficient evaluation". ASCE, V. 87, no. SM1, p. 29.

265. Seed, B. & Reese, L., 1957. "The action of soft clay along friction piles". A.S.C.E.Trans. V.122, p. 731.

266. Seed, B., 1965. "Settlement analysis, a review of theory". A.S.C.E., V. 91, no. SM2, p. 39.

267. Seed, H.B. & Reese, H.B., 1964. " Predicting the performance of friction piles in saturated clays". Congreso de Cimientos Profundos, Mexico, V.I, p.127

268. Serota, S. & Jennings, R., 1959. "The elastic heave of the bottom of excavations". Geotechnique, V.9, p. 62.

269. Sharman, F. A., 1961. "The anticipated and observed penetration resistance of some friction piles entirely in clay". 5th Int.Conf.Soil, Paris, V.II, p.135.

270. Shiraishi, S., 1964. "Stability of deep foundation". Japan, Soil and Foundation V. IV, no. 2, p. 55.

271. Silveira, E. S., 1967. "Thoughts concerning the applicability of the theory of elasticity to soils". 3rd Pan Americ. Conf. Soil, Caracas, V.I, p.3.

272. Simons, N.E., 1957. "Settlement studies on two structures in Norway". 4th Int.Conf.London, V. I, p.431.

273. Simons, N.E., 1965. "Consolidation investigation on undisturbed Fornebu clay". Norwegian Geot.Inst., publ. 62.
274. Simons, N., 1963. "Settlement studies on a nine storey apartment building at Økernbraten, Oslo". Proc. European Conf. Soil Mech. Found. Eng., Wisbaden V. I, p. 179.
275. Skempton, A.W., 1942. "An investigation of the bearing capacity of a soft clay soil". Journ. Inst. Civil Eng., p. 307.
276. Skempton, A.W., 1948. "Vane tests in the alluvial plain of the River Forth near Grangemouth". Geotechnique, V.I, p. 111.
277. Skempton, A.W. & Bishop, A.W., 1950. "The measurement of the shear strength of soils". Geotechnique, no.2, p.90.
278. Skempton, A.W., 1951. "The bearing capacity of clays". Build.Research Congress, London. The Inst.of Civil Eng., Div. I, p. 180.
279. Skempton, A.W. & Henkel, D.J., 1953. "The post-glacial clays of the Thames estuary at Tilbury and Shellhaven". 3rd Int.Conf., Zurich, V.I, p. 302.
280. Skempton, A.W., 1953. "Discussion Session 5".3rd Int.Conf.Zurich, V.3, p.173
281. Skempton, A.W. & McDonald, 1956. "The allowable settlements of buildings The Inst.of Civil Eng., London, p.721.
282. Skempton, A.W. & Henkel, D.J., 1957. "Tests on London clay from deep borings at Paddington, Victoria and the South Bank". 4th Int.Conf, London, V.I, p.100
283. Skempton, A.W. & Bjerrum, L., 1957. "A contribution to the settlement analysis of foundations in clay". Geotechnique, no. 7, p. 168.
284. Skempton, A.W., 1959. "Cast-in-situ bored piles in London clay". Geotechnique, V. 9, p. 158.
285. Skempton, A.W. & Sowa, V.A., 1963. "The behaviour of saturated clays during sampling and testing". Geotechnique, London, V.XIII, no.4, p. 269.
286. Skempton, A. W., 1966. "Summing up". Symp.on Large Bored Piles, London, p.155
287. Soderberg, L., 1962. "Consolidation theory applied to foundation pile time effects". Geotechnique, V.12, p.217.
288. Soderman, L.G. & Milligan, V., 1961. "Capacity of friction piles in varved clay increased by electro-osmosis". 5th Int.Conf., Paris, V.II, p. 143.
289. Sommer, H., 1965. "A method for the calculation of settlement, contact pressures, and bending moments in a foundation including the influence of the flexural rigidity of the superstructure". 6th Int.Conf., Montreal, V. II, p. 197.
290. Sowers, G.F., 1954. "Modern procedures for underground investigations". 5th Int.Conf., Paris, V.II, p.143.
291. Sowers, F. et al. 1961. "The bearing capacity of friction pile groups in homogeneous clay form-model studies". 5th Int.Conf., Paris, V.II, p.155.
292. Sowers, M.G.F., 1961. "Discussion Section 3B". 5th Int. Conf., Paris, V. III, p. 261.
293. Spencer, L., 1964. "The movement of soil beneath model foundations". Civil Eng'g. and P.W.R., London, no.695-696, p. 728-878.
294. Stefanoff, G. et al., 1965. "Anticipated and observed settlement of a high chimney". 6th Int.Conf., Montreal, V. II, p. 202.
295. Stoll, M.U.W., 1961. "Discussion Section 3B". 5th Int.Conf., Paris, V.III, p. 279.
296. Strachan, C. M., 1960. "Preconsolidation of foundation soils". 3rd Australian New-Zealand Conf.on Soil, p.135.
297. Stuart, J.C. & Hanna, T.H. et al., 1960. "Notes on the behaviour of mode pile groups in sand." Symp. on Pile Found., Stockholm, p. 97.
298. Sved, G. & Kwok, H.L., 1963. " The effect of non-linear foundation settlement on the distribution of bending moments in a building frame". 4th Australia-New Zealand Conf. on Soil, p.10
299. Széchy, M. C., 1961. "Discussion Section 3B". 5th Int.Conf., Paris, V.3, p.263
300. Szechy, C., 1965. "The influence of the shape of contact surface upon the bearing capacity and settlement of strip foundations". 5th Symp. Civl Eng. Deptº, Bangalore, V.3, p.C-2.
301. Tamez, H., 1964. "Pilotes eletrometalicos en las arcillas del valle de Mexico". Congreso sobre Cimientos Profundos, Mexico, V. I, p. 277.
302. Tassios, T., 1963. "Estimation pratique des tassements différés d'un pieu pendant un essai de chargement". Proc. European Conf.Soil, Wiesbaden, V.I, p.95
303. Tate, A.P.K., 1963. "Model behaviour of pile groups in clay". 2nd Pan Americ. Conf. Soil, Brasil, V.II, p.235.
304. Taylor, D. W.m 1948. "Fundamentals of soil mechanics". John Wiley & Sons.
305. Taylor, D. W., 1950. "A triaxial shear investigation on a partially saturated soils". A.S.T.M. Spec.Tech. Publ. 106, p. 180.
306. Taylor, D.K., 1967. "Notes on observation of building settlements". 5th Australia-New Zealand Conf. on Soil, p246
307. Teixeira, A. H. & Geotécnica S.A., 1959 "Typical sub-soil conditions and settlement problems in Santos, Brasil". 1st. Pan Americ.Conf.Soil, Mexico, V. I, p. 149.
308. Teixeira, A.H., 1966. "Correlação entre a capacidade de carga das argilas e a resistência à penetração". III Brasilian Cong.Soil, Belo Horizonte, V. I, p. III-55.
309. Terzaghi, K. & Peck, R., "Soil Mechanics in engineering practice". 1th Edition 1948 2th Edition 1967 John Wiley & Sons.
310. Terzaghi, K., 1943. "Theoretical soil mechanics". John Wiley & Sons.
311. Thomas, D., 1965. "Static penetration tests in London clay". Geotechnique, V. 15, no. 2, p.174.

312. Thurairajah, A., 1967. "One-dimensional consolidation characteristics of Thunukkai clay". 3rd Asian Conf.Soil Haifa, V. I, p. 318.
313. Thurman, A.G. & D'Appolonia, E., 1964. "Prediction of pile action by a computer method". Congreso sobre Cimientos Profundos, Mexico, V.I., p. 141.
314. Thurman, A.G. & D'Appolonia, E., 1965. "Computed movement of friction and, end-bearing piles embedded in uniform and stratified soils". 6th Int. Conf. Montreal, V. II, p. 323.
315. Tomlinson, M.J., 1957. "The adhesion of piles driving in clay soils". 4th Int.Conf., London, V.II, p. 66.
316. Trow, W.A., 1952. "Deepsounding methods for evaluating the bearing capacity of foundations in soil". Proc. 5th Canad.Soil Mech.Conf., p. 23.
317. Tschebotarioff, D., 1952. "Soil mechanics, foundations and earth structures". McGraw-Hill, p. 228.
318. Van der Veen, C., 1953. "The bearing capacity of a pile". 3rd Int. Conf. Zurich, V. II, p. 84.
319. Van der Veen, C. & Boersma, L., 1957. "The bearing capacity of a pile predetermined by a cone penetration test" 4th Int.Conf.,London, V.II, p. 72.
320. Van,Weele, A.F., 1957. "A method of separating the bearing capacity of a test pile into skin-friction and point -resistance". 4th Int. Conf., London, V. II, p. 76.
321.. Vey, E., 1957. "Frictional resistance of steel H-piling in clay". A.S. C.E., V. 83, no. SM1, p. 11.
322. Vesic, A., 1965. "Ultimate loads and settlements of deep foundations in sand". Symp.on Bearing Capacity and Settlements of Found,Duke Univ., p.53.
323. Wallace, G. & Otto, W., 1964. "Differential settlement at Selfridge Air Force Base A". A.S.C.E., V. 90, no. SM5, p. 197.
324. Ward, W.H. et al., 1965. "Properties of the London clay at the Ashford common shaft". Geotechnique, V.XV, no.4, p. 321.
325. Ward, W.H. et al., 1959. "Further studies of the properties of London clay" Geotechnique, V. 9, p. 33.
326. Whitaker, Th., 1957. "Experiments with model piles in groups". Geotechnique, V. 7, p. 147.
327. Whitaker, Th., 1960. "Some experiments on model piled foundations in clay". Symp. on Pile Foundations, p. 124.
328. Whitaker, T. & Cooke, R., 1961. " A new approach to pile testing".5th Int. Conf., Paris, V. II, p. 171.
329. Whitaker, J. & Cooke, R., 1966. " An investigation on the shaft and base resistance of large bored piles in London clay". Symp. on Large Bored Piles, London, p. 1.
330. Whitman, R. & Hoeg, K., 1965. "Two dimensional analysis of stress and strain in soils". M.I T. Soils Dept. Publ. no. 187.
331. Williams, G.M.J. & Colman,R.B., 1965. "The design of piles and cylinder foundations in stiff, fissured clay". 6th Int.Conf., Montreal, V. II, p. 347.
332. Woodward, R. & Boitano, J., 1961. "Pile loading tests in stiff clays" 5th Int.Conf.,Paris, V.II, p.177.
333. Yamagata, K., 1963. "The yield bearing capacity of bearing piles". Proc., Conf. Soil, Budapest, p. 325.
334. Yamaguchi, H. & Kimura, T., 1967. "On the rigorous solutions in one-dimensional consolidation of non-homogeneous clay layers", Japan, Soil and Foundations, V. VII, no. 1, p. 41.
335. Yong, R.N.Y., 1959. "Un estudio de las caracteristicas de asiento de zapatas modelos sobre limo". 1st. Pan Americ. Conf.on Soil, Mexico, V. I, p. 491.
336. Yong, R., 1963. "Model bearing tests on a remoulded clay". 2nd Pan Americ. Conf. on Soil,Brasil, V. I, p. 336.
337. Yu, T. M. et al., 1965. "Settlement analysis of pile foundations in Shanghai". 6th Int.Conf.,Montreal, V. II, p. 356.
338. Zaharescu, E., 1961. "Sur la stabilité des fondations rigides". 5th Int.Conf. Paris, V. I, p. 867.
339. Zeevaert, L., 1957. "Compensated friction-pile foundation to reduce the settlement of buildings on the highly compressible volcanic clay of Mexico City" 4th Int.Conf., London, V.II, p.81.
340. Zeevaert, L., 1958. "Consolidacion de la arcilla de la ciudad de Mexico", A.S.T.M. Spec.Tech.Publ. 232.
341. Zeevaert, L., 1959. "Reduction of point bearing capacity of pile because of negative friction". 1st. Pan Americ. Conf. Soil, Mexico, V. I, p. 331.
342. Zeevaert, L., 1963. "Dynamic design and behaviour of friction piles compensated foundations". Revista Ingenieria, Mexico, V. XXXII, no. 3.
343. Zeevaert, L., 1964. "General considerations on problems related with pile and pier foundations". Congreso sobre Cimientos Profundos, Mexico,V.I., p.65
344. Zeevaert, L., 1967. "Consolidation theory for materials showing intergranular viscosity" 3rd Pan Americ.Conf. Soil, Caracas, V. I. p. 89.