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GENERAL REPORT

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FOUDATIONS OF BUILDINGS IN CLAY

FONDATIONS DE STRUCTURES SUR ARGILE

V. F. B. de Mello, D. Sc. Prof.
University of Sao Paulo, Brazil.

In attempting to interpret and depict the State-of-the Art of Foundations of Buildings in Clay during the first three months of the year, it was well recognized that the only hope of attenuating the tendency for such efforts to become obsolete even as they were being intensely pursued, would be to restrict the scope of the work to a certain line of problems that the design engineer faces most frequently, and, within such an avenue, to discuss the solutions that have penetrated into a position of accepted practices, or have a good probability of so doing in the immediate future.

The contributions that were received for this Session, and other important papers that have been published in the interim so well bear out the above foretoken, that to begin with it is my self-assumed, almost immodest, duty to voice on behalf of the Society a vote of appreciation on the rate at which contributions in the field are being made, both in diversity and in depth. Thirty-six excellent contributions were channeled to this Session, within a scope significantly broader than the title would imply, well attesting to the diversity of problems challenging the interest of the profession.

On the other hand, it is partly true that due to the novel approach set up for this Conference by the Organizing Committee, some of the diversity of contributions received within this Session derives from the somewhat more restrictive nature of the other four Sessions. Foundations of Buildings in Clay has been widened to read Foundations, Buildings, Clay: and, in turn, Foundations may be read "behaviour of subsoil under engineering works", Buildings has been broadened to include the generalized concept of "superstructure" (including hydraulic works, storeyards, and embankments), and Clay has been extended, as was early forseen, to cover "engineering behaviour of general (c, ϕ) soils".

All of which has been and is received with the greatest enthusiasm, not only because one of the principal aims of such a Conference is to catalyze exchange of creative thinking, but also because any eventual restrictive compartimentalization would portend the onset of crystallization and consequent stagnation.

Nevertheless, the General Reporter herewith begs leave to retain, within the tantalizing richness of such contributions, his more modest scope and aim, in an attempt to retain as a keynote to the discussion the improvement of the levels of confidence of design decisions on problems besetting the foundation designer. Thus the reader will have to study the individual contributions to cull their deeper and/or wider intended lessons: what will be extracted from them for the purposes of this General Report surely will not do them full justice.

1. BEARING CAPACITY PROBLEMS.

The initial basic problem facing a foundation design is that of safety agains failure. Those who correctly claim that the foundation design problem is always one of settlements may pardon the reminder that one tends to forget the steps by which one did ascend to such a position of experience wherein the initial design feel is sufficient to ward off any perspective of failure, whereupon the problem is definitely one of settlements.

The papers submitted within this area of interest have been interpreted to contribute the following thoughts towards the chosen line of discussion.

Bent Hansen develops by means of the theory of plasticity, for an ideal rigid-plastic material, the mathematically correct solution for the undrained failure (τ_f = c) bearing capacity of a strip load founded at a shallow depth D, (D ≤ 2B). The contribution represents a refinement on part of the strip-loading Nc curve presented in Fig. 5, p. 56 (Apud Skempton 1951); it indicates that Skempton's recommended Nc values lie slightly on the unsafe side, by about 4 % for D = B, and about 6 % for D = = 2B, which is a correction of a far narrower precision than could obtain in the principal factors intervening in the problem at stake. It is to be hoped that in order for such a "more correct" solution to take hold, the endeavours be pursued until the entire set of recommendations embodied in the Skempton (1951) solution be covered.

Brown and Meyerhof present solutions, based on model tests, for circular and rectangular footings supported either on a stiff clay overlying a soft clay, or on a soft clay overlying a stiff clay. It is of interest to note that as a start the research centered on the classic case of model footings on homogeneous clay, being directed at some of the questions raised under items 2.3 and 2.4 of the State-of-the-Art Report: thus, the ϕ value applicable to the establishment of the failure surface is discussed, and a correction factor is investigated for use with the c value as determined by unconfined compression tests, in order to have the bearing value tally with theoretically established values of Nc which are taken as 5.14 for the strip loading and 6.05 (apud Eason and Shield, 1960) for the circular footing. Incidentaly, is a 3" x 6" model footing long enough to be compared with the Prandtl strip solution ? If Skempton's (1951) rectangle solution were applied, it appears that both for circular and for rectangular footings the correction factor to be applied would be 1.21. Is such a correction, for comparison with the theoretical rigid-plastic case, really applicable in the face of the stress-strain behaviour (cf. Ladanyi 1967) ? And, upon application of such a 20 % correction to UC values, what happens to the acceptably-established correlations for Nc as summarized in Table II of the State-of-the Art Report ? It would be of considerable interest to have such meticulous laboratory investigations interpreted statistically. Is there any perceptible influence from the slight ϕ value (compatible with the Sr = 95 %) ? Considering the importance of stress-strain phenomena, and the significant differences therein indicated as obtained in the two layered-clay cases, it would be of interest to establish within what precisions the "average" percent penetrations as described are established, and correspondingly to what extent the results presented are restrictive to the single clay tested or to the clays described as moderately brittle, slightly sensitive. How far would a change in rigidity, concomitant with the change of strength, affect the results ? The paper tackles a problem of very frequent interest to the foundation designer, and concomitantly it touches on problems of reappraisal pointedly raised in the State-of-the-Art-Report

It would appear to the design engineer that the vicissitudes of the bare application of mathematical derivations without the support of experimental observation are brought to attention in the paper by Mandel and Salençon. Jürgenson (1934) and, as pointed out by the authors, Livnel (1965) and Vyalov (1966), would lead to the conclusion that the bearing capacity of a soft clay would decrease significantly if underlain by rock at shallow depths. The authors reconsider the problem of the strip footing on the (c, ϕ) soil, applying the theory of limit equilibrium and the method of characteristics to establish the bearing capacity factors, which are found to increase very appreciably with the presence of the shallow bedrock. The case is of frequent practical interest as a limiting condition to one of the cases experimentally investigated by Brown and Meyerhof. Some model testing might be indicated to clear such untenable differences of opinion, incumbent upon distinct hypotheses governing the derivations.

Muhs and Weiss cast further light on the problem of eccentric and inclined loadings on rectangular shallow footings on sand, resorting to large-scale model tests for the more practical case of eccentricity and inclination along the longer dimension of the footing (compared with the several theoretical analyses and with Meyerhof's 1953 small model tests with eccentricity and inclination parallel to the short side because of the simulation of strip loading conditions). The results presented form part of a wider research program, and thus as yet cover only a single inclination(20°), single eccentricity (L/6), and single ϕ (40 - 42°). Interest is focussed also on the measurement of normal and tangential stresses under the footing along its long axis. It may appear somewhat early to postulate the extremely simple equation ..... 1 - tan δ , as sufficient to reflect the decrease due to inclination.

Incidentally, in view of the fact that recent trends of thinking have repeatedly pointed towards the need of an elasto--plastic model to substitute for the rigid--plastic Prandtl analysis, it is of interest to register herein the publication by Rhines (1969) suggesting the application of a modified Pasternak foundation model for precisely the above purpose.

2. INDIVIDUAL PILE AND PIER LOAD-SETTLEMENT BEHAVIOUR.

Considerable interest has centered on instrumented pile tests for interpretation of the load-settlement behaviour and ultimate load capacity.

Reese et al. investigate a large-diameter (30 ins. by 28 ft.) plain-bored pile executed in a stiff clay, above the water table. The load-test data presented are of considerable interest because the shaft performance has been measured at five or six intermediate levels besides the base and top levels at which Whitaker and Cooke's (1966) data were obtained. It is clear that much additional interpretation may be extracted from the data if printing space limitations were waived. To begin with, it would be of interest to derive from the data (which closely resemble earlier data on piles, mentioned under item 2.5.2 of the State-of the-Art Report) the detailed interpretations on β_2 values instead of working with overall β values which do not permit confident extrapolations to other lengths of piles (reference to Mattes and Poulos 1969 may prove useful)

Moreover , since four consecutive load tests to roughly the ultimate load were carried out, the stress-strain behaviour was investigated up to about 3.5 ins. of settlement, thereby permitting some insight into the residual skin resistances available after development of the respective peak. It may be noted that Burland et al (1966) had carried their load tests on London clays to settlements of 6 to 7 ins., but did not possess data for separating base and skin resistance contributions: it can only be concluded, by combining their curves with information adapted from Whitaker and Cooke's curves, that the skin friction follows a typical strain-softening tendency compensated by the increase of base resistance at greater settlements. Whitaker and Cooke's data, as mentioned earlier, had not proceeded beyond settlements of about 0.3 to 0.5 ins., at which point the shaft adhesion had been fully developed but only about 30 to 40 % of the base resistance had come into play. Indeed, from the data of Burland et al. it transpires that beyond a settlement of about 0.25 ins. the load--test data come close to "failure", and, moreover, it is clear that the total settlements of the structure will be significantly greater than the "immediate settlement" observed in the load test (see, for instance, Fleming and Steger, 1966, with respect to observed settlements of buildings on large-diameter piles in London). But, for the purpose of possible applications of the conclusions to other areas, where, for instance, different materials may be at play under the base and along the shaft, it is highly desirable to interpret the full stress-strain behaviour of the separate components - a point in which the present data complement the earlier publications.

Unfortunately the subsoil data presented are not particularly conducive to more generalized interpretation, as would be eagerly desired. In the face of difficulties in undisturbed sampling etc., the shear strain-strength information is limited to mention of erratic UC values between 2 and 8 kg/cm^2 accompanying the fairly consistent 15 to 45 blow N-values of the Texas Highway Department dynamic cone penetrometer. The underlying horizon only registers N values, varying in a highly erratic manner between 110 and 790. It is hoped that the authors may yet furnish collateral indications for the correlations that would be of great use. It would also be of interest to investigate(cf. Skempton 1959, item 2.7.2 State-of-the-Art Report) the influence of concreting water migration on the adhesion (affecting β_2 values):also, the time elapsed between the consecutive load tests (effect of time on re-healing of adhesion strained beyond its peak in the previous loading).

Finally, it is noted that implicit in the Authors' fitted formula lies the hypothesis that the strains necessary to develop adhesion would be proportional to the diameter, a hypothesis that cannot be proved or disproved by testing a single pile, but that may lead to misapplications if extended to other diameters: it appears that the displacements necessary to develop skin resistance should be essentially independent of diameter, execution effects excluded.

Kerisel and Adam present a very rich accumulation of instrumented steel-pile tests on two soils, a stiff clay, and a silt with indurated layers: present discussion will be restricted to the clay because the indurations in the silt clearly make it a rather special case. The clay shear strength (UC and vane) is defined by the equation $c = 6 + z\ (\pm 2.5)\ t/m^2$, with values ranging from $6\ t/m^2$ to $13\ t/m^2$ over the depth of interest. SPT values would thus tally with about (6.6 to 7.6) qu. The result worthy of special notice lies in the cone penetrometer results with three diameters and two speeds, which furnished point resistances of the order of 25 to 35 kg/cm^2, indicating Ncp values of the order of 30 (cf. item 2.6.1).

The pile cross-sections employed were hollow, closed-end (presumably) box shapes of three different configurations composed from steel sheet piles riveted or welded: the 7 m long piles were pushed into place to varying depths. The instrumentation permitted recording the load distribution along the piles at every 0.5 m of depth. The salient results summarized by the authors include: the high point resistance bearing capacity factor varying from about 25-30 (with respect to the vane c value) to about 12-15, with increase in the "diameter" of the piles (an "equivalent diameter" being reflected, for the different shapes of the pile sections, by the ratio of base area to perimeter); the skin resistance reflecting a β value of about 1.

Once again it appears that the publication has suffered from space restrictions, since the data presented would warrant much additional interpretative study. On one point additional clarification appears indispensable: the load testing was carried out in some cases right after the pushing-in, and in other cases after certain resting periods which were established at 4 weeks as a maximum; considering the interference of pore pressures etc., it seems important to specify for each result what was the rest period, since no minimum was set.

Sherman reports on five instrumented steel nondisplacement piles (3H piles and 2 open-end tube piles) driven into a stiff Tertiary clay stratum, and then load tested in tension and compression after a minimum rest period of two weeks. Firstly it would be of interest to know if the split-spoon penetration resistances are meant to be SPT values, since the apparent correlation for the scattered results furnished would average $qu \simeq SPT/22$ with a range between SPT//12.5 and SPT/50 (cf. item 2.8). Moreover, for more generalized interpretation it would be of interest to request indications on the sensitivity and/or stress-strain curves of the strength tests, and if additional results on the undisturbed samples might not be available to narrow the wide dispersion reflected by $c \approx 1.6 \pm 0.8\ kg/cm^2$.

The author emphasizes as a conclusion the fact that the computed adhesion values (presumably β_T values) approximately equalled the full undrained shear strength of the clay, and far exceeded "values normally expected in stiff to hard clay soils", basing this observation on β values derived from displacement piles. It is felt that the results presented are, in general trends and orders of magnitude, compatible with existing information, it being assumed that the clay is of low sensitivity, and that whipping effects may be excluded from consideration. When the tip resistance is practically non-existent, automatically the full loading must be taken up by adhesion: and if the adhesion were insufficient, automatically the pile would be driven deeper, or pushed down in the load test; only if a high sensitivity (or whipping) were able to interfere, at the top of the pile, with the accumulation of overall skin resistance, would such a reasoning fail, the pile failing to increase in bearing capacity with depth. It must be recalled that immediate displacements (observed in such load tests) involved in developing adhesion are very small, and bear no relation to the long term settlement resulting if such "maximum adhesions" are at play.

It would be of much interest if the authors clarified for what effective tip areas the calculations were made, arriving at the Ncp = 9.3 (average): also what were the average adhesions β of the piles at the maximum loadings: and, finally, if any pull-out test was carried far enough to confirm, approximately, the effective perimeter for adhesion computations.

The degree of mobilization of adhesion along piles driven into stiff clays is also investigated by Stermac et al., by means of a series of load tests on six different piles. The results presented are of interest principally through reflecting comparative behaviour of different types of piles, and between loading and extraction tests: as regards adhesion, β values appear to follow reasonable trends, and show significant increases with time.

Marioti and Khalid investigate bored piles executed in an overconsolidated swelling clay: the relatively stubby piles were fitted with flat jacks at the base so as to permit separate testing of the base and skin resistance (approximating the testing of instrumented piles). It must be noted that the paper devotes principal attention to the behaviour of the piles when subject to tensile forces due to the swelling: however, this subject is considered as pertaining to a special case, beyond the scope chosen for this General Report. The three compression piles were load tested, individually and as a group. Results of direct interest to the themes under consideration include: a base resistance Ncp value of about 19, only developing after the full adhesion had developed; the β values of about 1 developed, for the types of execution employed, within a month after concreting (the very summary description of the piles does not furnish indications on perforation and concreting water migrations, etc.); the β value remaining essentially constant at progressively greater pile load-test settlements; the group ultimate resistance per pile (at center-to-center distance of 4 diameter) was essentially identical with that of single piles, but at deformations roughly doubled.

The load capacity of slender steel piles in soft clays is treated by Hoadley et al., who carried out load tests on five instrumented piles, essentially confirming existing indications (for example, Glick 1948, et al.). It is of special interest to note also the collateral information, concerning long term and group interaction, which constitute, to the General Reporter's. knowledge, novel pieces of information. On the long term loading "there was a significant increase in the axial forces carried by the lower portion of the pile, indicating a relaxation in the friction in the upper soil layers". No significant effect on adjacent piles (spaced at 4 times the width in both directions) could be detected as one of the piles of the group was loaded to failure.

Regarding special types of piles devised for drawing greater benefit of the soil-pile interaction towards improved bearing capacity, Mohan et al. present new data on multi-reamed piles: an optimun spacing of bulbs of the order 1.25 to 1.5 times the pile diameter is indicated based on pull-out tests on full scale piles, it being hoped that further data on such investigations may be furnished to permit full assessment of their applicability to other situations.

Special attention must be called herein to the recent extensions by Mattes and Poulos (1969) and Poulos and Mattes (1969) of the derivations based on elastic theory, for prediction of the load-settlement behaviour of single piles. The former analysis pertains to a compressible floating pile of circular cross-section in an ideal elastic soil, and furnishes a clear insight into the problems of load transfer along the pile and the load carried by the pile base (whether enlarged or not) for cases of some piles in stiff clays such as the London large-bored pile load tests, or the cases presently reported by Reese et al, Kerisel and Adam, and Sherman. The other analysis (Poulos and Mattes, 1969) is similar to that followed for the study of negative friction (Mexico 1969) but taking into account a finite compressibility of the bearing stratum. The theoretical solutions presented by the authors enable the displacement of the top and the tip of a pile, and the distribution of load in the pile, to be calculated, once appropriate values of Eb/Es (moduli of elasticity of bearing stratum and soil layer) and pile stiffness factor K are assumed or determined. The behaviour of an end-bearing pile is shown to be influenced primarily by the length to diameter ratio L/d, and the above factors.

The publications are reputed to be of very considerable importance. To begin with, they should permit more appropriate evaluation (preferably statistical) of the wealth of scattered load test data published during the past twenty years, such as, for instance, with regard to β_3 factors (cf. item 2.5.1.b) or β factors connected with given β_1 and β_2 conditions. Moreover, they supplant the need for the step-integration solutions (cf. item 2.5.2): it appears that even for successions of different strata, for which these solutions would appear indicated, the principle of superposition of the elastic theory solutions permits easy use of the Authors' results. Finally, the most obvious immediate application of the results lies in the very clear insight they give into comparative behaviours of different piles, furnishing the theoretical skeleton for model-to-prototype or prototype-to-prototype relationships.

3. SETTLEMENTS OF SHALLOW FOUNDATIONS.

In discussing the computation of settlements of shallow foundations it had been arbitrarily decided for the State-of-the-Art Report to retain, as a start, the subdivision into "Immediate Settlements" and "Consolidation Settlements".

In connection with immediate settlements mention will first be made of the results of two very carefully observed large-scale load tests (Höeg et al, 1969) on a slightly preconsolidated quick clay; the observed initial linear relationship of pressure vs. immediate settlements was found to confirm the value of young's modulus $E = 1000 \times$ (undrained field vane strength): (cf. Leonards 1968, et al.).

For use in computation of such elastic theory immediate settlements, Sovinc submits the analytic solutions for rigid square and rectangular footings resting on a limited elastic layer ($\mu = 0.5$) of uniform thickness bounded by a rigid lower bed. Although the derivation is for frictionless contacts at the base of the footing and at the rigid lower boundary, it furnishes some indications of comparative conditions. For instance, for a square footing on an elastic layer 2.5 times (or more) as thick as the side of the footing, the uniform rigid settlement will be given as $0.62\,q\,B/E$, in the notations of item 3.1. The paper extends into consideration of contact pressures and inclinations due to loading moment, which lie beyond the scope set for this General Report.

Chin resorts to the technique, hitherto used in various fashions, of direct interpretation of size relationships for load tests on a lateritic clay. Reinterpreting Bond's (1961) and other data he gathers confirmatory evidence for the conclusion, from dimensional analysis, that the load-size-settlement relationships are $Q/d^3 = f(\rho/d)$ for sands and $Q/d^2 = f(\rho/d)$ for clays: and for model tests on a remoulded lateritic clay the most appropriate fit was obtained by the equation $(\rho/d)/q = M + Q(\rho/d)$. As is well known, the problem in such an approach for estimate of footing settlements lies in the fact that a minimum of two load tests would have to be performed for each soil, and each of the load tests carries a heavy responsibility with respect to experimental error or non-homogeneity: a more generalized approach such as Bond's although initially subject to wider margins of errors can hope gradually to accumulate data from various sites, so as to permit narrowing statistical confidence bands around significant parameters.

In any attempt to interpret such size relationships as subject to errors either at the seating of the plate or due to layered subsoil, it is important to investigate the probable distribution of compression contributions of the supporting soil "slabs". Szechy points to the fact that for a given accumulated-compression total settlement, the distribution of the unit compressions is not a maximum at the upper "slab" but rather at about a depth equivalent to the width of the footing B. The significant thickness subject to compressions extends to about 2 B. Only a part of the explanation derives from the lateral support at the upper slabs due to friction at the bottom of the footing. The principal contribution may be theoretically deduced by appropriately considering the relative displacements of the assumed boundary planes between slabs.

Regarding consolidation settlements and important intervening parameters (items 3.2 and 3.5), mention must be made of some important contributions that have come forth since preparation of the State-of-the-Art Report. Leonards (1968) firstly calls attention to the appreciable error that occurs in hitherto conducted stress distribution computations for shallow foundations by the routine Boussinesq approximation instead of a detailed excavation analysis: incidentally, since the error for the cases presented corresponds to an overestimate of the applied pressures by as much as 50 to 100 %, it will be necessary to scurry once again to reconsider all the settlement computations that have hitherto been made to tally closely with observed values, and which doubtly concealed compensating errors (cf. 3.5.2). As regards the application of oedometer test results, a special technique of testing and of interpreting the laboratory results is indicated: besides the care in determining the p_c value and the recompression index, the principal point comprises plotting the ϵ vs. log p curve on the basis of 0 and 100 % consolidation compression (by logt method) for the consecutive increments, and thereby computing the consolidation settlements; and separately computing the secondary compression, based on the consistent relationship discovered (Leonards and Girault, 1961) between the ratio R_s/R_{100} of the secondary compression R_s per log cycle to the 100 % consolidation compression R_{100}, versus the load-increment ratio. Incidentally, has the consistent relationship mentioned ever been observed to interfere in field pore pressure observations, or might it be strongly associated with the laboratory test ?

Höeg et al (1969) conclude from the carefully instrumented load tests above mentioned, that the pore pressure change in the undrained element of soil (given by the generalized form of the Skempton A coefficient formula) was practically equal to the added octahedral total stress determined with $\mu = 0.5$, and that beyond the point where applied loads caused local shear stresses exceeding the undrained strength of the clay, the noticeable increase in developed pore pressures should not be associated with an "artificially increased" A-value, but may be computed for an appropriately postulated condition of contained plastic flow. What part would the above Leonards-Girault secondary compression effect, and/or comparative piezometer time lags and " immediate" u alterations (Gibson 1963) at different stress ranges play in interpretations of such a precise nature?

At any rate, it seems to the General Reporter that the above two very instructive publications call for adaptations and complementations in order to transform them into implements of design decisions in the face of consolidation computations.

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Another paper that calls for the closest scrutiny is that on "Secondary settlements of buildings in Drammen, Norway" by Foss, representing a follow-up on Bjerrum's(1967) clearly expounded concept of the "delayed consolidation" preconsolidation, detectable through very meticulous sampling, handling, and consolidation testing on clays exhibiting significant "secondary compressibility". Setting aside minor revisions that might be considered in connection with the computations, it must be conceded that the existing data might merit, from a design engineer's point of view, the interpretation that they basically comprise an overestimate of the laboratory preconsolidation pressure p'_c in comparison with field p_c values (cf. item 3.5.4). Since most clays intuitively accepted as normally-consolidated (e.g. the Santos clays) must have existed for centuries or thousands of years, the delayed-consolidation preconsolidation effect must be quite general, to varying and unknown degrees. Therefore, it appears that the theory proposed for Drammen should be quite generally applicable, to varying degrees depending principally on the secondary compressibility.

What has been the experience of settlement computations ? Well, partly through hindsighted wishful thinking and partly through overt η corrections (item 5) the overall settlements have generally been made to agree. Such rare and especially interesting cases as the one covered by Moore and Spencer (1969), wherein all existing methods of calculation candidly lead to seriously underestimated settlements, may provide food for reconsideration in the light of the new theory. At any rate, in general, as regards normally-consolidated clays it must be conceded that, to begin with, p'_c values were recognizedly always somewhat lowered by sampling and testing techniques, and interpretations on the subsoil profile were always adapted for use of the virgin compression slope as a basis of "final" settlement computations. Therefore it can be seen that for "traditional" evaluation of such cases, some significant magnitude of total settlement was always available for minor adjustments to the observed values (all the more so since in the clays exhibiting more pronounced secondary compressibility the time of completion of field primary settlements has hitherto been subject to undisputable latitude of choice). As regards the intervening time-settlement curves, well, nobody could ever pay much heed to them anyway (cf. item 3.4).

It appears therefore that the problem falls back on piezometer readings, since very meticulous field pore pressure measurements would be required to detect, within the quasi-preconsolidated range, the clinching proofs of the theory proposed(incidentally, piezometer readings are briefly mentioned in the paper but only regarding groundwater conditions). Apparently, for such "aged" normally consolidated clays the basic problem, in comparison with "traditional practices" lies in the proportional significance attached to primary consolidation settlements or to secondary compression, reflecting in time-settlement curves and in long-duration "final" settlements (both rather elusive items at present). What have piezometer readings shown, routinely, in clays accepted as normally consolidated (ex. Hoeg et al, 1969) ? Can it be that none of these recorded cases coincided with an "aged" clay ?

Ironically, therefore, in the light of the design engineer's problems, the progressively greater care at determining laboratory p'_c values has raised a problem unsuspected when, for instance, more rudimentary techniques on less exotic clays permitted the Casagrande graphical p'_c determination (presumably on 24-hr laboratory loadings) to correlate acceptably with the presumed field p_c values. It would be of interest to establish what p'_c values would be obtained by the arbitrary "routine" graphical interpretation of routine oedometer tests, and within what statistical dispersions such p'_c values would be interpreted for the clay layers under study (cf. Fig 33, item 3.5.4). Thus, as a practical recommendation a correction factor Ψ may be suggested for application to p'_c values ($p_c = \Psi\, p'_c$) in such cases, for computation of the "final" settlements. Moreover, if the secondary compression does assume the overriding importance indicated, two points should be clarified: firstly, if one accepts the (ϵ, p_o) point as known, in order to follow along the curve corresponding to the "age of the clay deposit" ("final stage of deposition" ?) is not the latter information superabundant; secondly, for what load increment ratios, shown to be extremely important, should the laboratory testing be conducted to provide the set of parallel curves that the designer should use for his settlement computation ? It seems that by the proposed theory one has to start by computing the "final" settlement along the curve for the age of the clay (incidentally, the clay itself doubles its age meanwhile), and then rely on rate solutions to check on what should be the settlements within the short period of current interest. Can such an approach hold promise of improved precisions of design decisions ? Credence would be higher if the bands of dispersion of the data presented were candidly wider.

Finally, the reader's attention is called to the generalized method of predicting settlements proposed by Tsytovich,(Tsytovich and Dalmatov), under the denomination of the "equivalent layer method". It encompasses both the immediate and the consolidation settlement components, and the case of unsaturated soil is covered. Tabulated values are furnished that permit direct computation of the equivalent soil layer beneath a footing, whereby the one-dimensional compression of the respective layer should exactly reproduce the complex three-dimensional problem. The Author might expatiate on the field and/or laboratory tests employed to define the parameters at play, and on the respective precisions. One parameter of importance to the total settlement computation is pointed out to be the "structural compressive strength" of the soil skeleton, to be determined by carefully applying very small pressure increments until the "break" in the ϵ vs p curve is detected. Another parameter required is the "initial head gradient" below which the interstitial water does not flow. The Author further develops the same solution to furnish the time-settlement relationships. Finally, Dalmatov extends the method by developing an influence chart, analogous to Newmark's (1942), for use of the equivalent layer method in computing the mutual interference of adjacent buildings.

In connection with rate of consolidation solutions three advanced contributions that have been put forth recently must be classed as lying beyond the scope of this General Report, since they do not impinge on principal parameters conditioning design decisions on foundations. Davis and Lee present formal solutions for layered soil deposits, in extension of existing numerical and approximate solutions.The analysis furnishes data of considerable interest as an aid in assessing comparative situations likely to occur in practice. It appears, however, that the case of layered soils would indicate a strong predominance of horizontal permeability, whereby the vertical consolidation case would be of restricted interest. Berry and Wilkinson (1969) extend existing radial consolidation theory to take account of a varying permeability and compressibility, non-linear structural viscosity, and smear at the drainage boundaries (sand drains). Once again, the results at present serve principally for appraisal of comparative situations, until laboratory tests are more especially developed and widely used for determination of the necessary rheological parameters. Finally, Zaretsky et al. tackle general problems of the theory of nonlinear consolidation, including the investigation of the coefficient of initial pore pressure (in lieu of the Skempton A and B coefficients) for unsaturated soil, and the formulation of the process of consolidation and creep of such three-phase soils.

4. CASE STUDIES ON SETTLEMENTS.

Nonveiller and Kleiner report on a group of four heavy rectangular silos for which the settlement computations adhered closely to the Skempton-Bjerrum method, and the average settlement under full load is found to agree closely with the computed value. The Authors observe with interest, however, the fact that the settlement under dead load was much overestimated.

Appendino and Jamiolkowsky report on settlement computations, and settlement and pore pressure measurements on a 200 m high chimney on an overconsolidated clayey and sandy silt. The entire computation is carried out through theory of elasticity formulae, using $\mu = 0.3$ and appropriate E values extracted from oedometer test results, and attributing to the circular rigid foundation a "rigidity index ≤ 0.5" according to Gorbunov-Posadov 1953.

Dvorak reports on a special case of a multi-storey building founded on a reinforced-concrete raft which faced a relatively heterogeneous subsoil condition but successfully averted significant differential settlements.

Mathian and Paubel describe the case of a powerhouse-dam and lock on the Rhone river, founded on preconsolidated clay, with very interesting observations on heaves and settlements.

Bourges et al. report on the settlements of three embankments on soft clay, furnishing a set of very elucidative data on well instrumented tests. Attention is called to the important interference of lateral movements, and to the efficacy of sand drains, used under part of the fills for comparison. Of special interest also are the data derived on the increase of E values with consolidation (computed from immediate settlements of successive stages of the earth-filling), the comparison between laboratory and field pore pressure parameters (Skempton-Bjerrum) determined, and the discussion on the influence of the factor of safety of the loading on the lateral deformations.

Dastidar et al report on a fill loading test on a marshy area, proving the successful use of 2.5-inch "sand wicks" composed of sand-filled cylindrical bags of jute textile inserted into standard boreholes.

5. SPECIAL PROBLEMS.

Aitchison and Woodburn present a very elucidative report on some problems of design of foundations in desiccated areas, wherein at any given applied load large heaves or settlements may result from suction decreases or increases. Apparatus and techniques for measuring the necessary parameters are discussed, as a means towards rational estimation of vertical deformations.

Beles et al. report on a case of successful application of prewetting to a collapsible-structure loess, to anticipate the settlements (up to 2 m, and very erratic) of hydraulic structures, such as tanks of a water treatment plant, that cannot practically ensure avoidance of leakage into the subsoil during operation.

Lenssink and Wenz discuss the problems of storage yard foundations on soft cohesive soils, wherein the lateral deformations, and their appreciable influence (aptly demonstrated) on any deep foundations, play a major part in selection of the appropriate solution, economically optimized.

Egorov and Simvulidi consider the soil-structure interaction problem due to compression settlements, as affecting the stresses and structural behaviour of footing slabs and pile caps.

6. CONSIDERATION OF DIFFERENTIAL SETTLEMENTS.

Four papers were received comprising problems of very special interest directly related to design decisions on differential settlements.

Heil studies the foundation-superstructure interaction of reinforced-concrete structures subject to settlements, taking into account the fact that both the loads and the flexural rigidity increase during construction due to the additional floors, and, on the other hand, the creep of concrete decreases the rigidity. Inasfar as earlier solutions (e.g. Sved and Kwok 1963, et al.) assumed the settlement to start with the frame fully "ready", the paper establishes a step forward.

As regards the hypothesis of the linear stress-strain effect of redistributed loads on the footing settlements, it appears that the implicit consequence is to restrict the application of the paper to "direct settlements" of footings supported on clays (same influence factors for all footings); the case of settlements due to thick deeply-underlying compressible clay layers such as envisaged by Chamecki (1956) might be approached by introducing for each elemental area its appropriate influence factor, and, as necessary, appropriate stress-strain and strain-time relations. Thus, the Author's pointing out that application is restricted to cases (clays) in "which the settlements occur at a slower rate than the creep deformations of the concrete", brings forth the question as to the phase of the time-settlement phenomenon over which the method is applicable, since not only are such direct settlements quite rapid (item 2.9) but also the rates of settlements will drop significantly with time. The above relative-rate problem arises because the use of the same modulus of volume deformability for compressions and expansions is valid as part of the computational principle of superposition of stresses and strains at a given instant, but not for the volumetric strains that will have been consumated as time elapses. The Author might explain how the points were selected for application of the unit loads within the elemental loaded areas. Moreover, considering the major interference of wall--paneling on the rigidity of the building frame (e.g. Benjamin and Williams 1958, Rosenhaupt 1962, Rosenhaupt and Mueller 1963, et al.) it would be highly interesting to compute what additional redistribution of loads would obtain from such a factor. Finally, it would be of special interest to know, for the example used, what order of magnitudes of distortion redistribution factors λ (cf. item 5) would be computed for the various $\Delta \rho_2$ boundaries presently admitted as applicable (e.g. Fig. 39).

In view of the already well spread information with respect to some degree of interferences of building rigidities on differential settlements, it is hoped that in all cases in which authors (e.g. Leonards 1968, et al.) discuss the comparisons between computed and measured settlements and distortions (cf. item 5, the λ_1 and λ factors, which might be "statistically" related under common conditions, Jappelli 1962) some mention will hitherto always be included with respect to the significant structural characteristics and the computational simplifications followed.

Komornik and Zeitlen describe damages to stubby four-storey structures, due to differential settlements caused suddenly(upon a heavy rainfall), cyclically (during wet and dry seasons), and rapidly, over periods of months and/or few years, in the cases of buildings founded on short underreamed piles in desiccated preconsolidated clays. Since the rigidities of the buildings had been especially strengthened, and the damages varied from minor to almost ruinous, it would be of considerable interest if the Authors were to discuss the data in the light of damage criteria and design decisions. New areas will occasionally show up in which the soil phenomena reported may appear as a surprise, but the behaviour described is already recognized in many an area. Meanwhile it is of utmost importance to gather systematically all possible data towards improving present estimates of rigidities, λ values, and $\Delta \rho_2$ limiting conditions.

The opening up of new approaches towards quantifying the elements of "judgment" in the face of problems of foundation design must be hailed with the enthusiasm of curiosity because of the promises they might hold in store. In such a spirit have the two papers, by Shuk and by Resendiz and Herrera, been received by the General Reporter, irrespective of questionable facets that may be raised.

Shuk attempts to apply the reasoning of probabilities and the recognition of the statistical nature of soil properties (incidentally are structural factors so much more deterministic ?), to interpret and develop "judgment" in connection with the design of size of independent footings, subject to "direct settlements", so as to minimize the total building cost as a foundation cost, with due consideration of maintenance costs due to detrimental total or differential settlements. Indeed, similar thinking can already be found (e.g. Getzler 1968), even with some consideration of the dispersions of the basic intervening "data" such as the Author presents, for the summary idealized example, in his Figs. 1 and 2. Unfortunaltely, however, the paper does not initiate the eager reader into the practical steps that were followed in the example proffered, and that might be respected mutatis mutandis. How are the distributions determined ? by which and how many tests ? what significance is retained by the method and its conclusions within common bands of dispersion of the data ? With respect to one concept, moreover, a serious doubt arises. The Author apparently claims that the total expected cost of the whole building as a function of foundation cost is minimized by obtaining the summation of individually minimized costs for each footing: venturing into intuitions on the problem it is felt that the statement and consequent computational procedure is not valid (apparently Statistical Decision Theory might suggest minimizing the average cost).

Resendiz and Herrera present an elaborate probabilistic development for the case of a rectangular centrally-loaded foundation subject to average and tilting settlements of a magnitude to be determined. They interpret the subsoil compressibility as comprising besides the clear-cut changes of properties identified by conventional exploration techniques, a set of random variations, and conclude (with some demonstration based on laboratory data) that both the average settlement and the tilt are normally distributed random functions. As an interesting collateral derivation they demonstrate the marked increase in probability of tilting with increase of rigidity of the building. Two design criteria are developed and exemplified, one based on a certain probability of the average settlement and tilt not exceeding "tolerable limits" (the latter are extensively discussed for tilt), and one based on cost minimization (incidentally, equating the total cost and differentiating, as would be suggested above for the Shuk approach). Assuming that the much more frequently controlling design criterion is on maximum differential settlements (distortions) it is hoped that the method developed will be extended to cover such a case.

7. NEGATIVE SKIN FRICTION ON PILES

A subject that has brought forth the greatest concentration of invaluable contributions is the subject of negative skin friction. Four papers were presented directly to this Session and an additional thirteen were channeled through the Specialty Session specifically organized for appraisal of the subject. Considering the large number of contributions, and the fact that more detailed discussion and appraisal on them will be carried out at the Specialty Session, this General Report will limit itself to a tabulation of the principal points.

As an extremely summarized conclusion it may be stated that, with the exception of some questions with regard to group behaviour, the problem of negative skin friction appears satisfactorily equated: and, above all, as an engineering solution it appears highly recommended to employ the successfully demonstrated techniques for reducing or almost eliminating the heavy overload due to the phenomena.

Author	Case and method	Principal conclusions
1. Poulos and Mattes	Single cylindrical pile of various L/d ratios and stiffnesses K, point-bearing on rigid base. Theoretical solutions for soil as homogeneous isotropic elastic material ($\mu = 0$ and $\mu = 0.5$). Approx. soln. also for elasto-plastic case of local yield pile-soil (adhesion τ_a const. with depth, and increasing linearly with dept).	Downdrag force $P_- = I.E_s.\rho_s.L$ Curves of Influence I reflected marked effect of L/d and K. Consolidation settl. far from pile assumed linear with depth with ρ_s at surface. Distribution of adhesion τ_a has considerable influence. P for given ρ_s is greater for τ_a = const.

Author	Case and method	Principal conclusions
2. Fellenius and Broms	Two separate long instrumented precast piles through soft norm.-cons. clay into silt and sand.	High excess u recorded, and dissipation in 150 days. Negative skin friction observed due to clay reconsolidation $\approx 17\ \%\ \tau_f$ undrained, undisturbed. What remolded reconsolidated strength ?
3. Minou et al.	Four single driven instrumented pipe piles (comparing vertical closed-end point-bearing, ditto battered, vertical open-end point-bearing, vertical closed-end friction pile) within clay and fill with surface settling 15cm/year.	Soil strengths before and after. Skin friction $\approx \tau_f$ UC, but for open-end pile $\approx 60\ \%$. Neutral point (cf. item 4.4) appar. occurs at same relative depth on the four piles. Much data.
4. Bjerrum and Johannessen	Instrumented single steel pipe piles with special point driven to rock, through soft clays under fills.	
	a) Summary of 6 earlier cases	Very high downdrag, skin friction $\approx \bar{\sigma}_v\ (K \tan \phi')$; $K \tan \phi' \approx 0.2$ for clays; ≈ 0.25 for silty clays
	b) Six piles especially for reducing negative friction	Reduction of skin friction by 90 % in bitumen-coated pile with enlarged base to avoil scratching off.
	c) Three piles esp. for reducing negative friction, driven through coarse rock-rubble fill.	Electrosmotic treatment (cathode) moderately effective, very effective at high amperage. Bentonite slurry moderately effective.
5. Begemann H. K. S.	Stiff upper layer (e.g. sand) causing settlement of clay, around single end-bearing pile on rigid base. Approximate solution by formula of point load on infinite plate on elastic foundation.	Proposes limiting downdrag is given by pulling force of the pile.
6. Brons et al.	Instrumented piles driven into end-bearing sand layer: 7 cast-in Situ Vibropiles ; 2 Vibro-casing piles with bentonite-slurry to reduce f; 1 Vibro-casing pile with bitumen-coated casing; 1 tubular steel pile.	About 4-yr observations. Small ρ developed considerable f, steel the same as concrete Vibropiles. Compared approx. with Zeevaert-de Beer formulae computations. The two methods of reducing f virtually eliminated it.
7. De Beer and Wallays	Pulling resistance load tests of several piles through granular strata, accompanied by cone penetrometer tests: Franki piles, with and without enlarged base, piles cast-in-situ in bentonite-stabilized holes, cased piles.	Concerning skin friction, the cone penetrometer result yields conservative estimate of pulling resistance. Bentonite slurry does not influence.

Author	Case and method	Principal conclusions
8. Verruijt	Rigid cylind. pile end-bearing on rigid base, soil under uniform surface loading. Differential eq. of elastic equil. Limit condition of partial slip soil-pile along upper stretch.	For the special assumptions, total downdrag would depend strongly on f, on surface loading, and on square of soil layer thickness.
9. Locher	n rigid piles per unit area on rigid base, soil under uniform surface loading. Computer program for differential equation, adhesion taken as $\tau_a = c' + \sigma_{hz} \tan \phi$.	"Digital analog simulator" computer program.
10. Correa	Description and exemplified design computation for special telescopic pile patented for reducing negative skin friction.	Model of special telescopic joint satisfactorily tested.
11. Sultan	Summary description of problems, including over-simplified analyses of some cases.	
12. Davis and Poulos	Summary of series of elastic theory solutions presented by Poulos, Davis and Mattes on pile behaviour.	Compactly presented tables and graphs of results of theoretical analyses and comparisons with published observations.
13. Resendiz et al.	Design, field-testing, and observation case history of the Palacio de los Deportes, Mexico: 35 piles load tested.	Well-documented case including important problem of negative friction and special piles behaved almost exactly as foreseen.

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